



U.S. Department
of Transportation

**Federal Highway
Administration**

Memorandum

Western Federal Lands Highway Division
610 E. Fifth Street
Vancouver, WA 98661-3801

Date: December 15, 2016

Subject: **Geotechnical Memorandum GM33-16**

Results of Field Investigation & Final Geotechnical Recommendations
WY FLAP Sagebrush Connector Pathway Project
WY FLAP TET TR 200(1)
Grand Teton National Park, Teton County, Wyoming

From: Robert Kraig, Geotechnical Engineer

To: Kevin Gray, Project Manager

Reply to: HFL-17

Introduction

This memorandum provides the results of a field investigation performed in November 2016 and final geotechnical recommendations for the proposed Grand Teton Park WY FLAP Sagebrush Connector Pathway Project. The WFLHD Geotechnical Section previously provided a memorandum (GM 06-16) in February 2016 that provided preliminary geotechnical information and recommendations for the project. The information contained in this memorandum supersedes the information contained in the former.

The proposed project (the project) involves design and construction of approximately 1 mile of paved pedestrian pathway in Grand Teton National Park. Four small box culverts and one corrugated metal pipe culvert are planned for construction in irrigation ditches along the project.

The proposed pathway will connect two pedestrian pathway sections in the area. The proposed pathway will begin at the end of an existing pathway recently constructed by Teton County along the Spring Gulch Road and end near the Gros Ventre Intersection of U.S. Highway 26 (East Side Highway). A GoogleEarth aerial image showing the approximate location of the pathway alignment is presented as Figure 1.

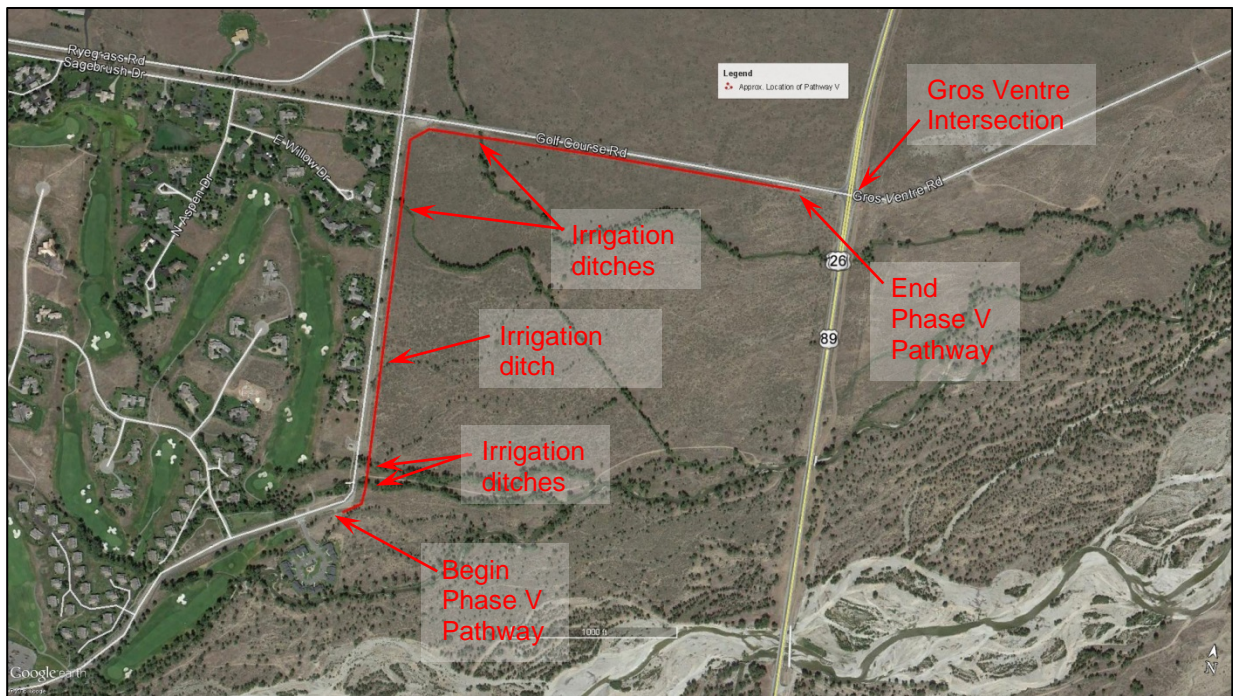


Figure 1 – Aerial photo showing the approximate location of the proposed pedestrian trail

Site Conditions & Results of Field Investigation

Site Conditions

The proposed alignment is located on the relatively flat ground of the Jackson Hole Valley floor. The alignment begins at approximately 6370 feet elevation and ends at about 6400, and crosses five relatively small irrigation ditches with widths between about 5 and 10 feet. The locations of the irrigation ditches are shown on Figure 1.

Results of Field Investigation

Five test pits (TP-1 through TP-5) were dug along the alignment, with one located near the end of the project (TP-1) and the remainder located near the irrigation ditches where culverts are proposed. Test pit locations are shown on Figure 2.

The test pit explorations were dug in early November 2016 using a small rubber-tire backhoe that resulted in test pits that ranged in depth from approximately 3.5 feet (TP-1) to about 6 feet (TP-2 through TP-5). Grab samples were obtained, bagged and sent to the WFLHD Materials Laboratory in Vancouver, WA. Laboratory testing was only completed on two of the grab samples, which were both collected in fine-grained materials from TP-5. Logs of the test pits, along with the material testing results, are contained in Attachment 2.

The test pits encountered native materials through the full depths of the explorations. Groundwater was not encountered in the test pits and the irrigation ditches were dry at the time of the explorations. We estimate that the groundwater table is a few feet above the stream elevation of the nearby Gros Ventre River. According to GoogleEarth (image date 8/2/2013), there is about a 15 foot difference in elevation between TP-5 and the river. It is not known whether the elevation measured at the stream is

the top of stream elevation or the bottom of the stream channel. However, based on this limited information we anticipate that the groundwater table is within several feet of the bottom of the test pit at the location of TP-5, but would fluctuate with seasonal conditions.

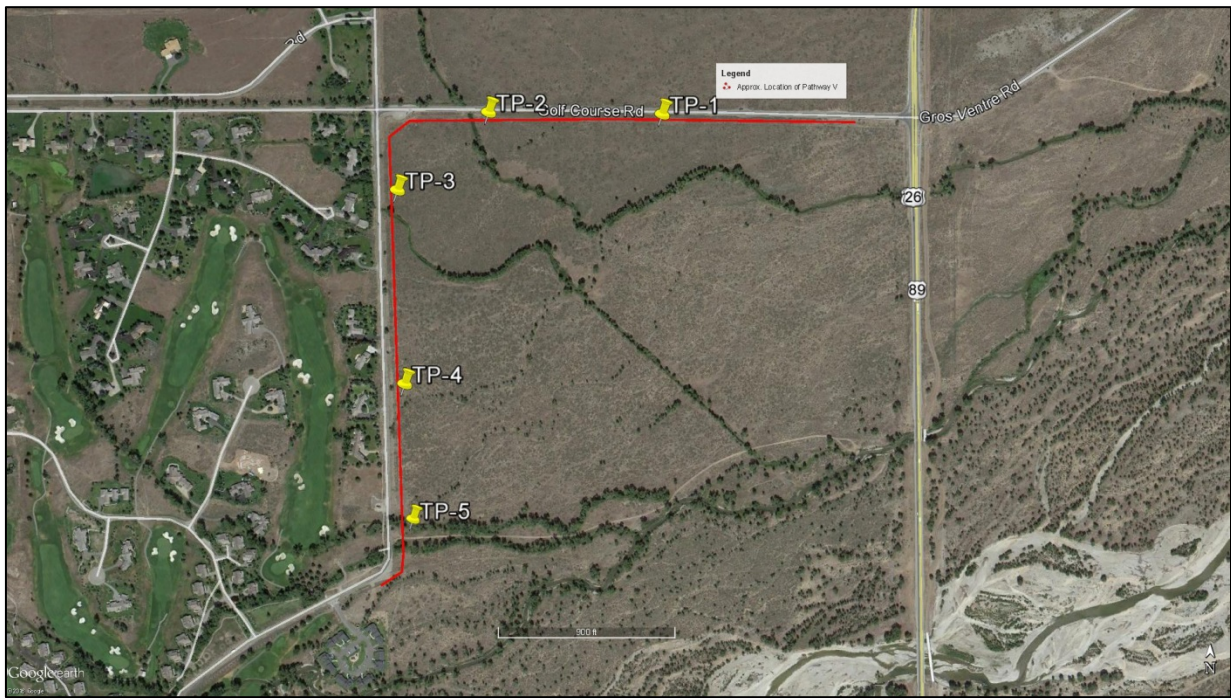


Figure 2 – Aerial photo showing the approximate location of the test pits (Image date 8/2/2013)

The native materials encountered in TP-1 through TP-4 include topsoil to about 0.8 to 1.0 foot underlain by alluvial deposits (Qtg) of the quaternary and (or) tertiary comprised primarily of sandy gravel with cobbles to the bottom of the pits. The Qtg material is shown to be related to the fourth (most recent) glaciation. (See Photos 1 through 12 in Attachment 1 and Figure 3 for an excerpt from the surficial geologic map.) The gravel and cobbles were observed to be subround to round with the cobble size typically measured up to 8 or 9 inches.

Test pit TP-5 encountered topsoil to about 0.7 foot underlain by two layers of lean clay with sand (CL) to about 4.6 feet bgs. (See Photo 13.) The first CL layer was measured to about 2.0 feet bgs. (Note that organics were noted in this layer.) The second CL layer extended to 4.6 feet, where found to be underlain by gravel and cobbles with sand (Photos 13 through 15) to the bottom of the excavation. The samples from both of the clay layers underwent limited laboratory tests and were shown to be slightly to moderately plastic with plasticity indexes (PIs) ranging from 10 to 15, and having R-values slightly above eight, thus indicating weak subgrade support conditions. Laboratory test results are contained in Attachment 2. The gravel and cobbles with sand layer was observed to be somewhat cleaner than the sandy gravel with cobbles layers encountered in TP-1 through TP-4.

As shown on the surficial geologic map, TP-5 (and possibly the entire southernmost section of the project) is located near the boundary of Gros Ventre River flood plain deposits, mapped as modern day alluvium (Qa), with isolated areas of swamp deposits (Qs) mapped nearby (though to the south of the Gros Ventre River as seen on Figure 3). The modern day alluvium includes *“flood-plain deposits comprised of sand and gravel overlain by silt and clay in low terraces; in places inundated during*

modern floods.” The swamp deposits are described as “dark-gray and brown clay, silt, and fine sand; rich in organics.” It is likely that the lean clay layers encountered in TP-5 belong to one of these mapped units. Based on these descriptions it is thought that the upper dark brown clay layer is related to the Qs unit owing to the observance of organics within that layer. Significant organics were not noted in the underlying light brown layer, which may or may not be of the same mapped unit.

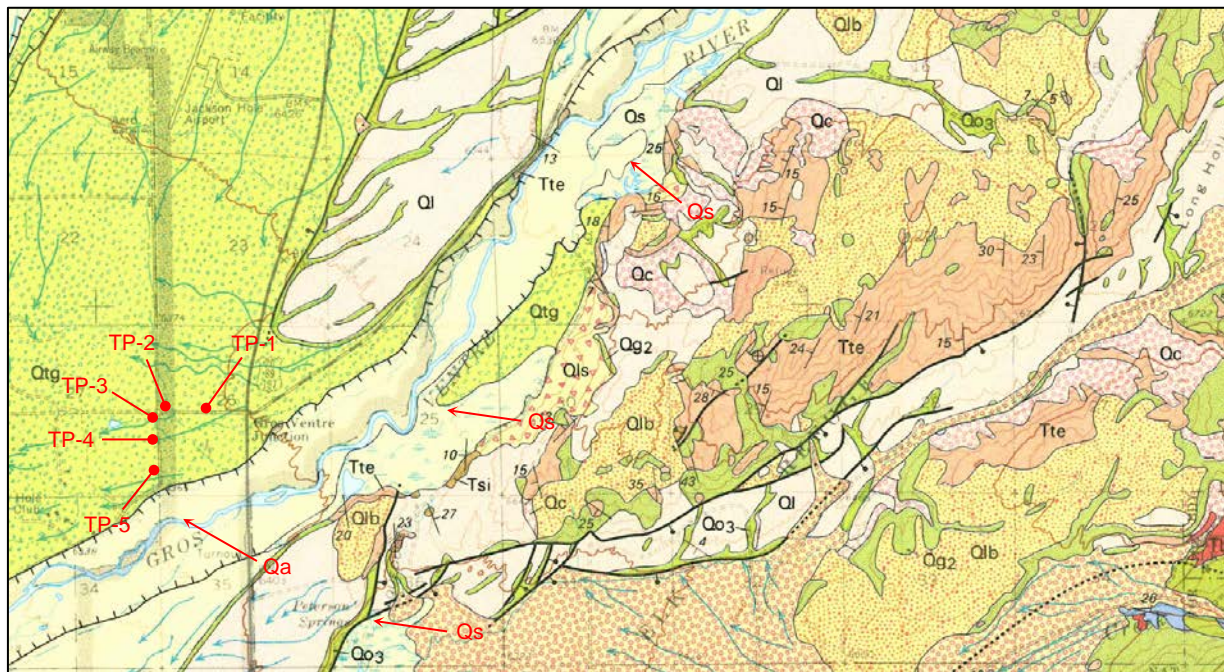


Figure 3 – Excerpt from the surficial geologic map

Final Recommendations

Embedment Depth

The embedment depth for a spread footing or other structure such as a box culvert is based on the anticipated local frost depth. The minimum 3 foot depth appearing in the preliminary recommendations is recommended.

Bearing Capacity

Bearing capacity of a spread footing (i.e., box culvert) was estimated based on presumptive values of 5 kips per square foot in GM 06-16. We consider this bearing capacity to be somewhat conservative for the sandy gravel with cobbles layer. NAVFAC DM 7.02 (Ch. 4, Table 1), shows gravel, gravel-sand mixtures with a consistency in-place of loose to medium-to-compact to have an *Allowable Bearing Pressure* range of 3 to 5 tsf (6 to 10 kips per square foot). Based on the results of the test pit explorations, the presumptive values for bearing capacity on relatively flat ground with a 3 foot embedment depth is now conservatively estimated to be 8 kips per square foot. This estimate applies to all planned culvert locations provided that the culverts are seated within or on the sandy gravel with cobbles layer.

According to NAVFAC DM 7.02, *homogeneous inorganic clay* of soft consistency has an *Allowable Bearing Pressure* of 0.5 tsf (1 kip per square foot). We interpret that the light brown clay layer encountered beneath the dark brown clay layer at a depth of approximately 2.5 feet below ground surface in TP-5 is representative of the *homogeneous inorganic clay* type described by NAVFAC. Bearing capacity of the dark brown clay layer (the upper layer) is not provided in this memorandum owing to the difficulty of correlating organic content in influencing bearing capacity.

Embankment Settlement Estimates

Settlements in coarse-grained granular soils occur primarily from the compression of the soil skeleton due to rearrangement of the soil particles during embankment construction, or other loading.

Settlements in saturated cohesive soil consists of the sum of three components that include immediate settlement (as the embankment is being constructed), consolidation settlement (occurring gradually over time as excess pore pressures generated by embankment construction are dissipated), and secondary compression (essentially controlled by the composition and structure of the soil skeleton occurring over time following the completion of consolidation settlement).

Both coarse-grained granular soils and cohesive soils were encountered in the test pits and will be encountered during construction. Estimated embankment settlements for both types of soils are discussed below.

Station 809+50 to the end of the project (embankments on granular soils)

Immediate, construction related settlement is estimated to be less than ½ inch along the project from about Station 809+50 to the end of the project with no appreciable long-term or differential type settlement expected due to the granular nature of the subsurface material encountered in test pits TP-1 through TP-4. This assumes that no unsuitable foundation conditions are encountered in this section. Unfavorable foundation conditions are discussed under *General Earthwork*.

The beginning of the project to about Station 809+50 (embankments on clay soils)

From the start of the project to approximate Station 809+50, we believe that immediate construction related settlements can be expected in the upper dark brown clay layer where significant organics were noted. The degree of this type of settlement is not easily quantifiable due to the organic content, and the lack of comprehensive laboratory testing. Given that, we provide no estimate for immediate type settlement in the dark brown clay layer.

We estimate that immediate construction type settlement will be relatively insignificant (<0.5 inch) in the relatively thin light brown clay layer based on the relatively shallow embankment fills planned for the project (a few inches to about 2.5 feet) and the relatively shallow thickness of this layer (between 2 and 3 feet).

As discussed earlier, consolidation type settlements occurs over time as excess pore water pressures are dissipated. At the time of the test pit excavations the clay layers were not subject to saturated conditions with natural moisture contents measured at 25.8 percent and 7.5 percent in the dark brown and light brown layers, respectively. We anticipate that the clay layers will not be under saturated conditions during embankment construction, thus no significant opportunity for development of excess water pressures. We provide no estimate for

consolidation settlement for the dark brown clay layer based on the organic content in this layer. Given the planned height of the embankments, the low natural moisture content, and the relatively shallow thickness of the light brown clay layer, we anticipate that consolidation settlement in that layer will be minor and could approach 1 inch.

Estimating the secondary compression is typically determined based on comprehensive and costly laboratory tests of the type considered impractical for the project. However, given the planned height of the embankments and the relatively shallow thickness of the light brown clay layer, we anticipate that Secondary Compression over time will be minor and estimate it at up to 1 inch.

The discussion below assumes that the dark brown organic clay layer will be removed and that the embankments will be constructed on the light brown clay layer.

The primary factor that will determine total settlement in the clay soils is the depth of the embankment fill, currently shown in the plans to range from only a few inches in some locations to a maximum of about 2.5 feet, with the shallow fill sections experiencing virtually no discernible settlement and the deeper fill sections experiencing the most. Based on the earlier discussions, we estimate that the total settlement in the thickest fill sections over the light brown clay layer could be 2.5 inches, or less, over time. For the embankments constructed on clay, we anticipate that the settlements could be differential in nature, thus resulting in a somewhat uneven travel surface over time. Mitigating differential settlement is discussed in the following section.

Embankment Settlement Mitigation

The risk of long term, consolidation, secondary compression, and differential type settlements can be significantly reduced (or nearly eliminated) through the process of surcharging the soils for an extended period prior to construction. This method of mitigation is considered to be impractical given the scope of the project. Alternatively, with the exception of the immediate construction type of settlement, the risk of consolidation and secondary settlements can be eliminated by removing the unsuitable soils and backfilling with acceptable fill, an option that would require subexcavating to about 4 to 5 feet bgs. While effective, full depth subexcavation would result in higher costs and may not prove to be the most cost effective approach.

If reduced settlements are acceptable, we recommend subexcavating the most settlement-prone layer (i.e., the dark brown lean clay layer containing organics) and backfilling with acceptable backfill. Additional long-term strength can be provided to the foundation by incorporating a layer or two of stabilization geogrid. Further support can be provided to the pavement section by placing a stabilization geotextile beneath the base course. The stabilization geogrid will provide additional stability against differential settlement and the separation geotextile will reduce the effect of differential settlements through the worst sections. A conceptual detail showing subexcavation and subgrade support is shown in Figure 4. (Refer to the final contract plan set for final design requirements.) We estimate that settlement through the thickest embankment sections following the use of this method would be in the order of 1 to 1.5 inches or less, over time. Furthermore, the effects of differential settlement as observed at the paved surface (i.e., cracking at the surface of the flexible pavement) would be reduced. These reductions would result from the redistribution of stresses within the embankment by incorporating the reinforcements.

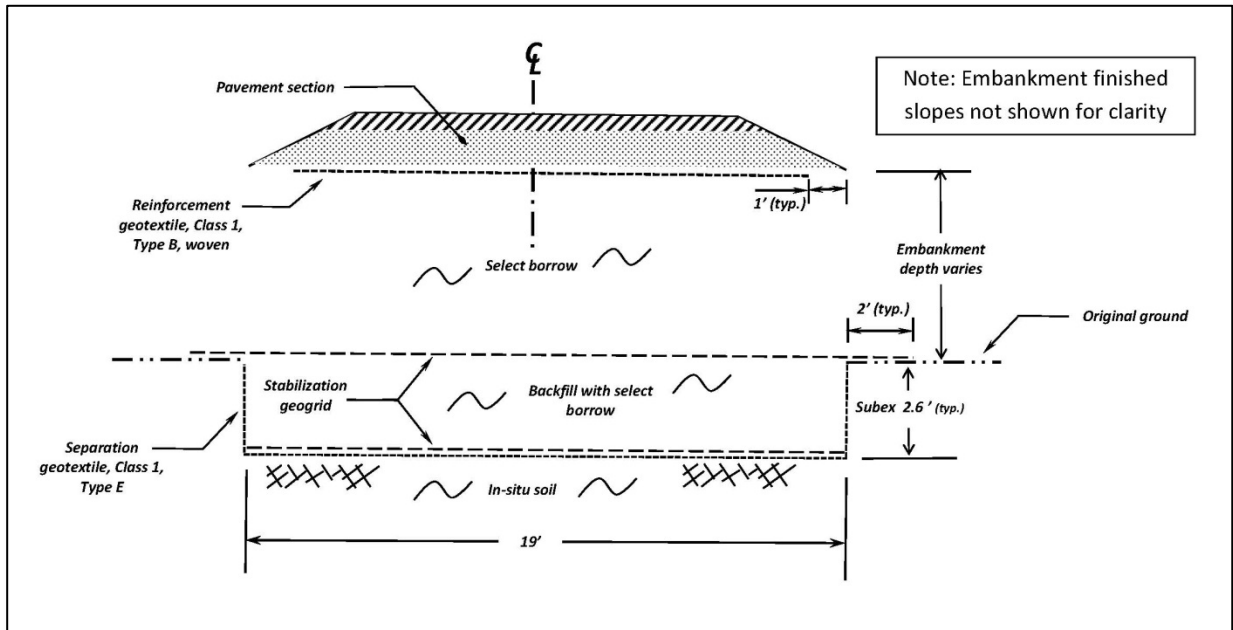


Figure 4 –Subgrade support recommendation detail below Station 809+50

Construction Schedule

We anticipate that the pathway between Station 809+50 and the end of the project can be constructed at any time from a constructability standpoint. However, we recommend that pathway construction in the lean clay soils anticipated from the start of the project to about Station 809+50 occur near the end of the dry season in August or early September when the subgrade soils have had a chance to dry out and precipitation events are less likely. Weather forecasts should be monitored and any open excavations through this section should be protected from rainfall events and possible runoff into the excavations.

General Earthwork.

Removal of topsoil and the dark brown, organic clay layer from the start of the project to approximate station 809+50 is recommended prior to constructing the pathway. Line all subexcavations with a separation geotextile.

The actual limits of the clay layers are unknown along the alignment. Inspection of the available topographic map did not provide much indication. We assume that since it was not encountered in TP-4, it begins at some point between it and TP-5. Based on the limited information available and the surface profile, we assume that the unsuitable clay soils will be encountered from the beginning of the project to Station 809+50.

Although not revealed by test pits TP-1 through TP-4, it is possible that isolated unsuitable subgrade conditions (“soft” spots) might be encountered along the project. We recommend that the contract include an allowance for this possibility—say 600 lineal feet. If unsuitable foundation conditions are encountered, remove the unsuitable soils and backfill with a suitable material. The detail shown in Figure 4 can be used to mitigate these areas.

Culvert Foundations

We anticipate that the excavations for the box culverts proposed at 802+65 and 803+90 will be into one of the lean clay layers identified in TP-5. *Allowable Bearing Pressure* was not provided for the dark brown clay layer (with organics) but was provided as a presumptive value for the light brown clay layer with the actual limit possibly higher, or lower. That said, it might be permissible to seat culverts within the light brown clay layer. However, due to uncertainties as to the actual *Allowable Bearing Pressure*, weak subgrade conditions as shown by low R-values, and the potential for differential settlements to occur in clay soils, we recommend that both of the clay soil layers be subexcavated at the culvert locations and replaced with suitable foundation material such as foundation fill or bedding material. This will require an additional subexcavation of approximately 2 feet below the proposed bottom of the culverts at these two locations.

Summary

This memorandum present the results of subsurface investigation, review of reference materials, and final geotechnical design and construction recommendations for the Grand Teton National Park, Phase V Project. The recommendations appearing herein are intended only for use on this project and should not be used for any other purpose without the explicit approval of the Western Federal Lands Highway Division, Geotechnical Section. For questions or additional information regarding this memorandum you may contact Robert Kraig at robert.kraig@dot.gov.

References:

Geologic Map of Grand Teton National Park, Teton County, Wyoming, Love, Reed, Christiansen, U.S. Geological Survey, 1992.

Soil Survey of Teton County, Wyoming, Grand Teton National Park Area, United Staates Department of Agriculture Soil Conservation Service, April 1982.

Department of the Navy, Naval Facilities Engineering Command, Design Manual 7.02, Foundations and Earth Structures, May 1982

Copies:

Kimber Miller
Carolyn Sourek
Brad Neitzke
Denise Steel
Geotechnical File

Attachment 1

Photographs

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Photo 1: TP-1



Photo 2: TP-1



Photo 3: TP-1



Photo 4: TP-2



Photo 5: TP-2



Photo 6: TP-2



Photo 7: TP-3



Photo 8: TP-3



Photo 9: TP-3



Photo 10: TP-4



Photo 11: TP-4



Photo 12: TP-4



Photo 13: TP-5



Photo 14: TP-5 Gravel with cobbles and some sand layer. Note that much of the finer material observed at the bottom



Photo 15: TP-5 gravel and cobble layer. Note this layer was observed to be cleaner than the other gravelly, cobbly layers encountered on the project.

Attachment 2
Test Pit Logs
Lab Test Results

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TP-1 Station 838+00 (approx.)		43° 34' 26.6" -110° 44' 17.3"
0 – 0.9'	Topsoil (Ziplock sample taken)	
3.5'	Sandy GRAVEL with COBBLES. Light brown, damp. Cobbles are round to subround to about 8" dia., but may go larger. Grab sample taken at 2.5'. Moisture sample taken at 2.5'.	
Notes: None.		

TP-2 Station 828+50 (approx.)		43° 34' 26.7" -110° 44' 29.3"
0 - 0.8'	Topsoil (Ziplock sample taken)	
3.6'	Sandy GRAVEL with COBBLES. Light brown, damp. Cobbles are round to subround to about 8" dia., but may go larger. Grab sample and moisture sample taken at 2.0 feet.	
6.0'	Sand possibly transitions to coarser. Grab sample and moisture sample taken at 4.5 feet.	
Notes: About 30' east of irrigation channel.		

TP-3 Station 820+00 (approx.)		43° 34' 22.8" -110° 44' 35.6"
0 – 1.0'	Topsoil (Ziplock sample taken)	
6.0'	Sandy GRAVEL with COBBLES, dense, brown, damp. Cobbles to 8". Grab samples and moisture samples taken at 4.0 feet and 6.0 feet.	
Notes: About 30' north of irrigation channel.		

TP-4 Station 810+70 (approx.)		43° 34' 13.1" -110° 44' 35.1"
0 – 0.9'	Topsoil (Ziplock sample taken)	
6.0'	Sandy GRAVEL with COBBLES, dense, brown, damp. Cobbles to 8". Grab sample taken at about 5.0 feet. Moisture sample taken at about 4.0 feet.	
Notes: About 30' north of irrigation channel.		

TP-5 Station 803+70 (approx.)		43° 34' 6.4" -110° 44' 34.5"
0 – 0.7'	Topsoil (Ziplock sample taken)	
2.0'	Lean CLAY, brown, damp, organics. Grab sample taken at about 1.5 feet. Moisture sample taken at about 1.3 feet.	
4.6'	Lean CLAY, light brown, dry. Grab sample taken at 3.0 feet. Moisture sample taken at 3.0 feet.	
6.0'	GRAVEL and COBBLES with some sand, damp. Cleaner than the sandy gravel layers encountered in the other pits. Operator said excavated like very dense material. No samples retrieved.	
Notes: About 15' south of irrigation channel.		



Western Federal Lands Highway Division
Materials Testing Laboratory
610 E. Fifth St, Vancouver, WA 98661

Test Report Issued: **30 Nov 2016**
Lab Control Number: **W-16-1412-SO**



Project Name: GRTE PH-V Connection

Project Number: WY NPS GRTE 700(5)

Acct. No.: 15A7561670005 R10.PE.15F0.56

Sample No:

Sampled By: Kraig

Date Sampled:

Submitted By: Robert Kraig

Phone: 360-619-7699

Address:

Sample of:

Quantity Rep:

Date Received: 11/15/2016

No. & Containers: 1 canvas bag/1

Dates Tested: 11-15-2016 to 11-22-2016

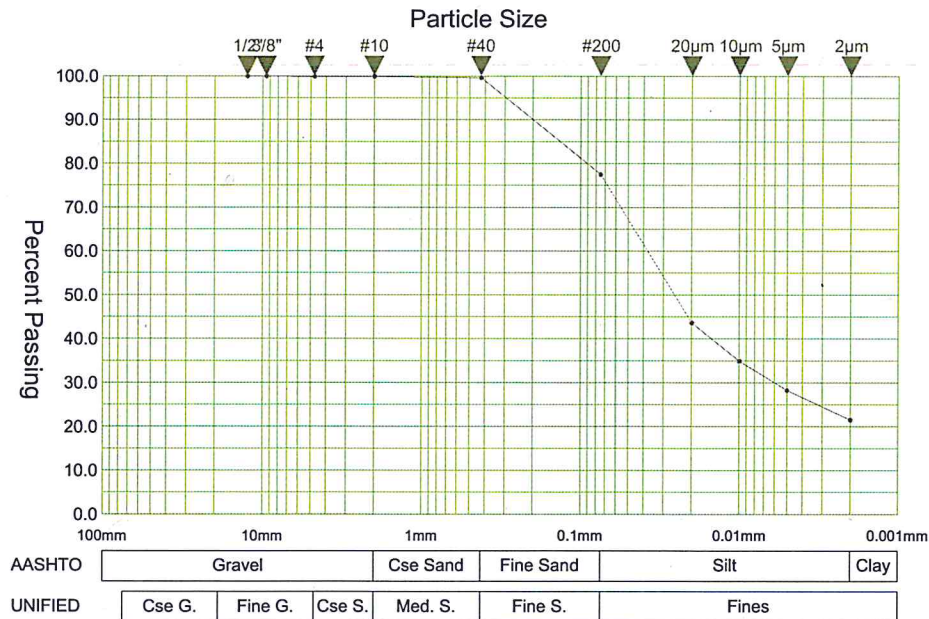
Boring No./Test Pit: TP-5

Depth: 1.5

Sieve Analysis

As Received
Sieve Size % Passing

1/2"	100.0
3/8"	100.0
#4	99.9
#10	99.9
#40	99.6
#200	77.5
20µm	43.7
10µm	35.0
5µm	28.3
2µm	21.6



Soil Classification (DL145)

AASHTO
Unified

A-6(11) GR-SA-CLAY
CL; Lean clay with sand

Apparent Specific Gravity (T100)

2.598


Natural Moisture (T265) (Sample dried at 230 °F), %	25.8
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Atterberg Limits (T89)

Liquid Limit	38
Plasticity Index	15

R Value Results (WL190)

R Value By Exudation at 300 psi	8.3
Density At R Value, pcf	100.4
Moisture at R Value, %	18.3



Walt Stong, Materials Laboratory Chief
For: Brad K. Neitzke, Materials Engineer



Western Federal Lands Highway Division
Materials Testing Laboratory
610 E. Fifth St, Vancouver, WA 98661

Test Report Issued: **30 Nov 2016**
Lab Control Number: **W-16-1413-SO**

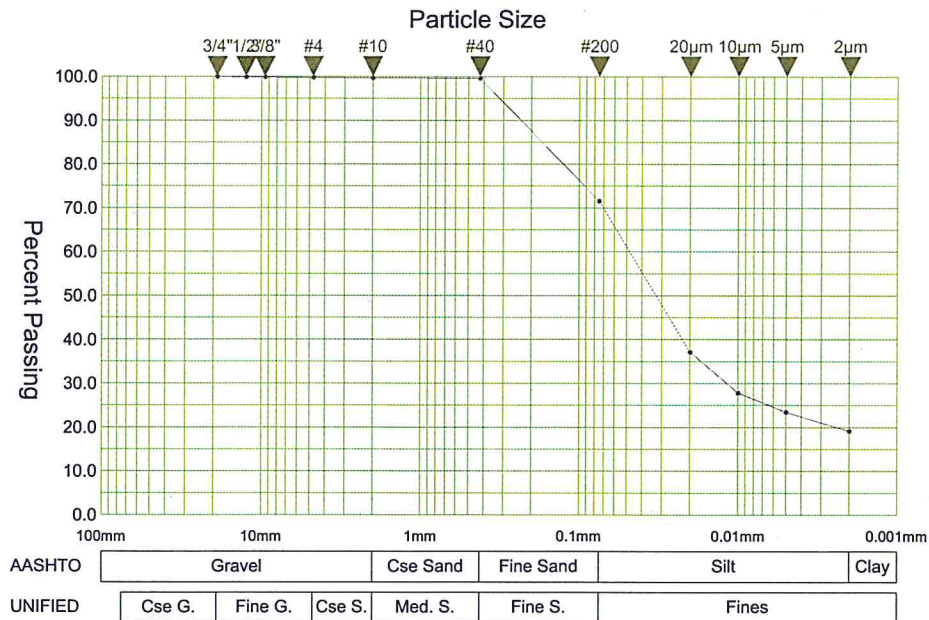


Project Name: GRTE PH-V Connection	Sample No:
Project Number: WY NPS GRTE 700(5)	Sampled By: Kraig
Acct. No.: 15A7561670005 R10.PE.15F0.56	Date Sampled:
Submitted By: Robert Kraig	Address:
Phone: 360-619-7699	
Sample of:	Date Received: 11/15/2016
Quantity Rep:	No. & Containers: 1 canvas bag/1
	Dates Tested: 11-15-2016 to 11-22-2016
Boring No./Test Pit: TP-5	Depth: 3.0

Sieve Analysis

As Received
Sieve Size % Passing

3/4"	100.0
1/2"	99.9
3/8"	99.9
#4	99.8
#10	99.7
#40	99.6
#200	71.6
20µm	37.2
10µm	27.9
5µm	23.5
2µm	19.2



Soil Classification (DL145)

AASHTO
Unified

A-4(6) GR-SA-SILT
CL; Lean clay with sand

Apparent Specific Gravity (T100)

2.612


Natural Moisture (T265) (Sample dried at 230 °F), %	7.5
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Atterberg Limits (T89)

Liquid Limit	32
Plasticity Index	10

R Value Results (WL190)

R Value By Exudation at 300 psi	8.2
Density At R Value, pcf	100.6
Moisture at R Value, %	16.9



Walt Stong, Materials Laboratory Chief
For: Brad K. Neitzke, Materials Engineer