GEOTECHNICAL INVESTIGATION REPORT
NEW PATHWAY ALONG NORTH HIGHWAY 89
JACKSON, WYOMING

Prepared for:

Thomas Kirsten
Jorgensen Associates
P.O. Box 9550
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June 10, 2009

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June 10, 2009

Thomas Kirsten
Jorgensen Associates
P.O. Box 9550
Jackson, Wyoming 83001

RE: GEOTECHNICAL INVESTIGATION REPORT FOR NEW PATHWAY ALONG NORTH HIGHWAY 89, NORTH OF JACKSON, TETON COUNTY, WYOMING

Dear Thomas:

We are pleased to present this report on our geotechnical investigation for the proposed new pathway along North Highway 89 north of Jackson, Wyoming. Three copies are enclosed. The report describes site conditions and presents conclusions and recommendations.

If you have any questions about this report, or if we may provide other services to you, please contact us. As the project progresses, we will be available to answer questions for you.

Respectfully submitted,

WOMACK & ASSOCIATES, INC.

Professional Engineer (Civil)
WESLEY R. WOMACK
4958
Date
WYOMING

Ray Womack 6-10-09

Ray Womack, P.E., P.G.

Enclosure: Reports

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1.0 INTRODUCTION

As authorized by Thomas Kirsten of Jorgensen Associates, Womack & Associates, Inc. (WAI) conducted a geotechnical site investigation for the proposed new pathway to be constructed along the east side of North Highway 89, north of Jackson, Wyoming (Figure 1). The purposes were to investigate surface and subsurface soil conditions, evaluate slope conditions and soil-engineering properties, and to provide recommendations for foundations, retaining walls, and construction.

2.0 PROPOSED CONSTRUCTION

The new paved pathway will be constructed along the east side of Highway 89, north of Jackson from the Flat Creek bridge to the Gros Ventre River bridge. A new pathway bridge will cross the Gros Ventre River on the east side of the existing vehicle bridge. An underpass will be constructed to accommodate access to the National Museum of Wildlife Art (NMWA). There are two proposed phases of design and construction: Phase I will extend from the north end of Flat Creek bridge to the developed pullout located approximately 0.7 miles northeast of the Grand Teton National Park (GTNP) boundary. Phase II will extend from the developed pullout to the Gros Ventre River bridge, and will also include construction of the underpass at the NMWA.

Based on preliminary maps provided by Jorgensen Associates (JA), it appears that the new pathway will follow existing topography/existing grade for the majority of its length. Small cuts and fills will be required in locations where the current grade does not match finished pathway grade, or where compressible soft clays underlie the pathway. Existing slopes have variable gradients: the northern and southern ends of Phase I and II are relatively flat along the Flat Creek alluvial flood plain and along the glacial terrace above the Fish Hatchery; steeper segments along the highway near the Fish Hatchery have slopes of almost 12%. It is our understanding that a cut section located north of Fish Hatchery Road, is planned for the pathway and will provide borrow material.

Three retaining walls will be utilized along the pathway prism (refer to JA Pathway Plan and Profile Sheets for station locations): The Starting Point Wall from stations 0+35 to 1+60 will be approximately 300 square feet in size with a height of 2 to 3 feet, the Wetland Wall from stations 76+90 to 79+88.6 will be approximately 1,200 square feet in size with a height of 4 to 5 feet, and the Fish Hatchery Wall from stations 160+20 to 167+71.9 will be approximately 5,000 square feet in size with a height of 7 to 8 feet. It is our understanding that a variety of retaining walls are being considered for these three locations: MSE, gravity, concrete, or Gabion, and that Hilfiker Gabion walls are the most likely prospect at this time.

SCALE (In Feet) 0 150 APPROXIMATE

Stationing is approximate

Geotechnical Site Investigation
North Highway 89 Pathway
Teton County, Wyoming

Borehole and Test Pit Locations: BH-1, TP-1, TP-2

FIGURE 1A

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SCALE (In Feet) 0 150 APPROXIMATE

Stationing is approximate

Geotechnical Site Investigation
North Highway 89 Pathway
Teton County, Wyoming

Flat Creek Motel WDEQ
Monitoring Well Locations

FIGURE 1B

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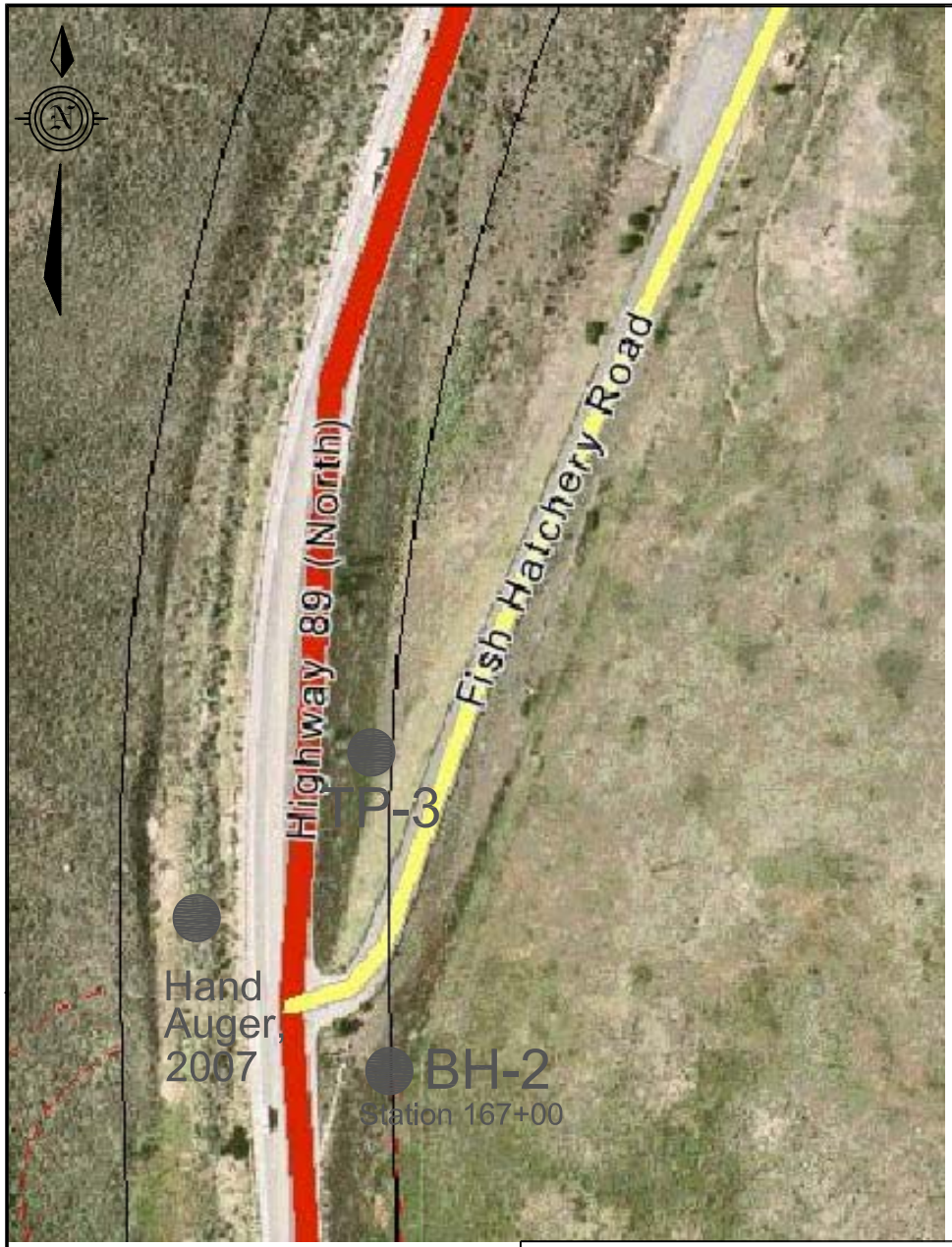

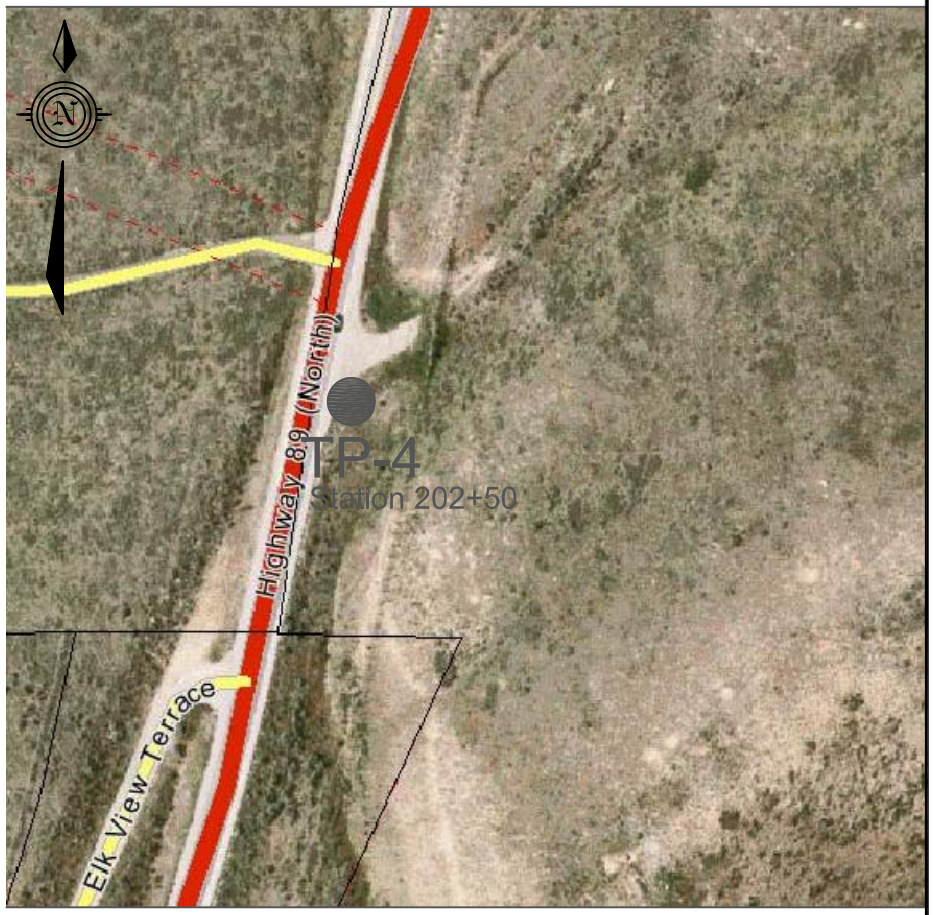

SCALE (In Feet) 0 150 APPROXIMATE

Stationing is approximate

Geotechnical Site Investigation
North Highway 89 Pathway
Teton County, Wyoming

Borehole and Test Pit Locations: BH-2, TP-3, TP-4

FIGURE 1C

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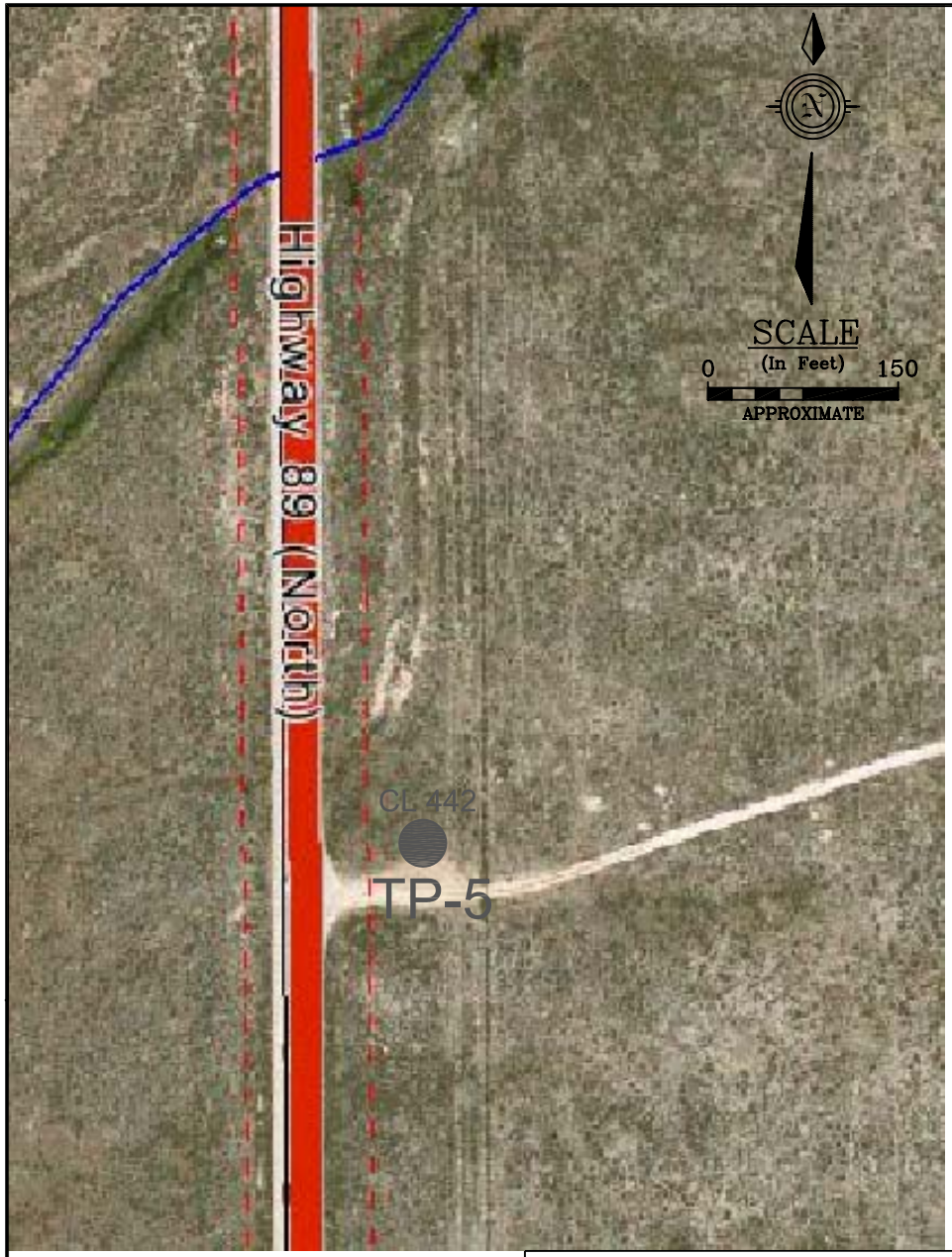

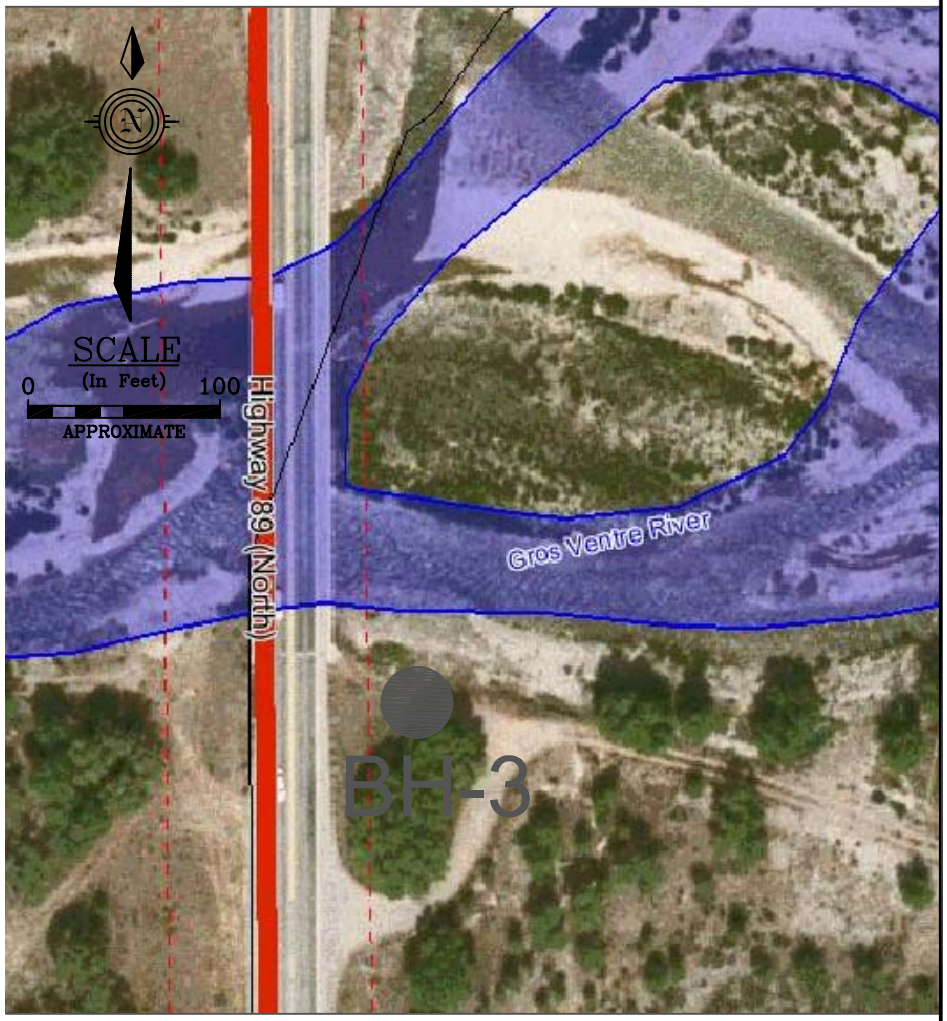

Stationing is approximate

Geotechnical Site Investigation
North Highway 89 Pathway
Teton County, Wyoming

Borehole and Test
Pit Locations:
BH-3, TP-5

FIGURE
1D

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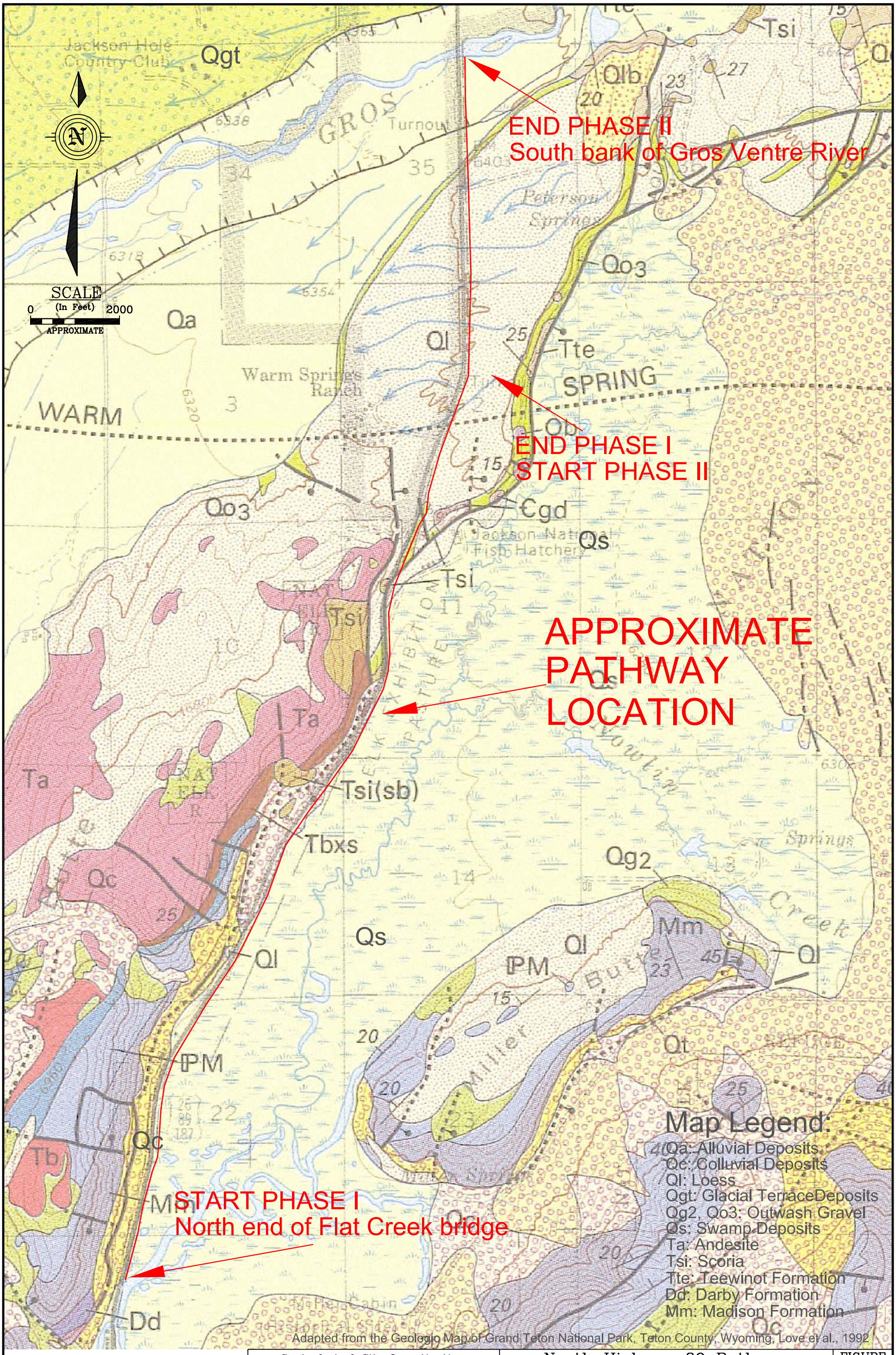

Adapted from the Geologic Map of Grand Teton National Park, Teton County, Wyoming, Love et al., 1992

Geotechnical Site Investigation North Highway 89 Pathway Teton County, Wyoming	North Highway 89 Pathway Geologic Map	FIGURE 2

UPDATE TIME: \ \ \dks \jh \060509\w\c\jorgensen\North89Pathway\Geo Map.dwg

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3.0 INVESTIGATION PROCEDURES

3.1 Field Investigation

The field investigation consisted of logging and sampling of three geotechnical boreholes and five exploratory test pits, and installation of one monitoring well. The boreholes were drilled and the monitor well was installed on May 13, 2009, and the test pits were excavated on May 21, 2009. Borings were drilled to depths between 16.5 feet below the ground surface (bgs) and 26.5 feet bgs. The test pits were placed along the pathway alignment and excavated to depths of 4.5 to 11.9 feet in order to observe subgrade soil and groundwater conditions and to obtain representative samples. The boreholes were placed at geotechnically-sensitive areas: the underpass across from the NMWA (Figure 1A), within the fill prism along the highway ramp by the Fish Hatchery (Figure 1C), and at the Gros Ventre River bridge (Figure 1D). Access and mitigation constraints precluded subsurface exploration on the north side of the Gros Ventre River (in GTNP), as such, exploration at the bridge consists of one borehole southeast of the bridge. A monitoring well was installed in the boring associated with the underpass at the Wildlife Museum (NWMA). Test pit and borehole locations are shown on Figures 1A through 1D. Descriptive logs of the test pits and boreholes are attached in Appendix A.

Soil types, consistencies, and stratigraphic thicknesses were observed and documented by a Geotechnical Engineer and an Engineering Geologist. Representative samples were obtained from critical soil horizons. Note that site conditions are variable and actual soil conditions may differ from those represented in the test pit logs.

The Wyoming Department of Environmental Quality (WDEQ) has installed monitoring wells around the Flat Creek Motel (also called the Red Barn Site) for the LAUST project, 7 of which are in close proximity to the pathway prism along the east side of the highway (Figure 1B). The pertinent WDEQ monitoring well logs along the pathway are shown on Figure 1B, attached in Appendix B, and are discussed in further detail in Section 4.3.

3.2 Laboratory Analysis

Laboratory testing for this investigation included: moisture content, classification, grain size analysis, Atterberg Limits, and a consolidation test from an undisturbed clay-silt sample taken at test pit TP-3; corrosion resistance testing was performed on two disturbed samples taken from boreholes BH-1 and BH-3. Laboratory test results are attached in Appendix C.

3.3 Report Preparation

The report describes the geological site conditions and includes a site layout and geologic map, test pit and borehole location map, test pit and borehole logs, and laboratory test results. The report also provides engineering analyses and recommendations for construction.

4.0 NATIONAL MUSEUM OF WILDLIFE ART UNDERPASS

An underpass will be constructed during Phase II of the project for access from the new pathway to the NMWA. The underpass will be located at station 122+00, and based on information from JA, footings will be approximately 10 to 12 feet below highway grade.

4.1 Geology and Soils

Borehole BH-1 was drilled just south of the proposed underpass location at station 122+00, and test pit TP-2 was located across Highway 89 from station 122+00. BH-1 reached a depth of about 27 feet bgs, and encountered about 5 feet of gravel fill overlying about 4 feet of clayey silt, which in turn overlies about 3.5 feet of silty sand. TP-2 reached a depth of about 10 feet bgs, and encountered topsoil overlying about 1.5 feet of clayey silt (or loess), which in turn overlies colluvial sandy gravels, cobbles, and boulders. It is our understanding that tunnel footings will be placed at about 10 to 12 feet below current highway grade (approximate grade at which depths in BH-1 were measured from). Based on information from our field investigation, footings on the western side of Highway 89 will be placed on coarse-grained colluvium, and footings on the eastern side of Highway 89 on silty sand. However, coarse gravel alluvium was encountered in BH-1 at a depth of 12.5 feet bgs, and it appears that a very slight over-excavation would be required to reach suitable bearing material.

4.2 Groundwater

BH-1 encountered groundwater at 15.0 feet bgs at time of drilling (ATD). Groundwater was measured in BH-1 on May 21, 2009 and June 10, 2009, and measured 10.8 and 9.8 feet bgs, respectively. Based on information from JA, underpass footings will be placed at about 10-12 feet bgs, and footings may be in groundwater seasonally.

4.3 Bearing Capacity

Bearing capacities were estimated using methods suggested by Bowles (1996). Presumptive pressures were derived based on visual classification of the soils. The allowable bearing capacity on coarse granular soils is approximately 5,000 psf and approximately 1,000 psf for fine-grained soils.

4.4 Settlement

It appears that underpass footings may be in coarse-grained colluvium on the west side of Highway 89, and in silty sand or fine-grained material on the east side of the highway. If the underpass crosses into different soil types, differential settlement may become a problem. However, BH-1 encountered coarse-grained alluvium at a depth of 12.5 feet bgs, showing that a small over-excavation and replacement with pit run may be necessary to place footings on suitable bearing materials. Significant consolidation of the coarse-grained colluvium is not expected. Topsoil, loess, and other fine-grained material are prone to consolidation and should be removed, if possible.

4.5 Lateral Earth Pressures

Lateral pressures on the underpass from earthquakes were estimated using the Mononobe-Okabe equations (Bowles, 1996). Because the maximum acceleration occurs only briefly during an earthquake, it is common practice when designing earth structures to reduce the design acceleration to ½ of the maximum design acceleration (Hynes and Franklin, 1984). We have calculated equivalent fluid pressures using a horizontal acceleration k_h of 0.11g (1/2 of k_h max) for the site. Lateral pressure design parameters for variable conditions of slope, seismic conditions, and wall configurations are summarized in the following tables. The values in the tables below assume that either fine-grained site material, i.e. clay or clayey silt, or coarse grained site material, i.e. sand and sandy gravels and cobbles, will be used as exterior backfill.

Equivalent fluid pressures (γK) will vary based on the slope of the ground surface adjacent to the foundation wall. Lateral pressure increases when the ground surface slopes toward the wall and decreases when the ground surface slopes away from the wall. Lateral pressures were calculated for at rest, active, and passive conditions with level backfill.

Table 4-1: NMWA Underpass – Lateral Pressure Parameters – Fine-Grained Materials

Condition	Coefficient of Earth Pressure*	γK (equivalent fluid pressure)$^+$
Static Conditions Level Backfill	K_o = 0.53 K_a = 0.36 K_p = 2.77	γK_o = 64 pcf γK_a = 43 pcf γK_p = 332 pcf
Earthquake Conditions Level Backfill Level Backfill	K_{ae} = 0.44 K_{pe} = 2.57	γK_{ae} = 52 pcf γK_{pe} = 309 pcf

*Assumes a unit weight for the backfill of 120 psf and a friction angle of 28 degrees.

Table 4-2: NMWA Underpass – Lateral Earth Pressure Parameters – Coarse-Grained Materials

Condition	Coefficient of Earth Pressure	γK (equivalent fluid pressure)
Static Conditions Level Backfill	$K_o = 0.46$ $K_a = 0.29$ $K_p = 3.39$	$\gamma K_o = 61$ pcf $\gamma K_a = 40$ pcf $\gamma K_p = 458$ pcf
Earthquake Conditions Level Backfill Level Backfill	 $K_{ae} = 0.36$ $K_{pe} = 3.18$	 $\gamma K_{ae} = 49$ pcf $\gamma K_{pe} = 429$ pcf

*Assumes a unit weight for the backfill of 135 pcf and a friction angle of 33 degrees.

4.6 Soil Friction

Terzaghi, et al (1996) suggests use of a maximum of 30 degrees for the friction angle along a concrete base in granular soils. Accordingly, a friction value of 0.58, which is the tangent of 30 degrees, is suggested. The friction value may be combined with the passive pressure to resist horizontal loads.

5.0 RETAINING WALLS

Three retaining walls will be utilized along the pathway prism (refer to JA Pathway Plan and Profile Sheets for station locations): The Starting Point Wall from stations 0+35 to 1+60 will be approximately 300 square feet in size with a height of 2 to 3 feet, the Wetland Wall from stations 76+90 to 79+88.6 will be approximately 1,200 square feet in size with a height of 4 to 5 feet, and the Fish Hatchery Wall from stations 160+20 to 167+71.9 will be approximately 5,000 square feet in size with a height of 7 to 8 feet. It is our understanding that a variety of retaining walls are being considered for these three locations: MSE, gravity, concrete, or Gabion, and that Hilfiker Gabion walls are the most likely prospect at this time.

5.1 Geology and Soils

The Starting Point Wall is most likely underlain by soft, organic clays and peat. No test pits were completed near this wall, and the closest test pit, TP-1, is underlain by coarse-grained debris fan material.

Seven WDEQ monitoring wells were installed in 1997 and 2002 between stations 61+00 to 69+00: MW-118 through MW-122, which were drilled to depths of 18 to 23 feet bgs, MW-512 (drilled to about 25 feet bgs), and MW-610 (drilled to 10 feet bgs). These monitoring wells encountered approximately 3 feet of sand and gravel fill overlying silt and/or clay. MW-119 through MW-121 encountered sand and/or gravel below the silt at about 14 to 20 feet bgs; all

other WDEQ monitoring wells did not penetrate past fine-grained material. WDEQ monitoring well locations are shown on Figure 1B and logs are attached in Appendix B.

The Fish Hatchery Wall is most likely underlain by clay and gravel fill, which in turn is probably underlain by soft to medium stiff clay-silt. BH-2 was located just east (downhill) of 167+00 at the toe edge of the highway fill prism. BH-2 was drilled to a depth of 21.5 feet bgs, and encountered 5 feet of soft clayey silt topsoil/alluvium overlying more than 16 feet of alluvial sand and gravel deposits. Fill was not encountered in the borehole. TP-3 was located directly north of the intersection with North Highway 89 and Fish Hatchery Road in a topographic low between the two fill prisms. TP-3 was excavated down to about 12 feet bgs, and encountered 4 feet of sandy clay and gravel fill overlying soft to medium stiff clayey silt. The test pit did not penetrate the clayey silt.

5.2 Groundwater

Groundwater in BH-2, located below the Fish Hatchery Wall at the toe of the highway fill prism, was measured at 6.0 feet bgs ATD. We do not anticipate groundwater to be of concern at the Fish Hatchery Wall because of the wall's position up the slope. Groundwater may be shallow at the other two retaining walls, and proper drainage practice should be observed. Retaining wall drainage is discussed further in Section 8.5.

5.3 Bearing Capacity

Bearing capacities were estimated using methods suggested by Bowles (1996). Presumptive pressures were derived based on visual classification of the soils. The allowable bearing capacity on coarse granular soils (fill, gravels and cobbles) is approximately 5,000 psf on flat ground. However, the bearing capacity is approximately 3,000 psf at the Fish Hatchery Wall because the bearing capacity is reduced for slope. The bearing capacity for fine-grained material is approximately 1,000 psf. The bearing capacity at the Starting Point and Wetlands Walls is approximately 1,000 psf.

5.4 Settlement

Topsoil and loess are prone to consolidation and should be removed, if possible, below the pathway prism. We strongly recommend removing fine-grained material beneath structures.

Soft to medium stiff silt or clay-silt may underlie the two southern retaining walls (the Wetland Wall and The Starting Point Wall) and clayey silt underlies the Highway 89 fill prism at the Fish Hatchery Wall. Fine-grained material may be on the order of 15 feet thick at the two southern walls according to WDEQ well logs, and BH-2 encountered about 5 feet of clayey silt near the Fish Hatchery Wall.

Settlement calculations assumed 15 feet for the two southern walls and 10 feet for the Fish Hatchery Wall; 10 feet was used rather than 5 feet in order to be consistent with thicker deposits that are thought to exist in proximity to BH-2, as documented in TP-3 and a hand-auger hole

completed in 2007 for another project, which is located approximately 200 feet from BH-2. BH-2 may be located on the distal end of the outwash deposits (Qg2) mapped by Love et al. (1992).

The Fish Hatchery Wall will be constructed on the upper portion of the existing Highway 89 fill slope (approximately 1.6H:1V), and although the road fill has already loaded the native soil, the wall will produce an additional surcharge. The exposed portion of the wall may be as tall as about 8 feet, and the toe may be about 12 feet vertically up the slope (above native ground). Because of this geometry, an average surcharge thickness of 4.5 feet (height at centroid of triangular fill wedge) was used to model the affects of the triangular fill wedge on the underlying highway fill and native fine-grained topsoil/alluvium. The highway fill was considered to be incompressible.

According to the calculations, settlement on the order of 1 to 2 inches is predicted at the Fish Hatchery Wall in the underlying silt material, given the above geometry, assumptions, and consolidation characteristics of the soil.

For the Starting Point and the Wetland Walls, a fill thickness of 4 feet was assumed, bearing directly on compressible material. Consolidation may be on the order of 3 inches at these locations. This estimate is higher than that estimated for the Fish Hatchery Wall because a thicker compressible layer was assumed and there is no intermediate incompressible material bridging the new fill and compressible material.

5.5 Lateral Earth Pressures

Lateral pressures were calculated using methods suggested by Bowles (1996). Equivalent fluid pressures (γK) will vary based on the slope of the ground surface adjacent to the foundation wall. Lateral pressure increases when the ground surface slopes toward the wall and decreases when the ground surface slopes away from the wall. Lateral pressures were calculated for at rest, active, and passive conditions with level backfill. Only passive parameters were calculated for sloping conditions (slope below) during an earthquake. The passive sloping conditions assume that the ground surface slope adjacent to the retaining walls is 29 degrees (2H:1V) or less along the highway fill slope, as is the case at the Fish Hatchery Wall. The other two walls will have less steep adjacent backfill slopes, and we assume that the backfill at the underpass to the NMWA will be level.

Because the maximum acceleration occurs only briefly during an earthquake, it is common practice when designing earth structures to reduce the design acceleration to ½ of the maximum design acceleration (Hynes and Franklin, 1984). We have calculated equivalent fluid pressures using a horizontal acceleration k_h of 0.11g (1/2 of k_h max) for the site. Lateral pressure design parameters for variable conditions of slope, seismic conditions, and wall configurations are summarized in the following tables. The values in the tables below assume that either fine-grained site material, i.e. clay or clayey silt, or coarse grained site material, i.e. sand and sandy gravels and cobbles, will be used as exterior backfill.

For retaining walls, which are allowed to deflect and develop a full active soil wedge, lateral pressure design should utilize active seismic pressures (γK_{ae}). The tables include design values for both level and passive sloping ground adjacent to the wall.

Table 5-1: Retaining Walls – Lateral Pressure Parameters – Fine-Grained Materials

Condition	Coefficient of Earth Pressure*	γK (equivalent fluid pressure)$^+$
Static Conditions Level Backfill	K_o = 0.53 K_a = 0.36 K_p = 2.77	γK_o = 64 pcf γK_a = 43 pcf γK_p = 332 pcf
Earthquake Conditions Level Backfill Level Backfill	K_{ae} = 0.44 K_{pe} = 2.57	γK_{ae} = 52 pcf γK_{pe} = 309 pcf

*Assumes a unit weight for the backfill of 120 psf and a friction angle of 28 degrees.

Table 5-2: Retaining Walls – Lateral Earth Pressure Parameters – Coarse-Grained Materials

Condition	Coefficient of Earth Pressure	γK (equivalent fluid pressure)
Static Conditions Level Backfill Slope Below (-29^0)	K_o = 0.46 K_a = 0.29 K_p = 3.39 K_p = 1.12	γK_o = 61 pcf γK_a = 40 pcf γK_p = 458 pcf γK_p = 151 pcf
Earthquake Conditions Level Backfill Level Backfill Slope Below (-29^0)	K_{ae} = 0.36 K_{pe} = 3.18 K_{pe} = 0.95	γK_{ae} = 49 pcf γK_{pe} = 429 pcf γK_{pe} = 128 pcf

*Assumes a unit weight for the backfill of 135 pcf and a friction angle of 33 degrees.

5.6 Soil Friction

Terzaghi, et al (1996) suggests use of a maximum of 30 degrees for the friction angle along a concrete base in granular soils. Accordingly, a friction value of 0.58, which is the tangent of 30 degrees, is suggested. These values apply the Fish Hatchery Wall, which will be installed within granular sandy clay and gravel fill.

A friction value of 0.53, which is the tangent of 28 degrees, is suggested in clays and clayey silts. The Starting Point Wall and the Wetland Wall will most likely be installed within clays and clayey silts. The friction values may be combined with the passive pressure to resist horizontal loads.

6.0 GROS VENTRE RIVER BRIDGE

The new Gros Ventre River bridge is located at the northern limit of the new pathway, and will be completed during Phase II construction. The bridge is proposed to run along the east side of the existing vehicle bridge.

6.1 Geology and Soils

BH-3 was located at the southeastern corner of the existing Gros Ventre River bridge, and reached a depth of 16.5 feet bgs. BH-3 encountered 4 feet of gravel and cobble fill overlying silty sandy gravel and cobble alluvial deposits.

6.2 Groundwater

Groundwater in BH-3, located near the Gros Ventre River bridge, was measured at 8.3 feet bgs ATD. It should be noted that 4 feet of fill overlies the native ground at BH-3. According to the USGS National Water Information System Web Interface flow levels and stage height in the Gros Ventre River peaked thus far this year on June 3, 2009. Stage height increased from about 3.2 feet on May 13 (ATD) to about 6.5 feet on June 3 – as such a similar increase in groundwater elevation could be expected.

6.3 Bearing Capacity

Bearing capacities were estimated using methods suggested by Bowles (1996). Presumptive pressures were derived based on visual classification of the soils. Bearing capacity for the coarse-grained alluvium is estimated to be approximately 5,000 psf.

6.4 Settlement

Significant consolidation of the coarse-grained alluvial and glacial terrace deposits at the new Gros Ventre River bridge is not anticipated. We strongly recommend removing any fine-grained material encountered beneath abutment foundations.

6.5 Lateral Earth Pressures

Lateral pressures for the bridge abutments from earthquakes were estimated using the Mononobe-Okabe equations (Bowles, 1996). Because the maximum acceleration occurs only briefly during an earthquake, it is common practice when designing earth structures to reduce the design acceleration to ½ of the maximum design acceleration (Hynes and Franklin, 1984). We have calculated equivalent fluid pressures using a horizontal acceleration k_h of 0.11g (1/2 of k_h max) for the site. Lateral pressure design parameters for slope, seismic conditions, and wall

configurations are summarized in the following tables. The values in the tables below assume that coarse-grained site material, i.e. sandy gravels and cobbles, will be used as horizontal exterior backfill.

Lateral pressures were calculated using methods suggested by Bowles (1996). Equivalent fluid pressures (γK) will vary based on the slope of the ground surface adjacent to the foundation wall. Lateral pressure increases when the ground surface slopes toward the wall and decreases when the ground surface slopes away from the wall. Lateral pressures were calculated for at rest, active, and passive conditions with level backfill.

Table 6-1: Gros Ventre River Bridge – Lateral Earth Pressure Parameters – Coarse-Grained Materials

Condition	Coefficient of Earth Pressure	γK (equivalent fluid pressure)
Static Conditions Level Backfill	$K_o = 0.46$ $K_a = 0.29$ $K_p = 3.39$	$\gamma K_o = 61$ pcf $\gamma K_a = 40$ pcf $\gamma K_p = 458$ pcf
Earthquake Conditions Level Backfill Level Backfill	$K_{ae} = 0.36$ $K_{pe} = 3.18$	$\gamma K_{ae} = 49$ pcf $\gamma K_{pe} = 429$ pcf

*Assumes a unit weight for the backfill of 135 pcf and a friction angle of 33 degrees.

6.6 Soil Friction

Terzaghi, et al (1996) suggests use of a maximum of 30 degrees for the friction angle along a concrete base in granular soils. Accordingly, a friction value of 0.58, which is the tangent of 30 degrees, is suggested. The friction value may be combined with the passive pressure to resist horizontal loads.

7.0 GENERAL PATHWAY CONDITIONS

7.1 Description

The new pathway will run along the east side of North Highway 89 for approximately 6 miles from the northern end of Jackson to GTNP (Figure 2). Existing elevations along the pathway range from about 6213 to 6417 feet above mean sea level. The pathway may traverse a moderate slope with gradients up to about 12% along the highway north of the Fish Hatchery; otherwise, the pathway follows generally flat topography along the Flat Creek alluvial flood plain north of Jackson and along the glacial terrace and alluvial deposits within GTNP. The ground surface of the pathway is generally covered with grasses and sagebrush.

7.2 Geology

Figure 2 presents the geologic map of the area (Love et al., 1992), which shows the location of surficial deposits, bedrock units, and geologic structures (i.e., faults and folds). It appears that the southern 2.5 to 3 miles of the pathway mostly overlies Quaternary-aged swamp deposits (Qs) from the Flat Creek alluvial flood plain and colluvium (Qc) derived from adjacent East Gros Ventre Butte. The pathway then crosses what is mapped as loess (wind-blown silt) above Fish Hatchery Road before entering Quaternary-aged alluvial deposits (Qa) derived from the Snake River and glacial terrace deposits (Qtg) near the Gros Ventre River. The soil observed in the boreholes and test pits excavated for this investigation are consistent with the published geologic map, though less loess was encountered than is mapped by Love.

7.3 Soils

Three boreholes were placed along the pathway alignment in locations where structures are planned: BH-1 was drilled just south of the proposed underpass location at station 122+00, BH-2 was drilled at the toe of the fill prism below where the large retaining wall will be placed from about station 162+00 to 168+00, and BH-3 was drilled on the south side of the Gros Ventre River in proximity to the new pathway bridge. Five test pits were excavated along the pathway prism.

Soil types are presented below in relation to approximate stationing along the pathway prism. Borehole and test pit locations are shown on Figure 1 and detailed logs are attached in Appendix A.

7.3.1 Stations 0+00 to 146+00

The soils encountered from stations 0+00 to 146+00 generally consist of fine-grained silty material overlying coarse-grained alluvial deposits, with lenses of clay and sand present within the silt. Fine-grained material was encountered in our investigation down to depths of 3 to 9 feet bgs, and monitoring wells installed by WDEQ in 1997 and 2002 encountered fine-grained material down to about 14 to 25 feet bgs.

Test pit TP-1, located at station 37+50, was excavated to a depth of 4.5 feet bgs and encountered poorly developed topsoil overlying silty sandy gravels, cobbles, and boulders described as brown, slightly moist, dense, and massive. TP-1 was most likely situated at the edge of a debris fan originating along the eastern margin of East Gros Ventre Butte.

Seven of the WDEQ monitoring wells installed in 1997 and 2002 are in close proximity to the proposed pathway prism between stations 61+00 to 69+00: MW-118 through MW-122, which were drilled to depths of 18 to 23 feet bgs, MW-512 (drilled to about 25 feet bgs), and MW-610 (drilled to 10 feet bgs). These monitoring wells encountered approximately 3 feet of sand and gravel fill overlying silt and/or clay. MW-119 through MW-121 encountered sand and/or gravel below the silt at about 14 to 20 feet bgs; all other WDEQ monitoring wells did not penetrate past

fine-grained material. WDEQ monitoring well locations are shown on Figure 1B and logs are attached in Appendix B.

Borehole BH-1 and test pit TP-2 were located in proximity to station 122+00 (BH-1 was located at station 120+20, and TP-2 was located across from station 122+00), the proposed location of the underpass at the NMWA. BH-1 was located on the east side of Highway 89, while TP-2 was located to the west of the highway. BH-1 reached a depth of about 27 feet bgs, and encountered about 5 feet of gravel fill overlying about 4 feet of clayey silt, which in turn overlies about 3.5 feet of silty sand. The clayey silt is described as dark brown, moist, very stiff, and slightly plastic. The silty sand is described as brown-gray, wet, medium dense, and massive. Underlying the finer-grained material, at a depth of about 12.5 feet bgs, is sandy gravel described as gray, wet, very dense, and massive. The sandy gravel consists of approximately 10% fines, 30% sand, and 60% gravel. Corrosion resistance testing was performed on a disturbed sample taken 10 to 11.5 feet bgs (Appendix C). Testing shows that the sample has a pH of 7.3, a resistivity of 24 ohm-meters, sulfate concentration of 24 ppm (parts per million), and chloride concentration of 13 ppm.

TP-2 reached a depth of about 10 feet bgs, and encountered topsoil overlying about 1.5 feet of clayey silt (or loess), which in turn overlies colluvial sandy gravels, cobbles, and boulders. The loess is described as tan, slightly moist, very stiff, and massive. The underlying coarse-grained colluvial material is described as brown, slightly moist to moist, dense, and massive, and consists of approximately 15% fines, 15% sand, and 70% angular andesite and round quartzite clasts up to 3 feet in size. It is our understanding that tunnel footings will be placed at about 10 to 12 feet below current highway grade (grade at which depths in BH-1 was measured from). Based on our information from our field investigation, footings on the western side of Highway 89 will be placed on coarse-grained colluvium, and footings on the eastern side of Highway 89 on silty sand. However, coarse alluvium was encountered in BH-2 at a depth of 12.5 feet bgs, and it appears that a very slight over-excavation would be required to reach suitable bearing material.

7.3.2 Stations 146+00 to 202+00

BH-2 was located just east (downhill) of 167+00 at the toe edge of the highway fill prism. BH-2 was drilled to a depth of 21.5 feet bgs, and encountered 5 feet of soft clayey silt topsoil/alluvium overlying more than 16 feet of alluvial sand and gravel deposits. The sand and gravel deposits are described as gray, wet, medium dense to dense, and massive. Field estimates predict the alluvial deposits consist of approximately 10 to 30% fines, 30 to 55% sand, and 15 to 55% gravel. Fill was not encountered in the borehole.

TP-3 was located directly north of the intersection with North Highway 89 and Fish Hatchery Road in a topographic low between the two fill prisms. TP-3 was excavated down to about 12 feet bgs, and encountered 4 feet of sandy clay and gravel fill overlying soft to medium stiff clayey silt. The fill is described as tan, moist, stiff, and massive, and consists of approximately 20% sand, 40% clay, and 40% rounded quartzite gravel and cobbles up to 6 inches in size. The

clay-silt is described as black, gray, and tan, moist to very moist, soft to medium stiff, and massive. TP-3 did not penetrate the clayey silt.

Laboratory testing performed on an undisturbed clay-silt sample taken from TP-3 at 5.2 to 6.2 feet bgs shows a natural moisture content of 32.5%, a grain size distribution of 97% fines and 3% sand, a liquid limit (LL) of 37, a plastic limit (PL) of 24.7, and a plasticity index (PI) of 12.3 classifying the sample as a clay-silt (CL-ML). Consolidation testing was performed on sample U1 of test pit TP-3 from a depth of 5.2 to 6.2 feet bgs (incidentally about 1.2 to 2.2 feet below estimated grade prior to construction of the highway fill); results are attached in Appendix C. Testing indicates a pre-consolidation (preload) pressure of about 3,300 psf for the sample. In-situ vertical effective stress for the sample is about 700 psf so data indicate that the sample is over-consolidated; however, we are not aware of any geologic process that would have preloaded the soils. Other events that could create a preload pressure are seasonal wetting and drying (desiccation) or construction loading related to the highway fill. Given the uncertainty in origin of the preload pressure and that desiccation probably terminates about 5 feet below the ground surface, our settlement calculations assume that the upper 5 feet of the silt is indeed preloaded and the underlying silt is not.

A hand auger hole completed in 2007 was located approximately 150 feet northwest of the intersection of Highway 89 and Fish Hatchery Road, on the west side of North Highway 89. The hand auger hole was advanced to 17.5 feet bgs and encountered tan, slightly moist to moist, stiff clayey silt loess in the upper 7 feet of the hole. The remainder of the hole encountered blue-gray, moist to wet, soft, plastic clayey silt.

7.3.3 Station 202+00 to Gros Ventre River Bridge

TP-4 was located at station 202+50, at the northeastern end of the highway ramp that leads into GTNP. TP-4 reached a depth of 5 feet bgs and encountered approximately 1.5 feet of coarse fill and 1 foot of buried topsoil overlying coarse sandy gravels and cobbles. The gravels and cobbles are described as dark brown, slightly moist to moist, loose to medium dense, and slightly stratified. Field estimates predict this material consists of approximately 5 to 10% fines, 15 to 20% sand, and 70 to 80% rounded quartzite gravel and cobbles up to 8 inches in size.

TP-5 was located at CL 442, approximately 0.3 miles south of the existing Gros Ventre River bridge. TP-5 reached a depth of 8.5 bgs and encountered 3 feet of clayey silt topsoil described as dark brown to black, slightly moist, soft, and massive. Underlying the topsoil is sandy gravel and cobbles described as light brown, moist to very moist, loose, and stratified. Field estimates predict the coarse gravel and cobble material consists of approximately 5-10% fines, 20-25% sand, and 70% rounded quartzite clasts up to 9 inches in size.

BH-3 was located at the southeastern corner of the existing Gros Ventre River bridge, and reached a depth of 16.5 feet bgs. BH-3 encountered 4 feet of gravel and cobble fill overlying silty sandy gravel and cobble alluvial deposits. The alluvial deposits are described as gray, wet,

and very dense, and consist of approximately 5 to 10% fines, 15 to 35% sand, and 55 to 75% rounded gravel and cobbles. Corrosion resistance testing was performed on two disturbed samples taken 10 to 16.5 feet bgs (the samples were combined for corrosion testing).. Testing shows that the samples have a pH of 7.0, a resistivity of 32 ohm-meters, sulfate concentration of 7 ppm (parts per million), and chloride concentration of 7.3 ppm. Laboratory test results are attached in Appendix C.

7.4 Groundwater

Groundwater was not encountered in the test pits, which reached a maximum depth of about 12 feet bgs. However, groundwater was encountered in all three boreholes, and depth to groundwater measured at time of drilling (ATD) ranged from 6.0 to 15.0 feet bgs. A monitoring well was installed in BH-1, and indicated water levels of 10.8 and 9.8 feet bgs on May 21, 2009 and June 10, 2009, respectively. Groundwater typically fluctuates in response to seasonal precipitation and snowmelt. This investigation was completed during the "wet" time of year, when groundwater is rising due to run-off.

7.5 Earthquakes and Ground Shaking

Jackson Hole is located within the Intermountain Seismic Belt, a zone of seismicity that extends from southern Utah through eastern Idaho and western Montana and encompasses western Wyoming and the Teton Range (Smith and Arabasz, 1991). The Teton fault along the eastern margin of the Teton Range, located about 6 miles to the west of the proposed pathway layout, is considered an important structural element of the Intermountain Seismic Belt. Predicted recurrence intervals for maximum credible earthquakes have passed for most of the fault systems capable of generating magnitude 7.5 events in western Wyoming (Case, 1997), implying that the risk of major earthquakes is relatively high.

There are three locations where large structures are planned to be constructed along the pathway: 1) the tunnel/underpass that connects to the NMWA at station 122+00, 2) the retaining wall along the highway fill prism from stations 162+00 to 168+00, and 3) the new pathway bridge across the Gros Ventre River. Site ground motion accelerations for the design response spectra were compared for each of the three locations. Peak ground acceleration (PGA) is estimated to be 0.23g. Sites underlain predominantly by coarse-grained soils classify as Site Class D, and sites immediately underlain by fine-grained soils may classify as Site Class E. Soft, organic soils along the Flat Creek alluvial plain, specifically near the Flat Creek bridge and in southern portions of the pathway, may be peaty and could classify as Site Class F. No significant structures are planned along the peaty soils (to our knowledge construction will be limited to the small Starting Point retaining wall and the pathway prism) so a discussion has not been included below, but it should be understood that there is a risk of settlement along the pathway if a *design* level earthquake were to happen.

Site ground motion accelerations and design response spectra were derived in accordance with the general procedure defined in the International Building Code (IBC). The approach adopted in

the 2006 IBC is intended to provide a uniform margin of safety against collapse at the *design* ground motion. The *design* earthquake ground motion is selected at a ground shaking level that is 2/3 of the *maximum considered earthquake* (MCE) ground motion. The MCE ground motion is defined with a uniform likelihood of exceedance of 10 percent in 50 years. The Site Ground Motion and Design Response Spectrum for the 2006 IBC is presented in Figure 3 for Site Class D and in Figure 4 for Site Class E.

Earthquake Loads – Site Ground Motion and Design Response Spectrum 2006 International Building Code (IBC)

Approximate Site Location: **Latitude = 43.54º Longitude = -110.74º**

Mapped Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameters:

Short Period (S_s) = **1.19**

1-Sec Period (S_1) = **0.46**

Site Class Definition:

D – Stiff Soil Profile

Site Coefficients and Adjusted MCE Spectral Response Acceleration Parameters:

S_{MS} = **1.22**

S_{M1} = **0.70**

F_a = **1.02**

F_V = **1.54**

Design Spectral Response Acceleration Parameters:

S_{DS} = **0.81**

S_{D1} = **0.47**

Site Class Definition:

E – Soft Soil Profile

Site Coefficients and Adjusted MCE Spectral Response Acceleration Parameters:

S_{MS} = **1.07**

S_{M1} = **0.10**

F_a = **0.90**

F_V = **2.40**

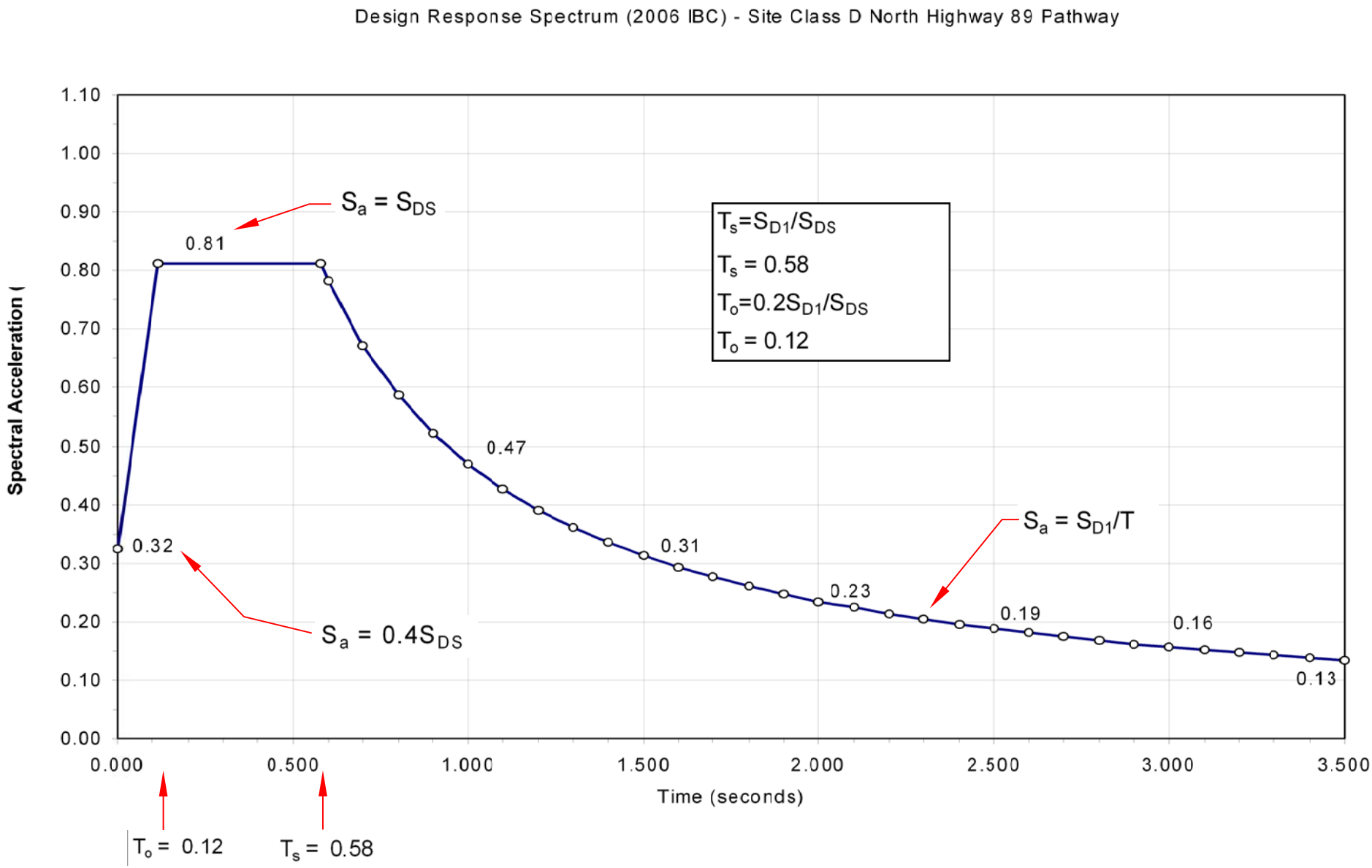
Design Spectral Response Acceleration Parameters:

S_{DS} = **0.72**

S_{D1} = **0.73**

7.6 Geologic Hazards

In the event of a large magnitude earthquake, strong ground shaking and ground cracking may occur. Loose, saturated sands and silty sand, and in some cases silt and gravel, may liquefy when exposed to seismic shaking. However, the soils along the pathway for the most part appear too coarse and dense to be susceptible to liquefaction. Sands with blow counts less than 15 bpf (blows per foot) could be vulnerable to liquefaction settlement, but not lateral spreading. BH-1 encountered a relatively thin sand layer at 9.0 feet bgs with blow counts of 15 bpf (Appendix A).



Geotechnical Site Investigation
North Highway 89 Pathway
Teton County, Wyoming

DESIGN RESPONSE SPECTRUM

FIGURE 3

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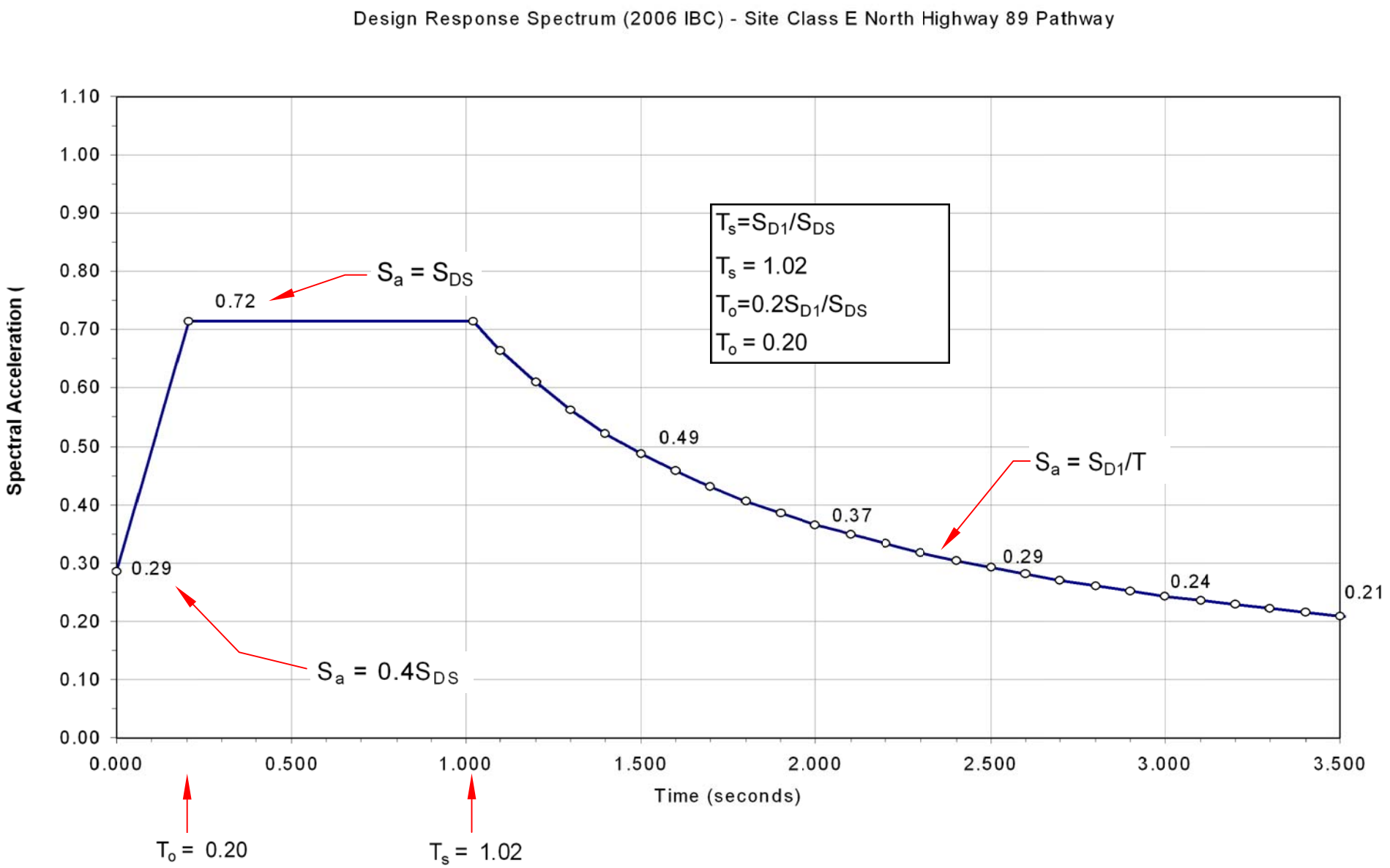

Geotechnical Site Investigation
North Highway 89 Pathway
Teton County, Wyoming

DESIGN RESPONSE SPECTRUM

FIGURE
4

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8.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

8.1 Pathway Prism Design Parameters

The lower, or southern, portion of the pathway appears to be underlain by fine-grained silt and clay-silt alluvial deposits, interrupted occasionally by coarse debris fan deposits derived from adjacent East Gros Ventre Butte. The middle portion of the pathway along the highway ramp near the Fish Hatchery appears to be underlain by clay and gravel fill, which was placed over clay-silt alluvium from the Flat Creek plain. The upper, or northern, portion of the pathway appears to be underlain by coarse-grained glacial terrace and alluvial deposits derived from the Snake River.

Flexible pavement designs have been prepared using methods suggested in Chapter 4 of the AASHTO Guide for Pavement Structures (1993) assuming low traffic level. Although the pathway is not designed for vehicular traffic, it falls into the low traffic level category both during construction (high loads for a short period of time) and civilian use. The subgrade soil below the pathway varies spatially and with depth. For fine-grained soils (clayey silt) underlying the pathway we have taken a conservative approach and considered the underlying soil to be "poor." For coarse-grained soils (sandy gravel and cobbles) underlying the pathway, we consider the soils to be "good." The pavement design recommendations assume that all topsoil and organic material will be stripped prior to road construction. The pavement sections were developed based on the following design assumptions:

US Climate Region - VI
Reliability – 75 %
Traffic Level – Low

Table 8-1: Flexible Pavement Design Parameters

Climate Region	Subgrade	Subgrade Quality	Traffic Level	SN
VI	Clay and Silt	Poor	Low	2.5
VI	Sandy Gravel and Cobbles	Good	Low	2.2

The design section is based on the following relationship, where the layer coefficient a_1 is equal to 0.33, a_2 is equal to 0.13, and a_3 is equal to 0.10. The layer thicknesses (in inches) of asphalt concrete (AC), base course (3/4-inch minus crushed gravel), and subbase (pit-run gravel) are represented by D_1, D_2, and D_3, respectively.

$$SN = a_1D_1 + a_2D_2 + a_3D_3$$

Table 8-2: Pavement Design Section

Subgrade Quality	Asphalt Pavement (D_1) inches	Base Course (D_2) inches	Subbase (D_3) inches
Poor	2	4	13
Good	2	4	10

Subbase may consist of approved pit-run gravel available from onsite borrow areas (foundation excavations) or other local sources. If the base course is placed directly on alluvial or glacial terrace gravels and cobbles, the subbase may be eliminated. The base course should consist of well-graded 1 1/2-inch minus road-mix gravel.

According to Jorgensen Associates, Teton County Pathways Program has specified the following pavement section: 2 inches of AC, 3 inches of crushed gravel, and 8 inches of subbase. Based on the above equation, the structural number (SN) for this pathway configuration is calculated at 1.8. The recommended SN for "poor" soil with low traffic levels is 2.5 while the recommended SN for good soil is 2.2. We believe that the configuration specified by the Teton County Pathways Program will be adequate for sections of the pathway underlain by "good" subgrade, i.e. coarse gravel and cobbles. However, the performance of the pavements sections in both cases (in particular the "poor" case) could be improved if constructed according to the parameters outlined in Table 8-2.

8.2 Settlement

Topsoil and loess are prone to consolidation and should be removed, if possible, below the pathway prism. In general, settlement is not expected along the pathway prism, except where additional embankment fill will be placed north of Fish Hatchery Road.

A 2H:1V embankment fill is planned on the north side of the fish hatchery access road, which will effectively widen the existing highway fill (which slopes at approximately 1.6H:1V) in order to accommodate the pathway section. The new fill will also load the silt material underlying the highway fill much like the retaining wall to the south; however, the pathway fill will load the silt material to the east of the existing highway fill. The proposed fill thickness could be as thick as about 14 feet.

For the above scenario settlement on the order of up to 5 inches is estimated beyond the toe of the highway fill, and about 1 to 2 inches is possible in the silt directly below the pathway. The lateral variability of settlement occurs because the new embankment fill will create a laterally variable surcharge; and as discussed in Section 5.4, the existing highway fill bridges the underlying silt. In our opinion over-excavation beneath the embankment fill is not necessary, as we do not expect major consequences of settlement, other than possible over-steepening of the proposed fill face beyond 2H:1V.

8.3 Cut and Fill Slopes

8.3.1 Permanent Slope Stability

Based on an estimated angle of internal friction of 33 to 35 degrees within coarse-grained soils, non-reinforced cut slopes should be no steeper than 1.5H:1V. Engineered fill slopes should be no steeper than 1.5H:1V. Raveling and shallow slump failures may occur if unreinforced cut and fill slopes are steeper than a 1.5H:1V grade.

Cut and fill slope angles in fine-grained clay or silt soils should be gentler than the slope angles in the coarse colluvial soils. Based on an estimated angle of internal friction of about 28 to 30 degrees for fine-grained soil, non-reinforced slopes should be no steeper than 2H:1V. Slope reinforcement will probably be required for steeper cut and fill slopes. In addition, the clayey silt loess is typically subject to erosion and slope protection is recommended.

8.3.2 Excavation and Cut Slope Stability

For the purpose of cuts and fills, the pathway soils may be divided into fine-grained (cohesive) soils and coarse-grained (non-cohesive) soils. OSHA regulations (29CFR1926) appear to classify the fine-grained soils as Type A (unless saturated, which will place them into Type C) and the coarse-grained soils as Type C. Simple cut and fill slopes in Type A soils should be no steeper than 0.75H:1V, and no steeper than 1.5H:1V in Type C soil. The contractor shall be responsible for adherence to OSHA and other safety regulations.

8.4 Retaining Walls

Three retaining walls will be required along the pathway alignment. Appendix D includes documentation comparing relative cost, design, and construction considerations that may be useful for the wall selection process. Although the material is dated, relative ranking in terms of cost should be similar today. In general for larger walls that would require tie-backs or reinforcement, MSE block walls are the least expensive option, gabion faced walls and H-Pile and lagging walls are more expensive, and reinforced concrete is the most expensive.

Table 8-3 summarizes the soil strength parameters to use when considering retaining wall design in the soils. Performance of retaining walls is highly dependent on effective drainage.

Table 8-3: Retaining Wall Parameters

	φ	Cohesion (psf)	Unit Weight (pcf)
Native Silt or Clay	28	190	100
Native Sandy Gravel and Cobbles/Fill	33	0	135
Compacted Silt or Clay	28	190	120
Compacted Sandy Gravel and Cobbles/Fill	33	0	135

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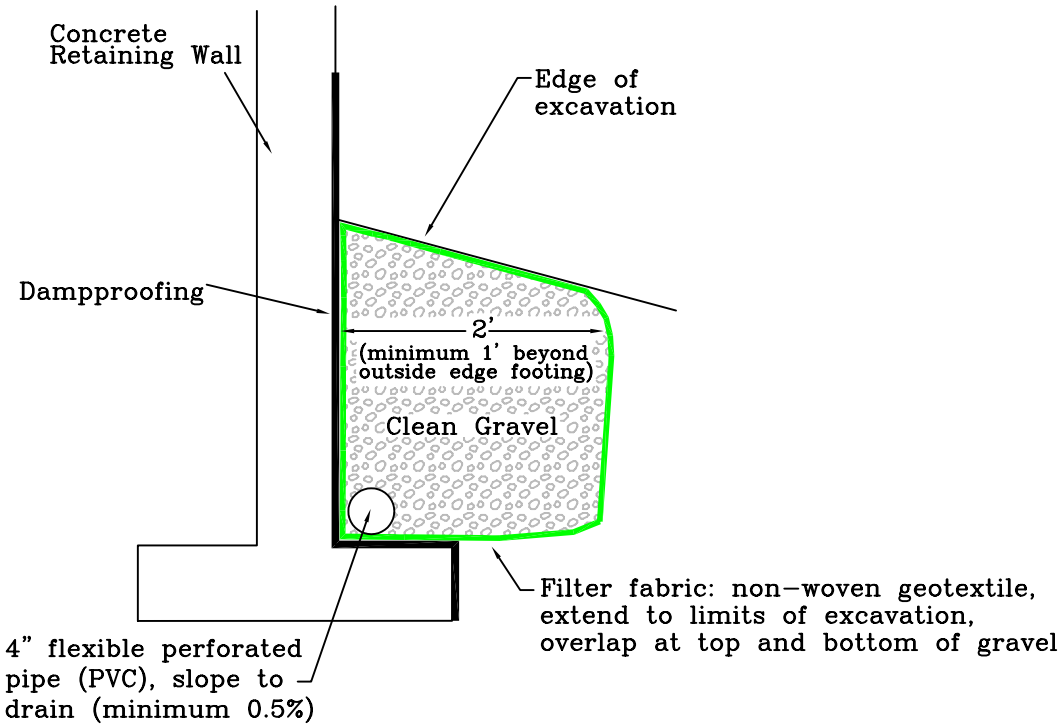

Schematic Drawing
Not to Scale

Geotechnical Site Investigation
North Highway 89 Pathway
Teton County, Wyoming

Concrete Retaining Wall
Foundation Drain

FIGURE
5

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8.4.1 Wall Reinforcement Selection

Some of the soils encountered along the pathway alignment contain significant amounts of material that are too large for use with most geogrids. Typical manufacturers' recommendations for backfill over uniaxial geogrids preclude the use of material over 2.5 inches in diameter. Selection of geogrid reinforcement should be made with coarse backfill or screening in mind.

8.4.2 Wall Foundations

In our opinion, the coarse alluvium and glacial terrace deposits, generally consisting of sandy gravel, cobbles, and boulders will provide adequate support for anticipated foundation loads of the proposed walls. Areas underlain by fine-grained deposits, as is most likely the case at the Starting Point Wall and the Wetland Wall, may require over-excavation and replacement with structural fill, depending on footing depths. Frost protection (depth) should be provided. Adequate drainage should be provided behind retaining walls. Specific drainage recommendations are provided in below in Section 8.5.

8.5 Foundation Drainage

Drains are strongly recommended for the retaining walls. WAI has reviewed the retaining wall schematic cross-sections that JA prepared for Teton County, and we believe the drainage plans to be acceptable. For the concrete walls, the following suggestions may improve drainage: the 2-inch washed rock should extend up the back wall to about the excavation line, and be wrapped in a non-woven geotextile filter fabric (Figure 5).

8.6 Corrosion

Two samples taken from BH-1 and BH-3 were tested for pH, sulfates, chlorides, conductivity, and resistivity. Evaluation of corrosion potential is determined from using the Culvert Service Life Guidelines from the Montana Department of Transportation (MDT) and FHWA GEC No 5 – Evaluation of Soil and Rock Properties. Laboratory results indicate resistivity ranging from 24 to 32 ohm-meters, pH of about 7, and 7 to 24 parts-per-million (less than 0.2%) sulfates. These results indicate non-corrosive soils. Laboratory test results are discussed in detail in Sections 4.3.1 and 4.3.3, and are attached in Appendix C.

According to the MDT guidelines, if the soil pH is between 6 and 8.5, sulfate concentration is less than 0.2%, and the resistivity is greater than 22 ohm-meters, corrosion protection for steel and concrete is not required. Furthermore, according to FHWA guidelines, the soils are not considered aggressive.

8.7 Observation during Construction and Plan Review

A representative of this office should observe construction of any foundation or drainage elements recommended in this report. If any unexpected soils or conditions are revealed during

construction, this office should be notified immediately to survey the conditions and make necessary modifications. As the plans for retaining walls and other design work are completed, this office should be involved in review.

9.0 LIMITATIONS

This report has been prepared based on a limited amount of data. Actual site conditions may vary. The report is for single use and under no circumstances are the figures and text to be used separately. These services have been performed in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing in this area under similar conditions. No other warranty is made or implied.

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APPENDIX A

TEST PIT AND BOREHOLE LOGS

WOMACK & ASSOCIATES, INC.
Geotechnical Engineering
Geology

P.O. Box 12650
Jackson, WY 83002
Telephone: (307)733-7209
Fax:

TEST HOLE LOG

PROJECT NAME: Jorgensen Associates - North Highway 89 Pathways	DATE: 5/13/09
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: BH-1

TEST HOLE LOCATION: ~30' west and 5' north of refuge gate, 3' north of asphalt on the north side of refuge sleigh ride access driveway, ~15' east of JA CL 117 stake, GPS coordinates: 43.52°N, -110.75°W

ELEVATION G.S. (ft.): ~6228	TOTAL DEPTH (ft.): 26.5	GROUNDWATER LEVEL (ft.): 9.8	MEASURED FROM: Surface

DRILL TYPE: CME-75	HAMMER: 140 # Automatic	DRILL CO: HazTech Drilling, Inc.	DRILLER: Dave	LOGGED BY: jrg

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION COMMENTS: Drilled with 8" OD augers and sampled with 2" ODX1.5"ID split spoon without liners.	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
1 2 3 4								0.0-0.2ft Clayey SILT: Moist, dark brown, slightly plastic [TOPSOIL] 0.2-0.7ft Gravel up to 1" in diameter [FILL/BASE COURSE] 0.7-5.0ft GRAVEL: Slightly moist, brown, subangular to subround gravel up to 2" diameter, larger clasts in spoils, moderate grinding [FILL]					
5 6 7 8		D1	4,7,7	23	33			5.0-9.0ft Clayey SILT: Moist, dark brown, very stiff, slightly plastic, some organics (straw, roots), sample is greatly disturbed, easy drilling [ALLUVIUM]					
9 10 11 12		D2	4,7,6	15	78			9.0-12.5ft Silty SAND: Wet, brown-gray, medium dense, fine, some orange oxidation, massive, easy drilling [ALLUVIUM] 9.8ft Groundwater measured on 6/9/09 10.8ft Groundwater measured on 5/21/09					
13 14 15 16		D3	15,28,50/5.5'	100+	67			12.5-25.0ft Sandy GRAVEL: Wet, gray, very dense, ~60% subangular to subround gravel, ~30% sand, up to 10% fines, massive, faint organic odor, easy drilling, slight grinding in gravels [ALLUVIUM] 15.0ft Groundwater ATD					
17 18 19 20 21 22 23 24		D4	7,28,50/5'	100+	100								

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TEST HOLE LOG

PROJECT NAME: Jorgensen Associates - North Highway 89 Pathways | DATE: 5/13/09

PROJECT LOCATION: Jackson, Wyoming | HOLE NO.: BH-1

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION COMMENTS: Drilled with 8" OD augers and sampled with 2" ODX1.5"ID split spoon without liners.	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
26		D5	9,28,37	75	100			25.0-26.5ft 9" of sluff Remainder: Gravelly SAND: Wet, gray, very dense, massive, 70% fine to coarse angular sand, 30% subround gravel up to 1/2" in diameter					
27								Monitoring well installation: 0-13' 2" PVC pipe. 13-23' slotted 2" PVC pipe. 0-11' bentonite. 11-15' sand. 15-26.5' heave. Flush mount lid at surface. Note: Hole heaved to ~15' prior to installation of monitoring well					
28													
29													
30													
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TEST HOLE LOG

PROJECT NAME: Jorgensen Associates - North Highway 89 Pathways	DATE: 5/13/09
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: BH-2
TEST HOLE LOCATION: Station 167 + 00, ~5' west of fence, GPS coordinates: 43.53°N, -110.74°W	

ELEVATION G.S. (ft.): ~6240	TOTAL DEPTH (ft.): 21.5	GROUNDWATER LEVEL (ft.): 6	MEASURED FROM: Surface

DRILL TYPE: CME-75	HAMMER: 140 # Automatic	DRILL CO: HazTech Drilling, Inc.	DRILLER: Dave	LOGGED BY: jrg

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
1		D0						0.0-0.1ft SOD 0.1-5.0ft Clayey SILT: Moist, dark brown, soft, slightly plastic, abundant organics, grades into below, very easy drilling [TOPSOIL/ALLUVIUM]					
2													
3													
4													
5													
6		D1	4,7,10	28	67	1.5		5.0-5.5ft Clayey/silty SAND: Wet, gray, very stiff to medium dense, ~40% fines, ~60% very fine sand, some organics, massive, grades from above [ALLUVIUM]					
7								5.5-21.5ft Clayey/silty SAND and GRAVEL: Wet, gray with black and white spots, medium dense, massive, ~20% fines, ~30% sand, ~50% subrounded to subangular gravel to 1/2" diameter, some mechanical breakage [ALLUVIUM/OUTWASH]					
8													
9													
10								6.0ft Groundwater ATD					
11		D2	6,12,10	28	44			10.0ft Silty SAND and GRAVEL: As above, gray, ~10% fines, ~35% sand, ~55% subrounded gravel, faint organic odor					
12													
13													
14													
15								15.0ft Upper 7": As above					
16		D3	6,8,30	53	39			Remainder: Silty gravelly SAND: Less than 10% fines, 40% gravel, 50% sand, massive, loose to medium dense					
17													
18													
19													
20								20.0ft GRAVEL and silty fine SAND: Wet, gray, very dense, ~15% gravel, ~30% fines, ~55% fine sand, spoon must have driven cobble, no mechanical breakage or clasts in shoe [ALLUVIUM/OUTWASH]					
21		D4	27,50/3"	100+	44								
22								Note: Drilling was fairly easy in sand and gravel. Hole backfilled with cuttings.					
23													
24													

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TEST HOLE LOG

PROJECT NAME: Jorgensen Associates - North Highway 89 Pathways	DATE: 5/13/09
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: BH-3
TEST HOLE LOCATION: ~20' south and 40' east of southeast end highway bridge abutment, GPS coordinates: 43.57N°, -110.73°W	

ELEVATION G.S. (ft.): ~6384	TOTAL DEPTH (ft.): 16.5	GROUNDWATER LEVEL (ft.): 8.3	MEASURED FROM: Surface

DRILL TYPE: CME-75	HAMMER: 140 # Automatic	DRILL CO: HazTech Drilling, Inc.	DRILLER: Dave	LOGGED BY: jrg

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION COMMENTS: Drilled with 8" OD augers and sampled with 2" ODX1.5"ID split spoon without liners.	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
1 2 3 4								0.0-4.0ft FILL: Gravel and cobble, heavy grinding to 5'					
5 6		D1	19,50/5"	100+	64			4.0-16.5ft Silty sandy GRAVEL and COBBLE: very heavy grinding [ALLUVIUM] 5.0ft Sandy GRAVEL and COBBLES: Slightly moist, gray, trace fines, very dense, ~35% sand, ~65% round gravel and cobbles, much mechanical breakage					
7 8 9								7.5ft As above, slightly less grinding and easier drilling 8.3ft Groundwater ATD					
10 11		D2	20,42,32	84	61			10.0ft Silty sandy GRAVEL and COBBLE: Wet, gray brown, very dense, ~10% fines, ~15% sand, ~75% rounded gravel and fragments (much mechanical breakage)					
12 13 14													
15 16		D3	19,30,50/6"	100+	72			15.0ft Silty sandy GRAVEL and COBBLES: Wet, gray brown, very dense, <10% fines, ~35% sand, ~55% gravel and cobble fragments (much mechanical breakage)					
17 18 19 20 21 22 23 24								Note: Hole backfilled with cuttings.					

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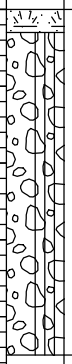
TEST HOLE LOG

PROJECT NAME: Jorgensen Associates - North Highway 89 Pathways	DATE: 5/21/09
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: TP-1

TEST HOLE LOCATION: Station 37+50, JA stake CL 38, Dirt pullout on east side of highway, ~15' west of refuge fence, GPS coordinates: 43.50°N, -110.76°W

ELEVATION G.S. (ft.): ~6221	TOTAL DEPTH (ft.): 4.5	GROUNDWATER LEVEL (ft.): N/A	MEASURED FROM: Surface
DRILL TYPE: Case 580 Super L	HAMMER:	DRILL CO: Fish Creek Excavation	DRILLER: Bill / LOGGED BY: ds

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION COMMENTS: Ground surface is grass	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
1 2 3 4							0.0-0.3ft Gravelly clayey SILT: Dry, light gray, loose, massive, ~10% angular limestone gravel up to 2" in diameter, ~25% clay, ~65% silt [TOPSOIL] 0.3-4.5ft Clayey sandy silty GRAVELS, COBBLES, BOULDERS: Slightly moist, brown, dense, massive, ~5% clay, ~10% sand, ~15% silt, ~70% angular quartzite and limestone clasts up to 2' in diameter, clast size increasing with depth, moisture content increasing with depth, sandy pockets, red clay pocket on south end of pit [DEBRIS FAN]					
5 6 7 8 9 10 11 12 13 14												

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TEST HOLE LOG

PROJECT NAME: Jorgensen Associates - North Highway 89 Pathways	DATE: 5/21/09
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: TP-2

TEST HOLE LOCATION: Across highway from station 122+00, ~75' north/northeast of elk statue at the Wildlife Museum, GPS coordinates: 43.52°N, -110.75°W

ELEVATION G.S. (ft.): ~6235	TOTAL DEPTH (ft.): 10.4	GROUNDWATER LEVEL (ft.): N/A	MEASURED FROM: Surface
DRILL TYPE: Case 580 Super L	HAMMER:	DRILL CO: Fish Creek Excavation	DRILLER: Bill / LOGGED BY: ds

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
1					1.5		0.0-1.3ft Gravelly clayey SILT: Very slightly moist, black, medium stiff, stratified, roots present to ~1' [TOPSOIL]					
2		D1			2.5-3		1.3-3.0ft Clayey SILT: Slightly moist, tan, very stiff, massive, no pinhole voids, clayey silt only present on east side of pit - see below for description on west side [LOESS]					
3							3.0-10.4ft Clayey silty sandy GRAVELS, COBBLES, BOULDERS: Slightly moist to moist, brown, dense, massive, ~5% clay, ~10% silt, ~15% sand, ~70% angular andesite and round quartzite clasts up to 3' in diameter, pockets of black organic clay at ~4.5', moisture content increasing with depth, very difficult digging [COLLUVIUM]					
4												
5												
6												
7												
8												
9												
10							10.0ft As above, trace (>5%) silt, ~10% clay, ~25% sand, ~60% clasts, slightly plastic clay pockets					
11												
12												
13												
14												

COMMENTS: Ground surface is grass, cobbles, and silt; ground may have been disturbed from previous statue

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TEST HOLE LOG

PROJECT NAME: Jorgensen Associates - North Highway 89 Pathways	DATE: 5/21/09
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: TP-3

TEST HOLE LOCATION: ~350' south of station 180+00 in topographic low north of intersection between highway and Fish Hatch Road, GPS coordinates: 43.53°N, -110.73°W

ELEVATION G.S. (ft.): ~6250	TOTAL DEPTH (ft.): 11.9	GROUNDWATER LEVEL (ft.): N/A	MEASURED FROM: Surface
DRILL TYPE: Case 580 Super L	HAMMER:	DRILL CO: Fish Creek Excavation	DRILLER: Bill / LOGGED BY: ds

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
							COMMENTS: Pit is located on low east side of highway fill prism; Ground surface is grass					
1, 2					2		0.0-4.0ft Sandy CLAY with GRAVEL and COBBLES: Moist, tan, stiff, massive, ~20% sand, ~40% clay, ~40% round quartzite gravel and cobbles up to 6" in diameter [FILL]					
3		D1										
4		D2			1-1.25		4.0-11.9ft CLAY-SILT: Moist, black, soft to medium stiff, massive, slightly plastic, grades into below [ALLUVIUM]					
5		U1				CL-ML		33	73.4	37	12	
6		D3					5.8ft As above, very moist, gray					
7					1.25-1.5							
8												
9							9.0ft As above, slightly moist to moist, tan					
10					1-2							
11		D4										
12												
13												
14												

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TEST HOLE LOG

PROJECT NAME: Jorgensen Associates - North Highway 89 Pathways	DATE: 5/21/09
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: TP-4

TEST HOLE LOCATION: Station 202+50, JA stake CL 315, ~15' north of Grand Teton National Park sign, GPS coordinates: 43.54°N, -110.74°W

ELEVATION G.S. (ft.): ~6356	TOTAL DEPTH (ft.): 5	GROUNDWATER LEVEL (ft.): N/A	MEASURED FROM: Surface
DRILL TYPE: Case 580 Super L	HAMMER:	DRILL CO: Fish Creek Excavation	DRILLER: Bill / LOGGED BY: ds

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION COMMENTS: Ground surface is grass and gravel fill	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
1							0.0-1.5ft Silty sandy GRAVEL and COBBLES: Dry, brown, loose to medium dense, massive, ~10% silt, ~25% sand, ~65% angular to subangular quartzite gravel and cobbles up to 4" in diameter, roots present to ~0.5' [FILL/BASE COURSE]					
2					1.25-1.5		1.5-2.5ft Clayey SILT: Moist, black, medium stiff, massive [BURIED TOPSOIL]					
3		D1					2.5-5.0ft Silty sandy GRAVEL and COBBLES: Slightly moist to moist, dark brown, loose to medium dense, slightly stratified, ~5-10% silt, ~15-20% sand, ~70-80% round quartzite gravel and cobbles up to 8" in diameter, some caving [GLACIAL TERRACE DEPOSITS]					
4												
5												
6												
7												
8												
9												
10												
11												
12												
13												
14												

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TEST HOLE LOG

PROJECT NAME: Jorgensen Associates - North Highway 89 Pathways	DATE: 5/21/09
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: TP-5

TEST HOLE LOCATION: JA stake CL 442, undeveloped road/pullout on east side of highway ~0.3 miles south of Gros Ventre River bridge, GPS coordinates: 43.56°N, -110.73°W

ELEVATION G.S. (ft.): ~6387	TOTAL DEPTH (ft.): 8.5	GROUNDWATER LEVEL (ft.): N/A	MEASURED FROM: Surface

DRILL TYPE: Case 580 Super L	HAMMER:	DRILL CO: Fish Creek Excavation	DRILLER: Bill	LOGGED BY: ds

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION COMMENTS: Ground surface is grass	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
1 2					1		0.0-3.1ft Clayey SILT: Slightly moist, dark brown to black, soft, massive, roots present to ~2' [TOPSOIL]					
3 4 5 6 7 8		D1					3.1-8.5ft Clayey sandy GRAVEL and COBBLES: Moist to very moist, light brown, loose, stratified, ~70% round quartzite gravel and cobbles up 9" in diameter, clast size increasing with depth, some caving [GLACIAL TERRACE DEPOSITS] 7.0ft As above, clast size increases to 12" in diameter					
9 10 11 12 13 14												

APPENDIX B

WYOMING DEQ MONITORING WELL INSTALLATION LOGS

BOREHOLE INSTALLATION DIAGRAM

Project Jackson Subsurface Investigation CFB-0290-D
Location Red Barns, Inc. Site No. 18
Contractor GeoScience & Engineering, Inc. 97105-1
Driller Dan Hawley - Hawley Brothers Drilling
Method 7.5 inch OD HSA - BK81
Sampler 1 3/8 inch ID Split Spoon

Borehole No. MW-118
Monitor Well No. MW-118
Date 9/8/97
Logged by BC, EB
Checked by CN
Sheet 1 of 2

Depth	Description/Lithology	Blows/ Depth	Field Screening Method PID (ppmv) Reading	Notes
0	Brown sandy FILL with gravel			
5	Gray SILT			
10	24 in gray SILT Groundwater @ 10 ft	4 3 4 3 9 - 11 ft	14.1	R = 24 in Soil sample MW-118-9-11
	Gray SILT			
15	24 in gray SILT with some clay	1 1 2 4 14 - 16 ft	10.8	R = 24 in R = Recovery

Depth to Water During Drilling 10 ft Date 9/8/97
Static Water Level Depth 10.58 ft Date 9/12/97
10.54 ft Date 10/14/97
Measured From North side PVC well casing
Remarks Along east side of highway.

BOREHOLE INSTALLATION DIAGRAM

Project Jackson Subsurface Investigation CFB-0290-D
Location Red Barns, Inc. Site No. 18
Contractor GeoScience & Engineering, Inc. 97105-1
Driller Dan Hawley - Hawley Brothers Drilling
Method 7 1/2 inch OD HSA - BK81
Sampler 1 3/8 inch ID Split Spoon

Borehole No. MW-118
Monitor Well No. MW-118
Date 9/8/97
Logged by BC, EB
Checked by CN
Sheet 2 of 2

Depth	Description/Lithology	Blows/ Depth	Field Screening Method PID (ppmv) Reading	Notes
15				
	Gray SILT			
	End of boring @ 18 ft			
20				
25				
30				

Depth to Water During Drilling 10 ft Date 9/8/97
Static Water Level Depth Date
Measured From
Remarks

BOREHOLE INSTALLATION DIAGRAM

Project: Jackson Subsurface Investigation CFB-0290-D
Location: Red Barns, Inc. Site No. 18
Contractor: GeoScience & Engineering, Inc. 97105-1
Driller: Dan Hawley - Hawley Brothers Drilling
Method: 7.5 inch OD HSA - BK81
Sampler: 1 3/8 inch ID Split Spoon

Borehole No.: MW-119
Monitor Well No.: MW-119
Date: 9/8/97
Logged by: EB
Checked by: CN
Sheet 1 of 2

Depth	Description/Lithology	Blows/ Depth	Field Screening Method PID (ppmv) Reading	Notes
0	Sand & gravel FILL			
5	Brown SILT			
10	3 in brown GRAVEL with silt 9 in brown SILT	6 5 8 10 9.5 - 11.5 ft	300	R = 12 in Slight petroleum odor Wet
	▼ Groundwater @ 12 ft 20 in black SILT 4 in gray SILT	2 3 3 5 11.5 - 13.5 ft	1,354	R = 24 in Strong petroleum odor Soil sample MW-119-11.5-13.5
15	Brownish black SILT			R = Recovery

Depth to Water During Drilling: 12 ft — Date: 9/8/97
Static Water Level Depth: 10.47 ft — Date: 9/12/97
10.64 ft — Date: 10/14/97
Measured From: North side PVC well casing
Remarks:

BOREHOLE INSTALLATION DIAGRAM

Project: Jackson Subsurface Investigation CFB-0290-D
Location: Red Barns, Inc. Site No. 18
Contractor: GeoScience & Engineering, Inc. 97105-1
Driller: Dan Hawley - Hawley Brothers Drilling
Method: 7 1/2 inch OD HSA - BK81
Sampler: 1 3/8 inch ID Split Spoon

Borehole No.: MW-119
Monitor Well No.: MW-119
Date: 9/8/97
Logged by: EB
Checked by: CN
Sheet 2 of 2

Depth	Description/Lithology	Blows/ Depth	Field Screening Method PID (ppmv) Reading	Notes
15	Brown SILT			
20	20 in SAND & GRAVEL	3 8 28 r 19 - 21 ft	46.2	R = 20 in
	SAND & GRAVEL			
25	End of boring @ 23 ft			
30				r = Refusal R = Recovery

Depth to Water During Drilling: 12 ft Date: 9/8/97
Static Water Level Depth: _____ Date: _____
Measured From: _____
Remarks: _____

BOREHOLE INSTALLATION DIAGRAM

Project: Jackson Subsurface Investigation CFB-0290-D
Location: Red Barns, Inc. Site No. 18
Contractor: GeoScience & Engineering, Inc. 97105-1
Driller: Dan Hawley - Hawley Brothers Drilling
Method: 7.5 inch OD HSA - BK81
Sampler: 1 3/8 inch ID Split Spoon

Borehole No.: MW-120
Monitor Well No.: MW-120
Date: 9/9/97
Logged by: BC, EB
Checked by: CN
Sheet 1 of 2

Depth	Description/Lithology	Blows/ Depth	Field Screening Method PID (ppmv) Reading	Notes
0	Sand & gravel FILL			
5	Brown SILT ROCKS ROCK Brown SILT			
10	8 in brown SILT	6 4 3 8 9.5 - 11.5 ft	16.1	R = 8 in
	▼ Groundwater @ 12 ft			
	17 in gray sandy SILT with clay	2 2 3 3 12.5 - 14.5 ft	9.3	R = 17 in
15	5 in gray sandy SILT with clay 17 in gray SAND & GRAVEL	5 8 10 15 14.5 - 16.5 ft	20.1	R = 22 in Soil sample MW-120-14.5-16.5 R = Recovery

Depth to Water During Drilling: 12 ft — Date 9/9/97
Static Water Level Depth: 10.45 ft — Date 9/12/97
10.59 ft — Date 10/14/97

Measured From: North side PVC well casing
Remarks:

BOREHOLE INSTALLATION DIAGRAM

Project Jackson Subsurface Investigation CFB-0290-D
Location Red Barns, Inc. Site No. 18
Contractor GeoScience & Engineering, Inc. 97105-1
Driller Dan Hawley - Hawley Brothers Drilling
Method 7 1/2 inch OD HSA - BK81
Sampler 1 3/8 inch ID Split Spoon

Borehole No. MW-120
Monitor Well No. MW-120
Date 9/9/97
Logged by EB
Checked by CN
Sheet 2 of 2

Depth	Description/Lithology	Blows/ Depth	Field Screening Method PID (ppmv) Reading	Notes
15				
	SAND & GRAVEL			
20	22 in brown SAND & GRAVEL	2 10 30 r 19 - 21 ft	15.4	R = 22 in
	SAND & GRAVEL			
25	End of boring @ 23 ft			
30				r = Refusal R = Recovery

Depth to Water During Drilling 12 ft Date 9/9/97
Static Water Level Depth _____ Date _____
Measured From _____
Remarks _____

BOREHOLE INSTALLATION DIAGRAM

Project Jackson Subsurface Investigation CFB-0290-D
Location Red Barns, Inc. Site No. 18
Contractor GeoScience & Engineering, Inc. 97105-1
Driller Dan Hawley - Hawley Brothers Drilling
Method 7.5 inch OD HSA - BK81
Sampler 1 3/8 inch ID Split Spoon

Borehole No. MW-121
Monitor Well No. MW-121
Date 9/9/97
Logged by EB
Checked by CN
Sheet 1 of 2

Depth	Description/Lithology	Blows/ Depth	Field Screening Method PID (ppmv) Reading	Notes
0	Sand & gravel FILL			
5	ROCKS & black SILT with gravel			
10	1 in brown SILT with gravel	11 9 9 7 9 - 11 ft	3.6	R = 1 in Wet
	22 in brown SILT Groundwater @ 12 ft	3 4 6 6 11 - 13 ft	2.0	R = 22 in
15	18 in gray fine SAND with silt	4 6 9 17 14 - 16 ft	3.5	R = 18 in Soil sample MW-121-14-16 r = Refusal R = Recovery

Depth to Water During Drilling 12 ft Date 9/9/97
Static Water Level Depth 10.33 ft Date 9/12/97
10.43 ft Date 10/14/97

Measured From North side PVC well casing
Remarks

BOREHOLE INSTALLATION DIAGRAM

Project Jackson Subsurface Investigation CFB-0290-D
Location Red Barns, Inc. Site No. 18
Contractor GeoScience & Engineering, Inc. 97105-1
Driller Dan Hawley - Hawley Brothers Drilling
Method 7 1/2 inch OD HSA - BK81
Sampler 1 3/8 inch ID Split Spoon

Borehole No. MW-121
Monitor Well No. MW-121
Date 9/9/97
Logged by BC, EB
Checked by CN
Sheet 2 of 2

Depth	Description/Lithology	Blows/ Depth	Field Screening Method PID (ppmv) Reading	Notes
15				
	Gray SAND with gravel			
20	18 in gray coarse SAND with gravel	9 13 31 33 19 - 21 ft	25.5	R = 18 in
	Gray SAND with gravel			
25	End of boring @ 23 ft			
30				R = Recovery

Depth to Water During Drilling 12 ft Date 9/9/97
Static Water Level Depth Date
Measured From
Remarks

BOREHOLE INSTALLATION DIAGRAM

Project: Jackson Subsurface Investigation CFB-0290-D
Location: Red Barns, Inc. Site No. 18
Contractor: GeoScience & Engineering, Inc. 97105-1
Driller: Dan Hawley - Hawley Brothers Drilling
Method: 7.5 inch OD HSA - BK81
Sampler: 1 3/8 inch ID Split Spoon

Borehole No.: MW-122
Monitor Well No.: MW-122
Date: 9/9/97
Logged by: EB
Checked by: CN
Sheet 1 of 2

Depth	Description/Lithology	Blows/ Depth	Field Screening Method PID (ppmv) Reading	Notes
0	Sand & gravel FILL			
5	Dark brown SILT with gravel			
10	16 in dark brown SILT Groundwater @ 10 ft	4 12 5 4 9 - 11 ft	7.9	R = 16 in Soil sample MW-122-9-11
	20 in tan sandy SILT With shells	1 1 2 7 11 - 13 ft	1.5	R = 20 in
15	17 in olive SILT	5 5 38 r 14 - 16 ft	1.2	R = 17 in r = Refusal R = Recovery

Depth to Water During Drilling: 10 ft — Date: 9/9/97
Static Water Level Depth: 10.46 ft — Date: 9/12/97
10.47 ft — Date: 10/14/97
Measured From: North side PVC well casing
Remarks:

BOREHOLE INSTALLATION DIAGRAM

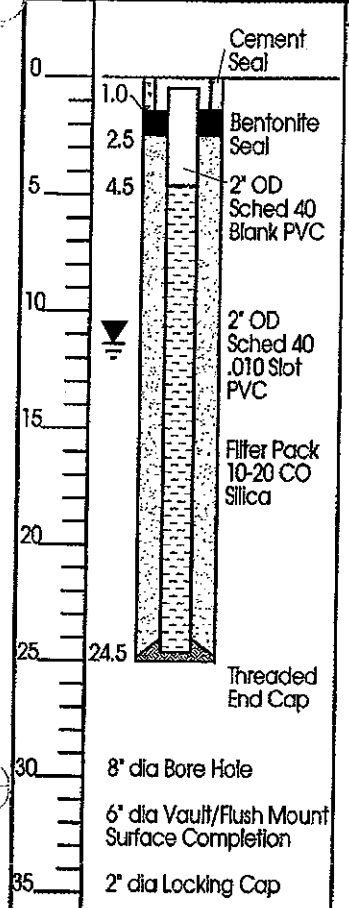
Project Jackson Subsurface Investigation CFB-0290-D
Location Red Barns, Inc. Site No. 18
Contractor GeoScience & Engineering, Inc. 97105-1
Driller Dan Hawley - Hawley Brothers Drilling
Method 7 1/2 inch OD HSA - BK81
Sampler 1 3/8 inch ID Split Spoon

Borehole No. MW-122
Monitor Well No. MW-122
Date 9/9/97
Logged by EB
Checked by CN
Sheet 2 of 2

Depth	Description/Lithology	Blows/ Depth	Field Screening Method PID (ppmv) Reading	Notes
15				
	COBBLES & brown SILT			
20	18 in olive sandy SILT with some gravel	9 12 8 11 19 - 21 ft	1.9	R = 18 in
25	End of boring @ 23 ft			
30				R = Recovery

Depth to Water During Drilling 10 ft Date 9/9/97
Static Water Level Depth Date
Measured From
Remarks

DEPTH (feet)	WELL CONSTRUCTION	PID (PPM)	BLOWS/FOOT	SAMPLE NUMBER	U.S.C.S. DESIGNATION	SOIL DESCRIPTION
0	Cement Seal; 1.0; 2.5 Bentonite Seal; 4.5 2" OD Sched 40 Blank PVC; 2" OD Sched 40 .010 Slot PVC; Filter Pack 10-20 CO Silica; 24.5 Threaded End Cap; 8" dia Bore Hole; 6" dia Vault/Flush Mount Surface Completion; 2" dia Locking Cap					Gravel Road Base
					GM	Gravelly Silt; brown to olive brown; with trace cobbles; and some sand.
					CL	Clay; dark brown; moist; with some fine to coarse gravels and some sand.
					CL	Clay; light gray; moist; with some sand.
						Boring terminated at 25 feet bgs.



SURFACE ELEVATION:
TOTAL DEPTH: 25 feet
DIAMETER OF BORING: 8 inches
WATER ENCOUNTERED AT: 11.32 feet

LOGGED BY: Ann Patton
DRILLER: Inberg-Miller
TYPE OF RIG: Mobile BK57
DATE DRILLED: 24 September 2002

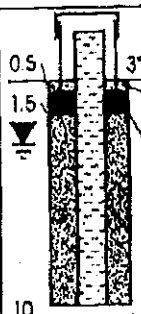
DRILLING METHOD: Hollow-Stem Auger
TYPE OF WELL: 2" Groundwater Monitoring
REG. NO.: U.W. 147431

ASSET ENVIRONMENTAL SERVICES II, LLC

BORING LOG OF MWA-12

Red Barns 512

PROJECT NO. AWY-103

DEPTH (feet)	WELL CONSTRUCTION	PID	BLOWS/FOOT	SAMPLE NUMBER	U.S.C.S. DESIGNATION	SOIL DESCRIPTION
		PPM				
0						Grass surface
						Loam
					CL	Clay; brown; moist; some fine grained sand; no odor.
						As Above; wet; increasing fine-grained sand.
10						Boring terminated at 10 feet bgs.

SURFACE ELEVATION:
TOTAL DEPTH. 10 feet
DIAMETER OF BORING: 6 inches
WATER ENCOUNTERED AT: 2.5 feet

LOGGED BY: John Patton
DRILLER: Remington Technologies
TYPE OF RIG: Cat Tractor
DATE DRILLED: 17 September 2003

DRILLING METHOD: Hollow Stem Auger
TYPE OF WELL: 2" Groundwater Monitoring
REG. NO.. U.W.

ASSET ENVIRONMENTAL SERVICES II, LLC

BORING LOG OF MW-610

Red Barns

PROJECT NO. AWY-103

APPENDIX C

LABORATORY TEST RESULTS

ATTERBERG LIMITS TEST
ASTM D 4318

CLIENT	Womack & Associates	JOB NO.	2292-25
BORING NO.	TP-3	DATE SAMPLED	05/21/09 --
DEPTH	5.2-6.2	DATE TESTED	05/30/09 LB
SAMPLE NO.	UI		
SOIL DESCR.			
LOCATION	HWY 89 Pathways		

Plastic Limit Determination

	1	2	3
Wt Dish & Wet Soil	6.55	6.59	6.62
Wt Dish & Dry Soil	5.48	5.52	5.53
Wt of Moisture	1.07	1.07	1.09
Wt of Dish	1.14	1.15	1.16
Wt of Dry Soil	4.34	4.37	4.37
Moisture Content	24.65	24.49	24.94

Liquid Limit Determination — Device Number 0860

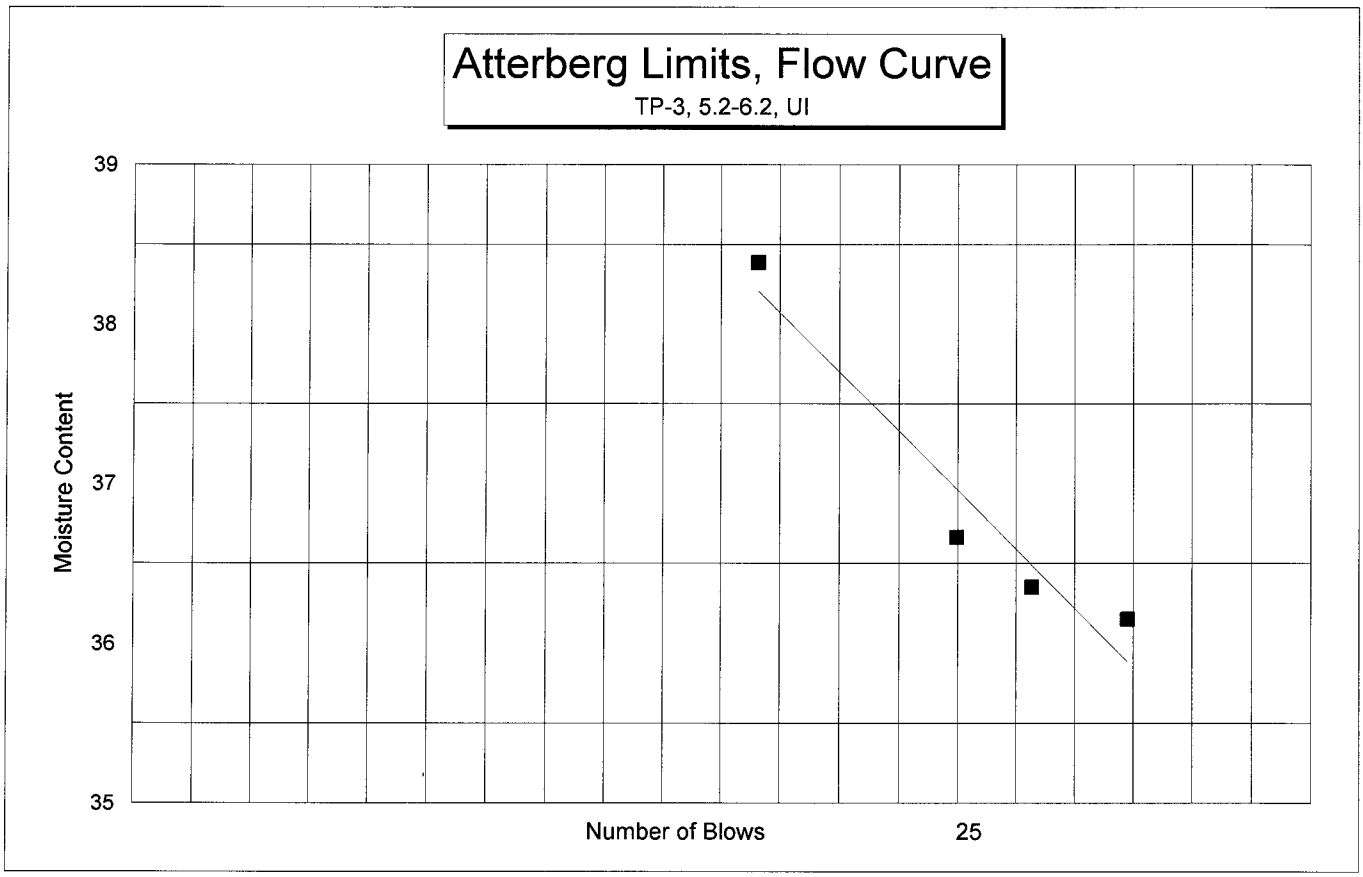
	1	2	3	4
Number of Blows	17	35	25	29
Wt Dish & Wet Soil	12.65	11.62	11.96	11.38
Wt Dish & Dry Soil	9.46	8.84	9.06	8.65
Wt of Moisture	3.19	2.78	2.90	2.73
Wt of Dish	1.15	1.15	1.15	1.14
Wt of Dry Soil	8.31	7.69	7.91	7.51
Moisture Content	38.39	36.15	36.66	36.35

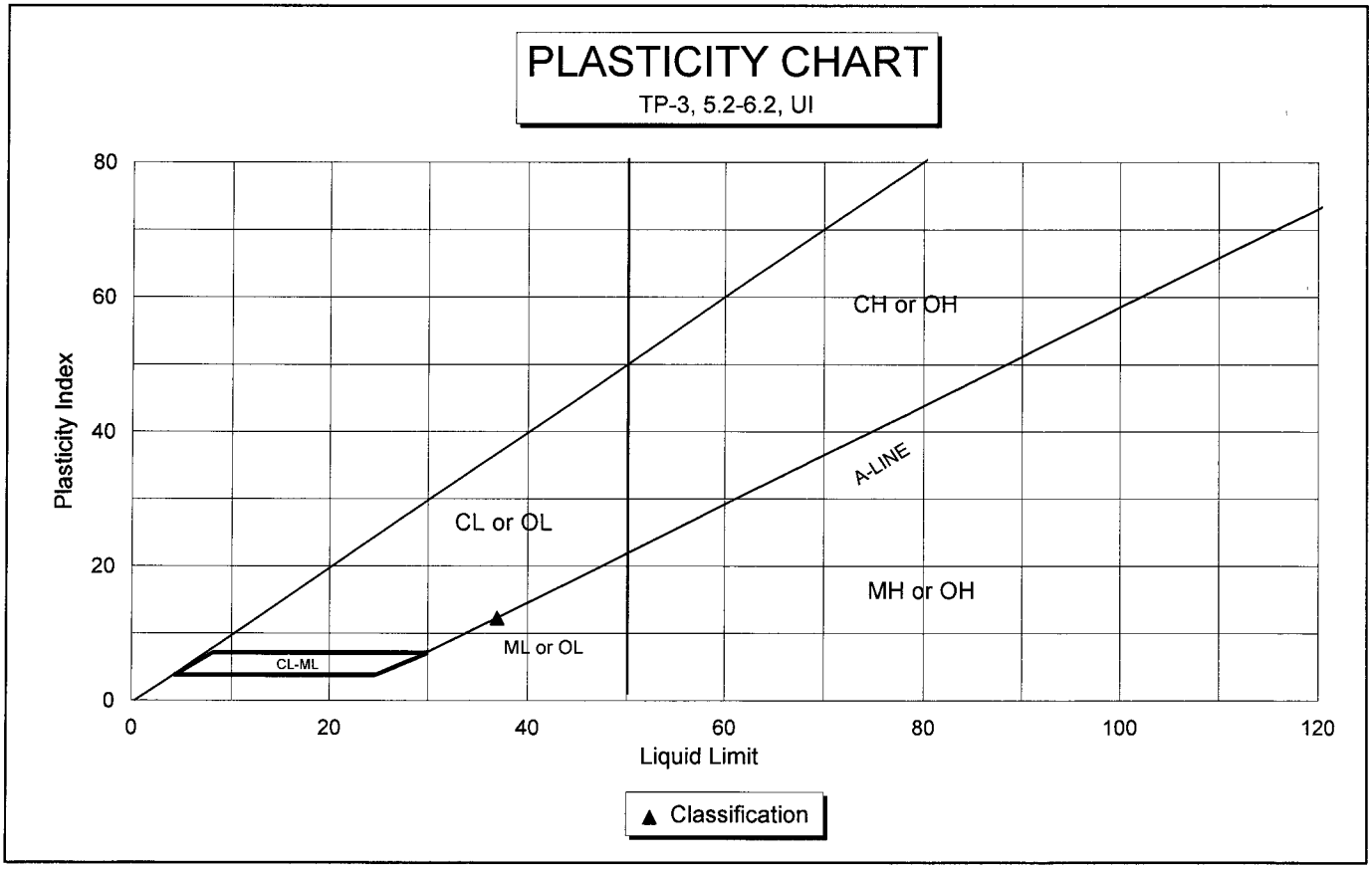
Liquid Limit	37.0
Plastic Limit	24.7
Plasticity Index	12.3

Atterberg Classification ML

Data entry by: MLM Date: 06/01/2009
Checked by: LB Date: 6-02-2009
FileName: WOG03UI

ADVANCED TERRA TESTING, INC. ATT


Atterberg Limits, Flow Curve
TP-3, 5.2-6.2, UI
39
38
37
36
35
Moisture Content
Number of Blows
25


PLASTICITY CHART
TP-3, 5.2-6.2, UI
80
60
40
20
0
Plasticity Index
CH or OH
A-LINE
CL or OL
MH or OH
ML or OL
CL-ML
0
20
40
60
80
100
120
Liquid Limit
Classification

MECHANICAL ANALYSIS - SIEVE TEST DATA - ASTM D 1140
MOISTURE AND DENSITY DETERMINATIONS - ASTM D 2216 & ASTM D 2937

CLIENT Womack & Associates

JOB NO. 2292-25

BORING NO.	TP-3	SAMPLED	
DEPTH	5.2-6.2	DATE TESTED	06/01/09 TMR
SAMPLE NO.	UI	WASH SIEVE	Yes
SOIL DESCR.		DRY SIEVE	No
LOCATION	HWY 89 Pathways		

WASH SIEVE ANALYSIS

Wt. Wet Soil & Pan Before Washing (g)	155.8
Wt. Dry Soil & Pan Before Washing (g)	151.7
Weight of Pan (g)	8.2
Wt. of Dry Soil Before Washing	143.5
Wt. Dry Soil & Pan After Washing (g)	12.7
Wt. of Dry Soil After Washing (g)	4.5
-#200 Wash. Out %	96.9

Sieve Number (Size)	Pan Weight (g)	Indiv. Wt. + Pan (g)	Indiv. Wt. Retain.	Cum. Wt. Retain.	Cum. % Retain.	% Finer By Wt.
#200	8.18	12.65	4.47	4.47	3.1	96.9

Data entered by: MLM Date: 06/02/2009
Data checked by: DPM Date: 6/2/09
FileName: WOS0P3UI

CALCULATION WORKSHEET
USCS Classification
D 2487

CLIENT	Womack & Associates	JOB NO.	2292-25
BORING NO.	TP-3	SAMPLED	
DEPTH	5.2-6.2	DATE TESTED	06/01/09 TMR
SAMPLE NO.	UI	WASH SIEVE	Yes
SOIL DESCR.		DRY SIEVE	No
LOCATION	HWY 89 Pathways		

ATTERBERG CLASSIFICATION: ML

% Gravel =	0.00
% Sand =	3.11
% Fines =	96.89

D60 =	N/A
D30 =	N/A
D10 =	N/A

Cu =	N/A
Cc =	N/A

Classification = ML, Silt

Data entered by: MLM
Data checked by: DPM
FileName: WOS0P3UI

Date: 06/02/2009
Date: 6/02/09

ADVANCED TERRA TESTING, INC. ATT

Corrosion Test Summary

Client: Womack & Associates
Project Name: HWY 89 Pathways
Date Sampled:

Job Number: 2292-25
Date Tested: 06/01/09 QRS
Project #:

Sample ID	Temperature Deg. C	pH		Conductivity (MICRO-MHO's/cm)	Resistivity (OHM-meters)		
BH-3	25.6	6.96		311	32		
0.0-16.5	Sulfates				Chlorides		
D2/D3	Reading	Dilution		ppm	Reading	Dilution		ppm
	7.0	1 (Solution)	1 (Total Volume)	7.0	7.3	1 (Solution)	1 (Total Volume)	7.3

Sample ID	Temperature Deg. C	pH		Conductivity (MICRO-MHO's/cm)	Resistivity (OHM-meters)		
BH-1	25.3	7.33		421	24		
10.0-11.5	Sulfates				Chlorides		
D2	Reading	Dilution		ppm	Reading	Dilution		ppm
	24.0	1 (Solution)	1 (Total Volume)	24.0	13.0	1 (Solution)	1 (Total Volume)	13.0

*Resistivity in OHM-meters is equal to 10,000 divided by the conductivity in micro-Mho's/centimeter

Data entry by: MLM Date: 06/01/2009
Checked by: LB Date: 6-02-2009
FileName: WOCSBH31

ADVANCED TERRA TESTING, INC. ATT

CONSOLIDATION TEST
ASTM D-2435

CLIENT	Womack & Associates	JOB NO.	2292-25
BORING NO.	TP-3	SAMPLED	05/21/09 --
DEPTH	5.2-6.2'	TEST STARTED	05/27/09 DPM
SAMPLE NO.	U1	TEST FINISHED	06/03/09 DPM
SOIL DESCR.		CELL NUMBER	ATT-16
LOCATION	HWY 89 Pathways		
SAMPLE DIA.(in)	2.410	INITIAL HT.(in)	1.000

MOISTURE/DENSITY DATA	BEFORE TEST	AFTER TEST
Wt. Soil & Ring(s) (g)	160.4	160.7
Wt. Ring(s) (g)	43.9	43.9
Wt. Soil (g)	116.5	116.8
Wet Density PCF	97.3	120.4
Wt. Wet Soil & Pan (g)	119.6	120.0
Wt. Dry Soil & Pan (g)	91.1	91.1
Wt. Lost Moisture (g)	28.6	28.9
Wt. of Pan Only (g)	3.1	3.1
Wt. of Dry Soil (g)	87.9	87.9
Moisture Content %	32.5	32.9
Dry Density PCF	73.4	90.6

LOAD (PSF)	CONSOL. (IN.)
100	0.0000
100	0.0004
200	0.0017
400	0.0054
800	0.0093
1600	0.0177
3200	0.0382
6400	0.0716
12800	0.1299
25600	0.1898

	LOAD (PSF)	LOG p	CONSOL. (IN.)	DEFL. (IN.)	VOID RATIO e
	100	2.000	0.0000	0.0000	1.2275
Inundate	100	2.000	0.0004	-0.0004	1.2266
	200	2.301	0.0017	-0.0017	1.2237
	400	2.602	0.0054	-0.0054	1.2155
	800	2.903	0.0093	-0.0093	1.2068
	1600	3.204	0.0177	-0.0177	1.1881
	3200	3.505	0.0382	-0.0382	1.1424
	6400	3.806	0.0716	-0.0716	1.0680
	12800	4.107	0.1299	-0.1299	0.9381
	25600	4.408	0.1898	-0.1898	0.8047

% Saturation	69.4
Sp. Gr. (g/cc)	2.620
Init. Ht. Solids (cm)	1.140
Init. Ht. Voids (cm)	1.400
Init. Void Ratio	1.2275

Data entry by: MLM Date: 06/05/2009
Data checked by: Cal Date: 6/5/09
FileName: WOC0TP3U

ADVANCED TERRA TESTING, INC. ATT

CLIENT	Womack & Associates	JOB NO.	2292-25
BORING NO.	TP-3	SAMPLED	05/21/09 --
DEPTH	5.2-6.2'	TEST STARTED	05/27/09 DPM
SAMPLE NO.	U1	TEST FINISHED	06/03/09 DPM
SOIL DESCR.		CELL NUMBER	ATT-16
LOCATION	HWY 89 Pathways		

TIME READING DATA

400 psf Load

Elapsed Time (min)	SQRT Time (min)	Dial Reading (in)	Defl. (in)
0.0	0.00	0.0020	0.0000
0.1	0.32	0.0040	-0.0020
0.3	0.50	0.0041	-0.0021
0.5	0.71	0.0042	-0.0022
1.0	1.00	0.0043	-0.0023
2.0	1.41	0.0045	-0.0025
4.0	2.00	0.0047	-0.0027
9.0	3.00	0.0049	-0.0029
16.0	4.00	0.0050	-0.0030
30.0	5.48	0.0051	-0.0031
60.0	7.75	0.0053	-0.0033
120.0	10.95	0.0056	-0.0036
240.0	15.49	0.0057	-0.0037
480.0	21.91	0.0058	-0.0038

800 psf Load

Elapsed Time (min)	SQRT Time (min)	Dial Reading (in)	Defl. (in)
0.0	0.00	0.0061	0.0000
0.1	0.32	0.0081	-0.0020
0.3	0.50	0.0082	-0.0021
0.5	0.71	0.0083	-0.0022
1.0	1.00	0.0084	-0.0023
2.0	1.41	0.0087	-0.0026
4.0	2.00	0.0088	-0.0027
9.0	3.00	0.0090	-0.0029
16.0	4.00	0.0092	-0.0031
30.0	5.48	0.0094	-0.0033
73.0	8.54	0.0097	-0.0036
120.0	10.95	0.0099	-0.0038
240.0	15.49	0.0103	-0.0042
480.0	21.91	0.0106	-0.0045

1600 psf Load

Elapsed Time (min)	SQRT Time (min)	Dial Reading (in)	Defl. (in)
0.0	0.00	0.0108	0.0000
0.1	0.32	0.0136	-0.0028
0.3	0.50	0.0138	-0.0030
0.5	0.71	0.0140	-0.0032
1.0	1.00	0.0143	-0.0035
2.0	1.41	0.0145	-0.0037
4.0	2.00	0.0148	-0.0040
9.0	3.00	0.0153	-0.0045
16.0	4.00	0.0154	-0.0046
30.0	5.48	0.0159	-0.0051
60.0	7.75	0.0163	-0.0055
120.0	10.95	0.0169	-0.0061
240.0	15.49	0.0176	-0.0068
480.0	21.91	0.0183	-0.0075

3200 psf Load

Elapsed Time (min)	SQRT Time (min)	Dial Reading (in)	Defl. (in)
0.0	0.00	0.0199	0.0000
0.1	0.32	0.0249	-0.0050
0.3	0.50	0.0258	-0.0059
0.5	0.71	0.0265	-0.0066
1.0	1.00	0.0273	-0.0074
2.0	1.41	0.0282	-0.0083
4.0	2.00	0.0292	-0.0093
9.0	3.00	0.0305	-0.0106
16.0	4.00	0.0315	-0.0116
30.0	5.48	0.0327	-0.0128
60.0	7.75	0.0341	-0.0142
120.0	10.95	0.0356	-0.0157
240.0	15.49	0.0372	-0.0173
480.0	21.91	0.0391	-0.0192

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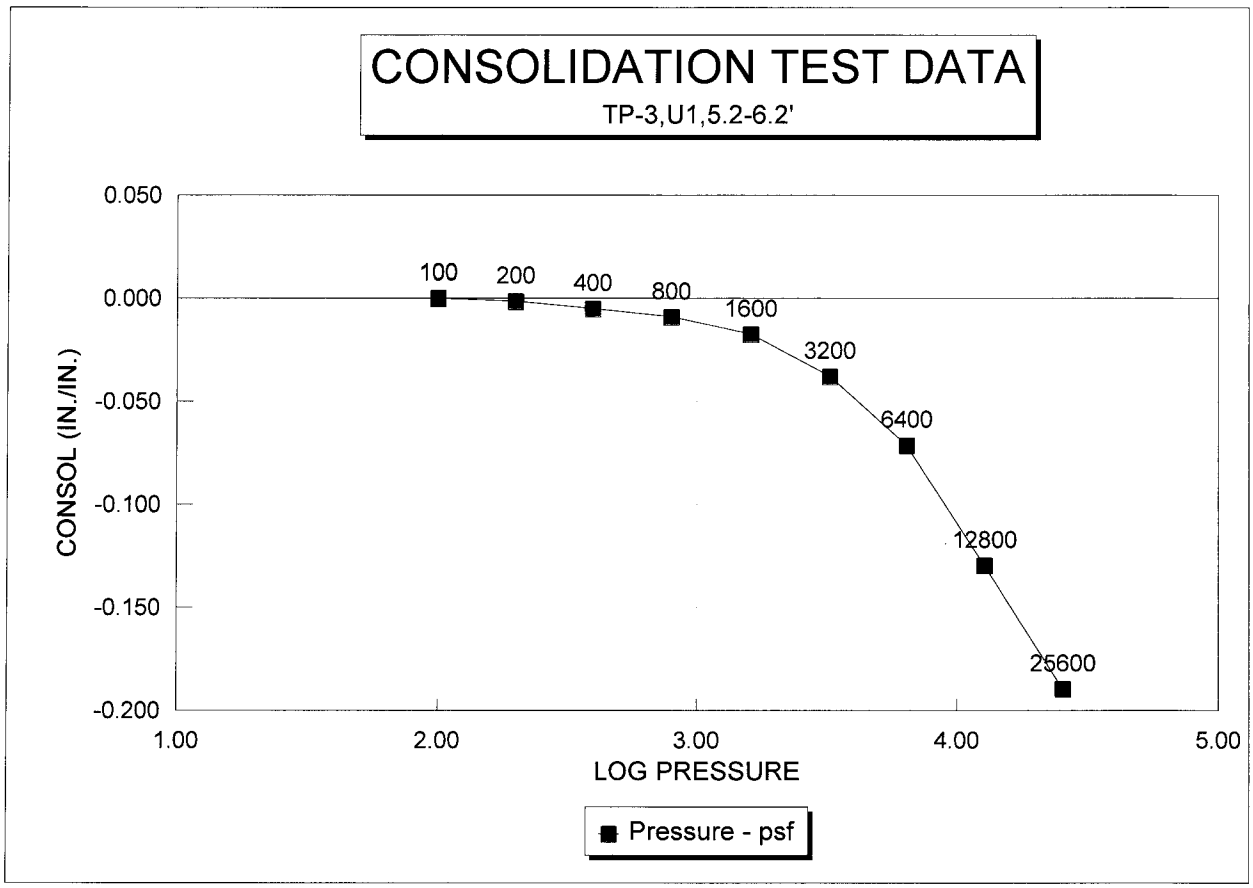
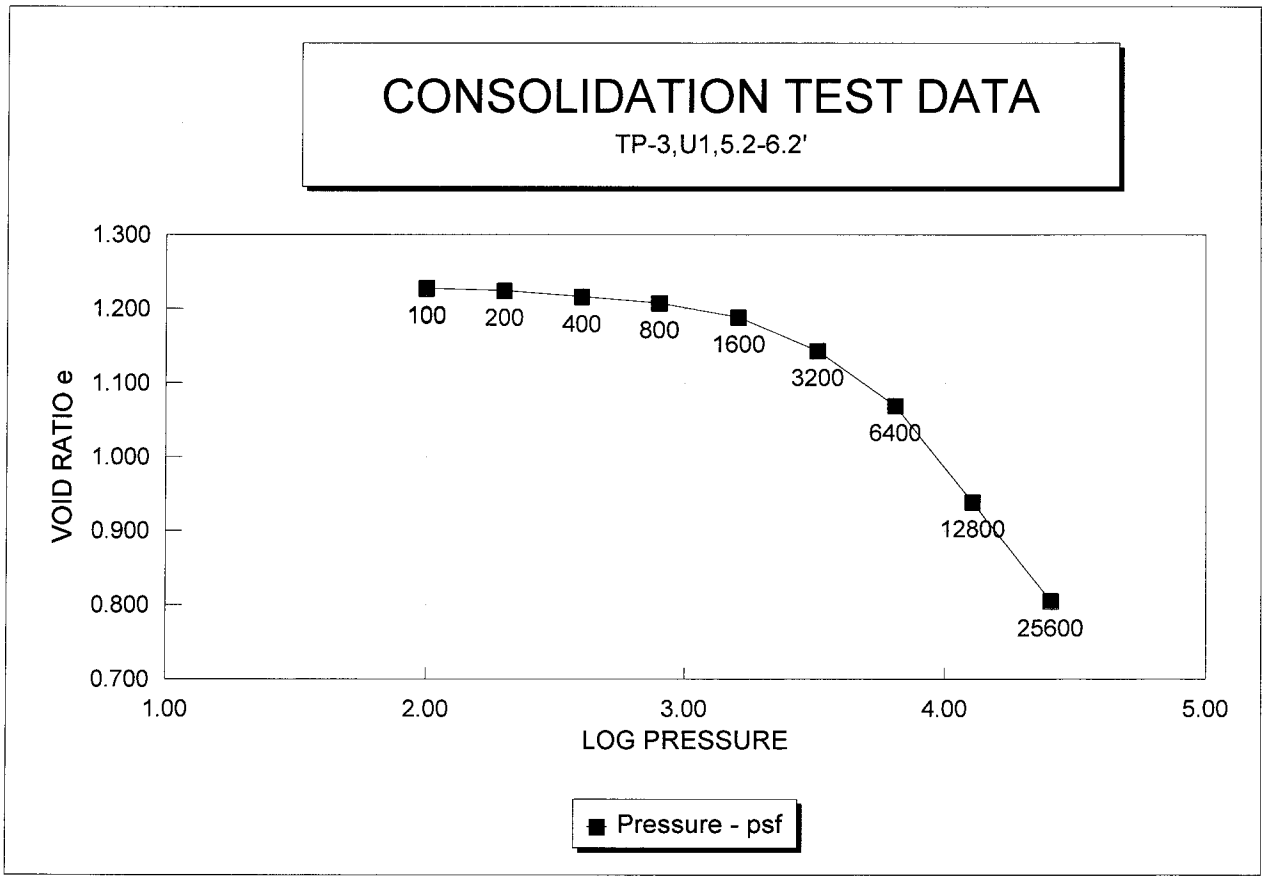
Data checked by: Cal Date: 6/5/09
FileName: WOC0TP3U

ADVANCED TERRA TESTING, INC. ATT


CONSOLIDATION TEST DATA
TP-3,U1,5.2-6.2'
0.050
0.000
-0.050
-0.100
-0.150
-0.200
CONSOL (IN./IN.)
100
200
400
800
1600
3200
6400
12800
25600
1.00
2.00
3.00
4.00
5.00
LOG PRESSURE
Pressure - psf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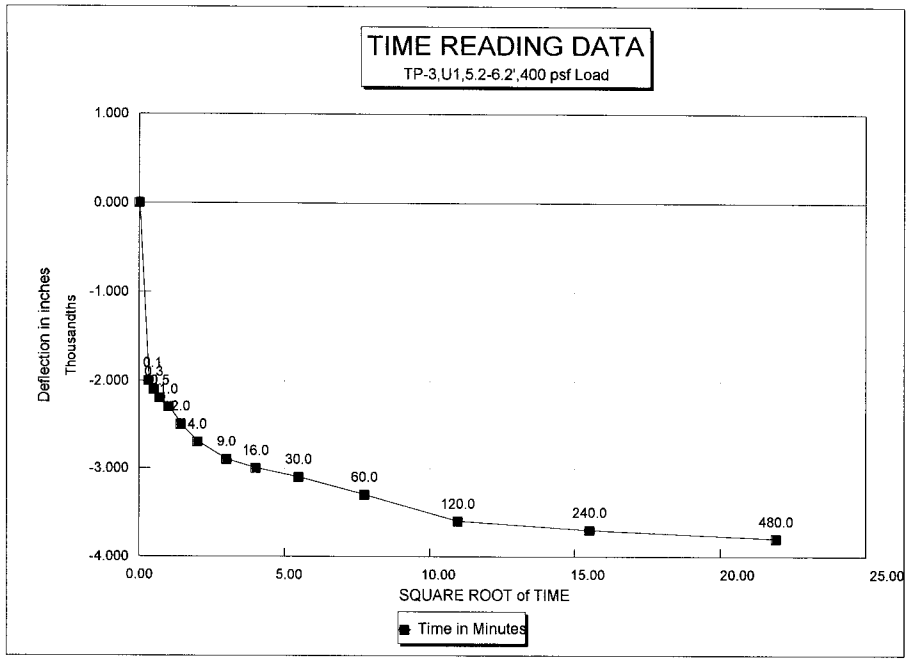
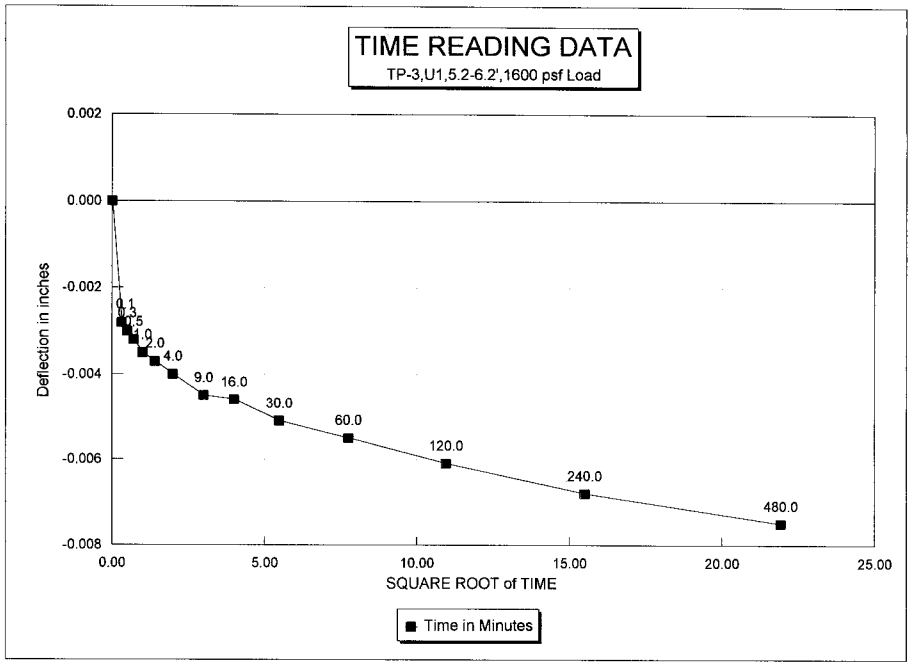
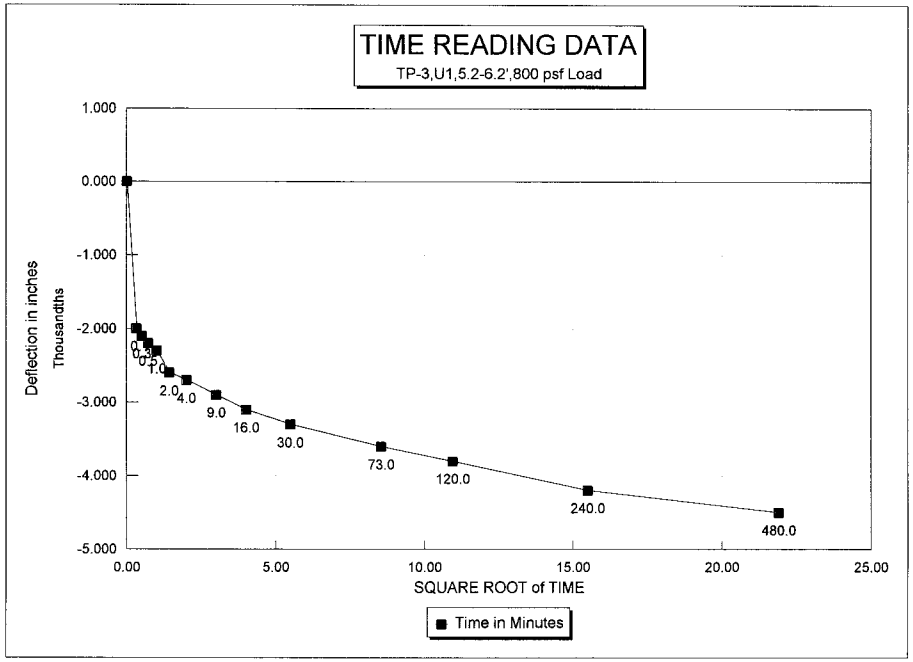
TP-3,U1,5.2-6.2'
1.300
1.200
1.100
1.000
0.900
0.800
0.700
VOID RATIO e
100
200
400
800
1600
3200
6400
12800
25600
1.00
2.00
3.00
4.00
5.00
LOG PRESSURE
Pressure - psf


TIME READING DATA
TP-3,U1,5.2-6.2',400 psf Load
Deflection in inches
Thousandths
1.000
0.000
-1.000
-2.000
-3.000
-4.000
0.00
5.00
10.00
15.00
20.00
25.00
SQUARE ROOT of TIME
0.1
0.3
0.5
1.0
2.0
4.0
9.0
16.0
30.0
60.0
120.0
240.0
480.0
Time in Minutes


TIME READING DATA
TP-3,U1,5.2-6.2',1600 psf Load
Deflection in inches
0.002
0.000
-0.002
-0.004
-0.006
-0.008
0.00
5.00
10.00
15.00
20.00
25.00
SQUARE ROOT of TIME
0.1
0.3
0.5
1.0
2.0
4.0
9.0
16.0
30.0
60.0
120.0
240.0
480.0
Time in Minutes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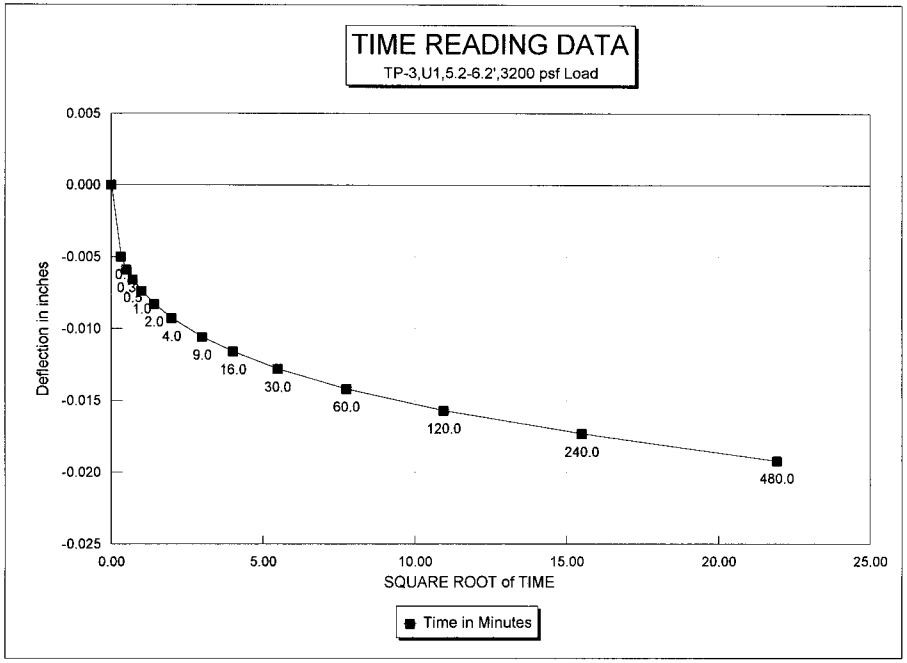
TP-3,U1,5.2-6.2',800 psf Load
Deflection in inches
Thousandths
1.000
0.000
-1.000
-2.000
-3.000
-4.000
-5.000
0.00
5.00
10.00
15.00
20.00
25.00
SQUARE ROOT of TIME
0.1
0.3
0.5
1.0
2.0
4.0
9.0
16.0
30.0
73.0
120.0
240.0
480.0
Time in Minutes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Deflection in inches
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0.000
-0.005
-0.010
-0.015
-0.020
-0.025
0.00
5.00
10.00
15.00
20.00
25.00
SQUARE ROOT of TIME
0.1
0.3
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1.0
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4.0
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30.0
60.0
120.0
240.0
480.0
Time in Minutes

APPENDIX D

RETAINING WALL CONSIDERATIONS

Table 9-6 Classification of Retaining Wall Systems(14m)

Wall Category	Wall Type	Construction Type[1]	Wall Group	Design	Constraints
Externally Stabilized Wall Types	Anchored Walls (Sheeting or Soldier Pile & Lagging Walls)	Cut Wall	Deadman Anchors	Designed & detailed in contract.	Typical Height Range: 5 to 20m
			Grouted Tiebacks	Detailed in contract. Designed by Contractor's Design Consultant.	
			Braced Walls	Designed & detailed in contract.	
	Soldier Pile & Lagging Walls	Cut Wall		Designed & detailed in contract.	Typical Height Range: 2 to 5 m
	Sheeting Walls	Cut Wall		Designed & detailed in contract.	Typical Height Range: 2 to 5 m Maximum Wall Height= 4.5 m
Internally Stabilized Wall Types	Mechanically Stabilized Earth System (MSES)	Fill Wall		Detailed in contract. Designed by Contractor's Designer/Supplier (Proprietary Wall)[2].	Typical Height Range: 3 to 20m
	Geosynthetic Reinforced Earth System (GRES)	Fill Wall		Designed & detailed in contract.	Typical Height Range: 2 to 15m
	Soil Nail Wall System	Cut Wall		Detailed in contract. Designed by Contractor's Design Consultant	Typical Height Range: 3 to 20m
	Mechanically Stabilized Segmental Block Retaining Wall System	Fill Wall		Detailed in contract. Designed by Contractor's Designer/Supplier (Proprietary Wall)[2].	Typical Height Range: 2 to 11m
Gravity and Cantilever Wall System	Gravity Wall	Primarily Fill Wall. May be installed as a Cut wall.	Segmental Block Retaining Wall System	Detailed in contract. Designed by Contractor's Designer/Supplier (Proprietary Wall)[2].	Typical Height Range: 1 to 2 m
			Sta-Wall	Detailed in contract. Designed by Contractor's Designer/Supplier (Proprietary Wall)[2].	Maximum Wall Height = 7 m
			T-Wall	Detailed in contract. Designed by Contractor's Designer/Supplier (Proprietary Wall)[2].	Typical Height Range: 2 to 15 m
			Evergreen Wall	Detailed in contract. Designed by Contractor's Designer/Supplier (Proprietary Wall)[2].	Maximum Wall Height = 7 m
			Gabion	Designed & detailed in contract.	Typical Height Range: 2 to 6 m Maximum Wall Height = 6 m
			CIP Mass Gravity	Designed & detailed in contract.	Typical Height Range: 1 to 3 m Maximum Wall Height = 7 m
	Cantilever Wall	Primarily Fill Wall. May be installed as a Cut wall.	Precast Cantilever Wall	Designed & detailed in contract.	Typical Height Range: 2 to 9 m
			CIP Cantilevered Wall	Designed & detailed in contract.	Typical Height Range: 2 to 9 m Maximum Wall Height = 7 m

[1] Cut wall construction is refers to a wall system in which the wall is constructed from the top of the wall to the base (i.e., "top-down" construction). Fill wall construction is refers to a wall system in which the wall is constructed from the base of the wall to the top (i.e., "bottom-up" construction).

[2] Use of proprietary systems must be justified in writing, in accordance with 23 CFR 635.411.

Table 2 Average Wall Costs in Northwest U.S.
(Reprinted with permission of R.G. Chassie, 1988)

Wall Type	Average Cost in Dollars per S.F. of Wall Face
Reinforced Earth	$18 - $22
VSL Retained Earth	$18 - $22
Wire Wall	$13 - $20
Geotextile Walls	
w/o Permanent Facing	$ 6 - $12
w/ 3" (76mm)Shotcrete Face	$15 - $25
Tensar Walls	
w/o Permanent Facing	$15 - $25
w/ Permanent Facing	N/A
Metal Bin Wall (H=10'[3m] to 30'[9m])	$20 - $40
Gabion Wall (H=6'[1.8m] to 21'[6.4m])	$18 - $35
Doublewall	$20 - $25
Criblock	$12 - $25
Permanent Tieback Walls	$45 - $60
Soil Nailed Walls	
(U.S. Experience is Limited)	
Temporary	$17 - $25
Permanent (Shotcrete Face)	$25 - $30
Permanent (Architectural Facing)	$35 - $50
Cantilever Soldier Pile	
w/Wood Lagging (H=5'[1.5m]to[4.6m]15')	$12 - $18
Cantilever Sheet Pile Wall	
(H=5'[1.5m] to [4.6m]15')	$12 - $18
Concrete Cantilever	
(H=6'[1.8m] to 30'[9.1m])	$25 - $40

NOTES:

1. Average prices given above are <u>total</u> in-place costs including wall materials, erection, and select wall backfill. For MSE walls, the select backfill is the backfill contained within the reinforced volume. Since excavation quantities can vary greatly job-to-job, excavation cost is <u>not</u> included.
2. Unless otherwise noted, the prices given cover walls in the <u>10'[3m] to 40'[12m]</u> height range.
3. Alaska prices are higher.
4. Not all systems listed above are marketed in all states.
5. N/A denotes no cost experience to date in Region 10 [Northwest U.S.].

Long term costs of alternate wall systems are difficult to assess when both monitoring and maintenance data are only available for a small percentage of the system's designed life. The owner must rely on the engineering staff to review the longevity of innovative designs. Estimation of the design life and a long term cost is more difficult for wall systems constructed with materials other than concrete, steel and wood. New wall materials such as plastics, fiberglass, and other

TABLE 1: ENGINEERING CONSIDERATIONS FOR SELECTION OF NON-PROPRIETARY PERMANENT RETAINING WALLS[1]

Engineering considerations / Wall type	Relative construction time	Maximum height[2]	Sensitivity to differential settlement	Approx. base width	Potential settlement of retained mass	Typical cost per sq.ft. of wall face[3]
Cast-in-place concrete	long	50± ft.	high	0.5 x wall height	medium	$60 to $100
Concrete cribbing	short	16 ft. (24 ft.[7])	low	0.5 x wall height	medium to high	$35 to $60
Sheet piling or H-pile and lagging--cantilever	short	15± ft.	low (high[5])	< 3 ft.	high	$30 to $60[6] $50 to $90[5,6]
Sheet piling or H-piles and lagging--tied back	medium	50± ft.	high	< 5 ft.[7]	low	$40 to $ 80[6] $60 to $110[5,6]
Gabion wall	medium	25± ft.[4]	low	0.5 x wall height	high	$20 to $60
Geotextile wall	medium	20± ft.[4]	low (medium[8])	0.5 x wall height	medium to high	insufficient data

[1] Based on NYSDOT experience
[2] Greater heights require special study
[3] Does not include cost of excavation and temporary excavation support. Includes cost of backfill.
[4] Requires batter
[5] With cast-in-place concrete facing
[6] Does not include cost of pre-drilling for H-piles where this is required.
[7] Easement needed for tiebacks if they extend outside of right-of-way
[8] With shotcrete facing.

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TABLE 2: ENGINEERING CONSIDERATIONS FOR SELECTION OF PROPRIETARY PERMANENT RETAINING WALLS[1]

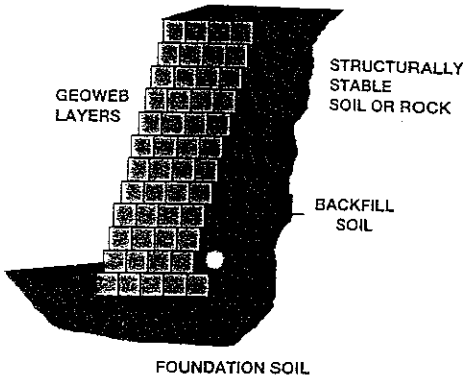
Engineering considerations / Wall type	Relative construction time	Maximum height[2]	Sensitivity to differential settlement	Approx. base width	Potential settlement of retained mass	Typical cost per sq.ft. of wall face[3]
MSES (Reinforced Earth® or Retained Earth™) (2, 3)	medium	50± ft.	medium[4]	0.75 x wall height[5]	low to medium	$30 to $45[6]
Doublewall® (4)	short	40± ft.	medium	0.5 x wall height	medium	insufficient data
Evergreen® (5)	short	30± ft.[7]	medium	0.5 x wall height	medium to high	insufficient data
Tensiter N (6)	medium	16 ft.	high	0.5 x wall height	medium	insufficient data

(1 ft. = 30.48 cm)

[1] Based on NYSDOT experience
[2] Greater heights require special study
[3] Does not include cost of excavation and temporary excavation support. Includes cost of backfill. 1989 prices
[4] Reinforced Earth Company criterion: diff. sett. must be less than 1 ft. per 100 ft. of wall (?)
[5] Greater for walls supporting bridge abutments
[6] For walls with a face area of at least 1500 sq.ft.
[7] Requires batter

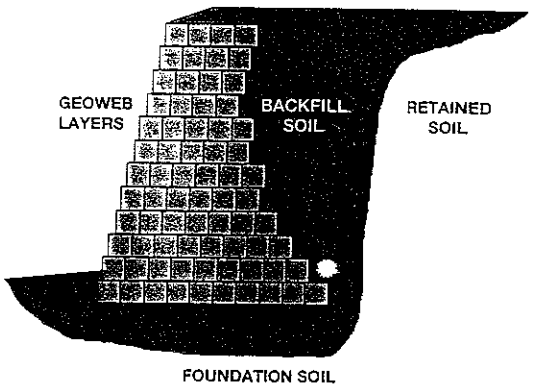
Retaining Walls

The Geoweb® system's multi-layer design makes it very adaptable to specific retaining wall application requirements. Classic methods of stability analysis can be applied, and computerized evaluation is available for a broad range of infill, backfill and surcharge parameters. For the wall exterior, choose from a range of non-structural facias or take advantage of the Geoweb® system's vegetative support capabilities. Simple, effective construction techniques make the Geoweb® system ideal for installations in remote or restricted access sites.



Gravity Retaining Walls

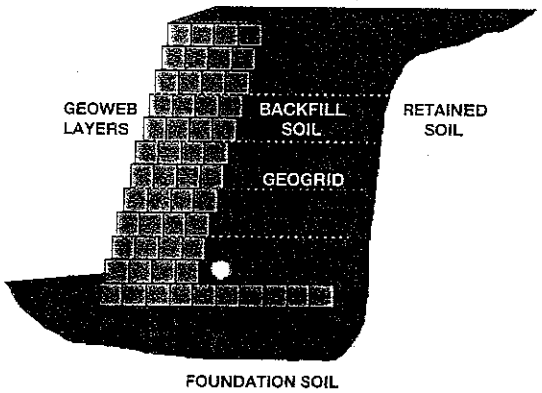
The Geoweb system confines and reinforces granular fill, producing a uniform structural mass that resists lateral pressures and maintains structural integrity. Significant subgrade deformation can occur without loss of structural integrity. Geoweb sections can be used in vertical or stepped designs and the system supports non-structural facias or natural vegetative cover.



Composite Retaining Walls

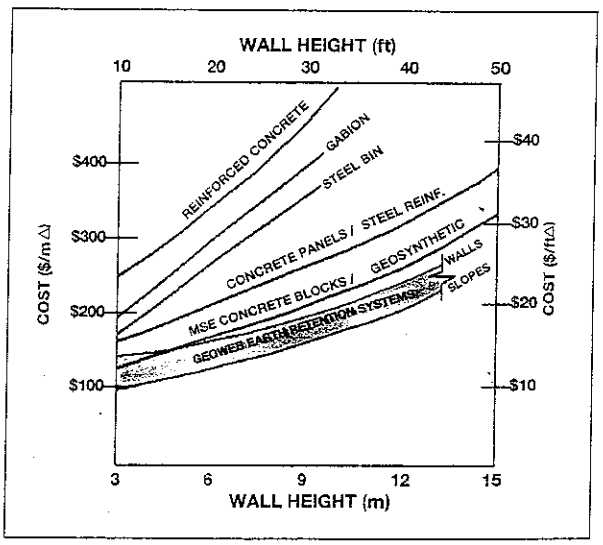
Cellular confinement with the Geoweb system eliminates the need for expensive structural facia panels. The Geoweb system creates a totally confined wall facing that can be united with the backfill using a variety of tie-back systems. Outer cells can be filled with topsoil to support natural vegetative cover.

The Geoweb retaining wall system is environmentally friendly, blending into the environment with vegetated facings and a choice of natural colors. The vegetated face treatment also provides a surface which has noise-absorbing tendencies.



Cost-Effective Option

Geoweb retention structures are cost-competitive with conventional earth retention systems. This graph illustrates that depending upon wall height, Geoweb retention structures can offer a 25% to 50% cost savings over cast-in-place concrete retaining walls. The installed cost for all earth retention systems will vary with site-specific conditions such as accessibility, soil conditions, cost of infill and compaction, labor rates, surcharge loading, length of wall, etc. This installed cost graph indicates relative cost competitiveness by comparing Geoweb structures built in 1988 with the cost of more conventional earth retention construction methods as compiled by the California DOT in 1986.



Retaining Walls

APPLICATIONS AND DESIGN CHALLENGES

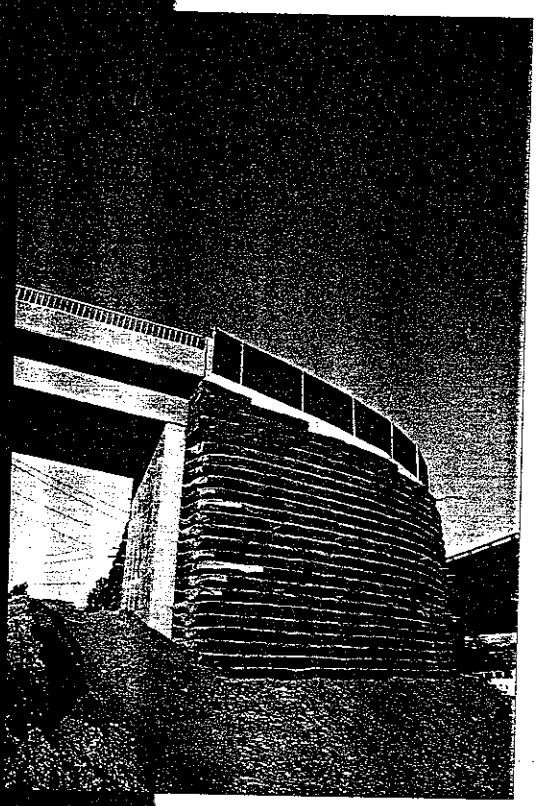
Retaining walls allow property owners to maximize their land use. However, building a concrete gravity or crib wall is often impractical because of the high construction cost.

GEOTEXTILE SOLUTIONS AND BENEFITS

Geotextiles are used for a wide assortment of reinforcement applications, including embankments over soft soils, levees and retaining walls. Geotextiles are well-suited to wall construction with timber, precast panel and modular block facing.

Amoco Geotextiles make retaining walls financially feasible. In fact, a geotextile retaining wall can be built for less than half the cost of a conventional wall. Our woven geotextiles offer other significant advantages over conventional methods, such as simplified installation and construction, and the ability to use on-site backfill material.

Engineers like the fact that Amoco Geotextiles give them flexibility in design. Amoco reinforced geotextile walls can be designed using AASHTO, FHWA, NCMA and U.S. Forest Service design methods. Our technical support resources can help you select the property values required for each.



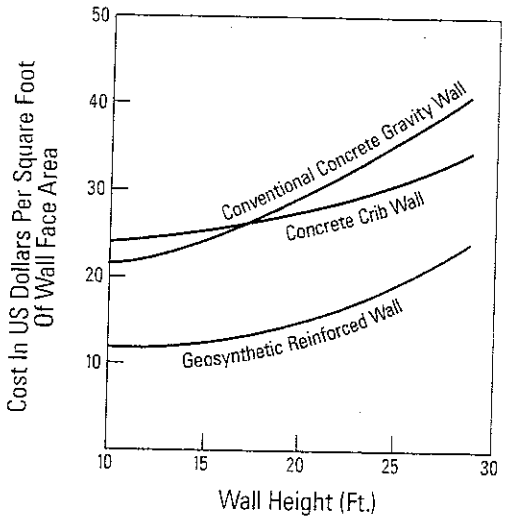


Figure 1 - Cost comparison of conventional and geosynthetic reinforced retaining walls.

Among geosynthetics commonly used to reinforce retaining walls, geotextiles stand out as the most cost-competitive.