Geotechnical Report No. 15-08

Geotechnical Investigation

KOBUK BRIDGE

AK DEN 2007(1)
NORTHWEST ARCTIC BOROUGH
ALASKA

December 2008

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

WESTERN FEDERAL LANDS HIGHWAY DIVISION
VANCOUVER, WASHINGTON

Geotechnical Services

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INTRODUCTION

This report provides results and recommendations from a geotechnical investigation conducted for a bridge replacement near Kobuk, Alaska. The proposed project involves replacing a small all terrain vehicle (ATV) bridge in rural northwest Alaska. The existing structure is in poor condition and is not safe for support of ATVs or snow machines which frequently use the trail.

The bridge crosses Wesley Creek (upper Wesley Creek) on a trail approximately 7 miles northwest of the Native Village of Kobuk and approximately 10 miles northeast of the village of Shungnak (Figure 1). The bridge provides an alternative route between these two villages when conditions prevent the use of the "lower trail" as well as access to points north. Kobuk and Shungnak are located in the Northwest Arctic Borough approximately 200 miles east of Kotzebue, Alaska. Access to this region is by airplane, or by snow machine or dog sled in the winter. Gravel airstrips are located in both Kobuk and Shungnak. A river barge travels to the area once a year in the spring when water conditions in the Kobuk River allow passage.

Purpose and Scope

The purpose of the investigation was to evaluate geologic conditions at the project location, and provide geotechnical recommendations for the proposed bridge replacement. Investigation involved surface reconnaissance, material sampling, and density probing of in-situ materials. Extensive subsurface investigation was not feasible due to the remote location and budgetary constraints.

The bridge to be replaced is a timber bridge approximately 36 feet in length. The proposed bridge is a steel structure approximately 60 feet in length, spanning the creek and existing abutments. No work is proposed for the approach roads or existing abutments. An agreement was made between partner agencies to leave the abutments in place to provide scour protection and prevent in-water work. Additionally, it was agreed that the new structure would be supported on shallow foundations. Limited heavy equipment is available in the area and much, if not all, of the construction work will be completed using hand labor and any available power equipment. Project photographs are shown in Appendix A and a site map is shown in Figure 2.

General Geologic Setting

The project is located in a mountain valley on the southern flanks of the Brooks Range near the transition from the mountainous region to the north and the wide, flat Kobuk river valley to the south (Figure 1). Elevation at the project site is approximately 600 feet.

According to the report and accompanying map, *Geology and Geochemistry of the Cosmos Hills, Ambler River and Shungnak Quadrangles, Alaska*, by C. E. Fritts, 1970, bedrock underlying the area consists of weakly metamorphosed sedimentary rock with metaconglomerate common. Although no bedrock outcrops were observed at or near the project location, it is interpreted that this rock unit underlies the site at unknown depth. Based on field observations, bedrock is overlain at the project site by alluvial material consisting mainly of gravel, sand, and cobbles.

Climate

According to the Western Climate Data Center's information for Kobuk, AK, the area receives an average of 17 inches of precipitation a year, which includes 54 inches of snow. Average maximum temperature in the warmest summer months is in the mid to upper 60s Fahrenheit and the average minimum temperature in the coldest winter months is about -20 degrees Fahrenheit.

EXPLORATION AND LABORATORY TESTING

Explorations

A geologic field investigation was conducted in June 2008. Field work involved observations and descriptions of geologic materials and processes in the area of the proposed bridge replacement. On-site surficial materials were described following general geotechnical practices and the Unified Soil Classification System (USCS).

Hand auger holes were attempted at four different locations, two at each abutment. However, due to the prevalence of gravel and cobbles, the hand auger could not be advanced deeper than about 10 inches.

Density drive probes were completed at 5 different locations, 3 locations at the south abutment and 2 locations at the north abutment (Figure 2). The drive probes were completed using a Wildcat Dynamic Cone Penetrometer, manufactured by Triggs Technologies. This method involves driving 1 inch diameter steel rods with a sacrificial conical tip into the ground using a 35 pound manual hammer dropping 15 inches.

Resistance to driving the Wildcat density probe was then correlated to Standard Penetration (SPT) N values using the manufacturer's recommended conversion factor. Drive probe WCH1 at the south abutment was advanced to 8 feet below ground surface (BGS) before refusal and drive probe WCH3 reached close to 5 ½ feet BGS prior to refusal. All other attempts reached refusal in less than 4 feet BGS, likely on large cobbles or boulders. Drive probe results are shown in Appendix C.

Four representative grab samples were collected of on-site soil materials. Two samples were collected from each abutment, one from the roadway and one from the abutment fill slope. Sample locations are illustrated in Figure 2. Samples were obtained using either the hand auger or a shovel at depths ranging from about 0 to 12 inches below ground surface. Properties of sampled materials were field evaluated and recorded. Samples were then placed in air tight bags to prevent moisture loss and shipped to the WFLHD materials testing laboratory.

Laboratory Testing

Basic engineering index tests were completed by the WFLHD materials testing laboratory in Vancouver, Washington. Tests performed included: grain size analysis (T88), Atterberg limits (T89 and T90), specific gravity (T100) and natural moisture content. Test results were used to refine field classification of geologic materials for use in analysis and recommendations. Material testing results are shown in Appendix B.

SITE CHARACTERIZATION

Surface Conditions

In the vicinity of the proposed bridge replacement, the road/trail trends generally north/south following Wesley Creek. The gravel roadway ranges from approximately 8 to 12 feet in width and is passable with a four-wheel drive vehicle but is best suited, in its current condition, to ATV travel. Immediately downstream of the bridge, a relatively wide and shallow section of the creek is used as an alternate crossing at low water levels and in winter when snow and ice cover the creek.

Wesley Creek is a relatively small mountain drainage with an average bankfull width of approximately 15 to 20 feet at the project location. Peak discharge, estimated scour, and other hydraulic information is contained in the WFLHD Hydraulics Bridge Design Report (Appendix D).

Surficial materials in the area of the bridge are alluvial deposits consisting primarily of gravel and sand with cobbles and some silt. This material likely classifies as GM, GP-GM, or GP in the USCS. Boulders are relatively common in the streambed and surrounding area up to approximately 2 feet in diameter, and boulders were observed up to 5 feet in diameter in the vicinity. Bedrock was not observed in the project area.

Abutments and Subsurface Conditions

The existing abutments consist of placed fill partially retained by log crib walls. Thickness of abutment fill is approximately 7 to 10 feet at both abutments. Full height of the crib walls is unknown, but they appear to be approximately 4 to 7 feet with the base of the walls near the existing stream bed. Condition of the walls is poor to moderate with some degradation and failure evident. It was not possible to determine if the walls are undermined as the base of the walls were not visible beneath the water at the time of investigation.

There did not appear to be any embankment settlement, and foundation conditions for the abutment fill are expected to be good. For both abutments, the fill is likely founded on alluvial deposits consisting mainly of sand and gravel, with cobbles and silt. The abutment fill slopes appear to be stable at a slope angle equal to or less than approximately 36 degrees.

Based on visual observations and results of the material testing, the abutment fill is alluvial material composed dominantly of gravel with varying amounts of sand, silt, and cobbles. It is expected that this fill was obtained from excavating in-situ materials just downstream of the existing bridge. A few boulders were observed in the abutment fill, up to approximately 2 feet in diameter. The sampled material classified as either a Silty Gravel with Sand or a Poorly Graded Gravel with Silt and Sand, a GM or GP-GM respectively in the USCS. Testing results are shown in Appendix B and summarized in Table 1 below. In table 1, Fines is defined as the material passing the # 200 sieve which is material less than .0029 inches in diameter.

Table 1: Summary of Material Testing Results

Sample	Abutment	Location	Depth (in)	USCS	Percent Gravel	Percent Sand	Percent Fines
KB1	South	Roadway	0 - 10	GM	51.2	35.9	12.9
KB2	South	Fill Slope	6 - 12	GM	59.8	24.3	15.9
KB3	North	Roadway	0 - 9	GP-GM	68.2	24.3	7.5
KB4	North	Fill Slope	4 - 12	GM	57.2	22.6	20.2

Relative density of the material at both abutments is generally medium dense to dense. The 3 drive probes completed at the south abutment indicate material densities ranging from very loose to very dense. However the majority of the readings indicated the material was medium dense or denser. The 2 drive probe locations at the north abutment indicated material densities ranging from medium dense to very dense with the majority of the results showing medium dense to dense material. It should be noted that reliability of the drive probe results is low due to the presence of large gravel, cobbles, and boulders and the inherent uncertainty associated with evaluating consistency of coarse grained soils with this or any other type of device. Complete results are shown in Appendix C.

Existing Bridge Conditions

The existing bridge is a non-engineered, timber bridge approximately 36 feet long and 8 to 9 feet wide. The bridge is constructed of uncut logs varying in size from about 4 to 10 inches in diameter. Foundation conditions are unclear but it appears that the three primary, longitudinal timber girders are sitting directly on abutment fill. Timber struts have been added at each abutment for additional support and in some locations are being undermined by slope erosion.

Groundwater and Permafrost

Groundwater elevation is expected to closely match elevation of the river. Drive probes were dry or only slightly damp when removed and thus did not provide any evidence of groundwater within the zone of exploration. It is expected that permafrost has not developed in the abutment fills due to the free draining nature of the fill material and the elevation of the fill above the surrounding terrain.

CONCLUSIONS AND RECOMMENDATIONS

It is the understanding of the WFLHD Geotechnical group that an agreement has been made between the partner agencies to allow for the bridge to be founded on shallow foundations. Further, potential scour concerns will be partially mitigated through retention of the existing log crib walls for sacrificial erosion protection and by providing a longer span to allow for some erosion of the abutment slopes. It is also understood that heavy equipment will likely not be available for placement and construction of the new structure. Therefore the following recommendations are based on the assumption that the bridge foundations will be constructed using hand labor.

Bridge Length

It is recommended that the length of the new bridge be at least 60 feet. This will allow for failure of the existing crib walls and development of eroded slopes of 30 degrees without undermining the bridge foundations. Based on field observation and alignment of the existing channel, it appears that erosion of the south abutment is more likely. Therefore, it is recommended that the bridge be slightly offset to the south side to allow for additional erosion at the south abutment while still permitting at least a 30 degree slope to develop at the north abutment if the crib wall fails.

Foundation Type

A spread footing consisting of a sill/strip type foundation is recommended for each abutment with at least 1 foot of embedment below ground surface. The minimum dimension of the strip foundations should be 11 feet by 2 feet for each abutment with the long axis of the foundation perpendicular to the long axis of the bridge. A 6 inch leveling pad of granular material is recommended beneath the foundation. This material should be compacted as well as possible by tamping with a heavy object or other hand method. It is recommended that the footing be constructed using treated timber or reinforced concrete.

Bearing Capacity and Settlement

At the south abutment, ultimate bearing capacity was calculated to be 2300 psf based on an 11 by 2 feet strip foundation. Using a factor of safety (FS) of 2.5, allowable bearing capacity is 920 psf and using a FS of 2, allowable bearing capacity is 1150 psf. Bearing capacity calculations were made using allowable stress design methodology following the recommendations, equations, and correction factors as presented in the Federal Highway Administration Geotechnical Engineering Circular No. 6, Shallow Foundations, 2002. Calculations were based on an estimated angle of

internal friction of 32 degrees, a unit weight of 120 pcf, and groundwater at an elevation of 586 feet. Groundwater is based on information presented in the Hydraulics Report (Appendix D). Bearing capacity is relatively low at the south abutment for two primary reasons: 1) the close proximity of the footing to the top of the slope and 2) the possible high groundwater level.

At the north abutment, ultimate bearing capacity was calculated to be 8000 psf based on an 11 by 2 feet strip foundation. Using a FS of 2.5, allowable bearing capacity is 3200 psf and using a FS of 2, allowable bearing capacity is 4000 psf. Bearing capacity calculations for the north abutment were made based on the same soil and site condition criteria as presented above for the south abutment and followed the same calculation methodology.

Long term consolidation settlement is expected to be negligible based on the assumed granular nature of subsurface materials. Immediate, construction-induced settlement is expected to be minimal and likely less than ½ inch.

General Earthwork

It is likely that boulders up to at least 2 feet in diameter and possibly larger will be encountered in the excavation for the foundations. Encountered boulders may require levering bars or power equipment adequate for safe removal.

On-site native materials in the vicinity of the bridge and existing abutment materials appear to consist dominantly of sand and gravel with some cobbles, boulders, and silt. This material is suitable for building embankments and placing as backfill. It should be compacted as well as possible whenever disturbed and replaced. Optimum moisture content for compaction is expected to be in the range of 12 to 14 percent. The natural moisture content of the material, as sampled during the June 2008 investigation, ranged from about 5 to 12 percent (Appendix B). Any disturbed and reconstructed slopes, or newly constructed slopes, should be built no steeper than 1V:1.5H (approximately 34 degrees). Temporary excavation slopes should be cut no steeper than 1V:1H.

Other Considerations

To preserve the life of the bridge and protect the investment, it is recommended to eventually armor the abutment slopes using riprap. Based on current conditions, armoring the south abutment should take priority over armoring the north abutment, although it is recommended to provide revetment to both abutments.

CONCLUDING REMARKS

This report was prepared to provide results of the geotechnical investigation conducted for a potential bridge replacement over "upper" Wesley Creek, northwest of Kobuk, Alaska, and to provide geotechnical recommendations for construction of the project. Recommendations presented herein are applicable only to this site and should not be used for other purposes. It should be understood that extensive subsurface investigations were not conducted and potentially unfavorable subsurface conditions may exist which could potentially impact the project. If adverse subsurface conditions are encountered during construction, it is recommended that a geotechnical engineer be consulted for further evaluation and revised recommendations.

Report By:
Nathan Jenks, Geological Engineer
1/30/09

Reviewed By:
Malcolm Ulrich, Geotechnical Team Lead
2/2/09

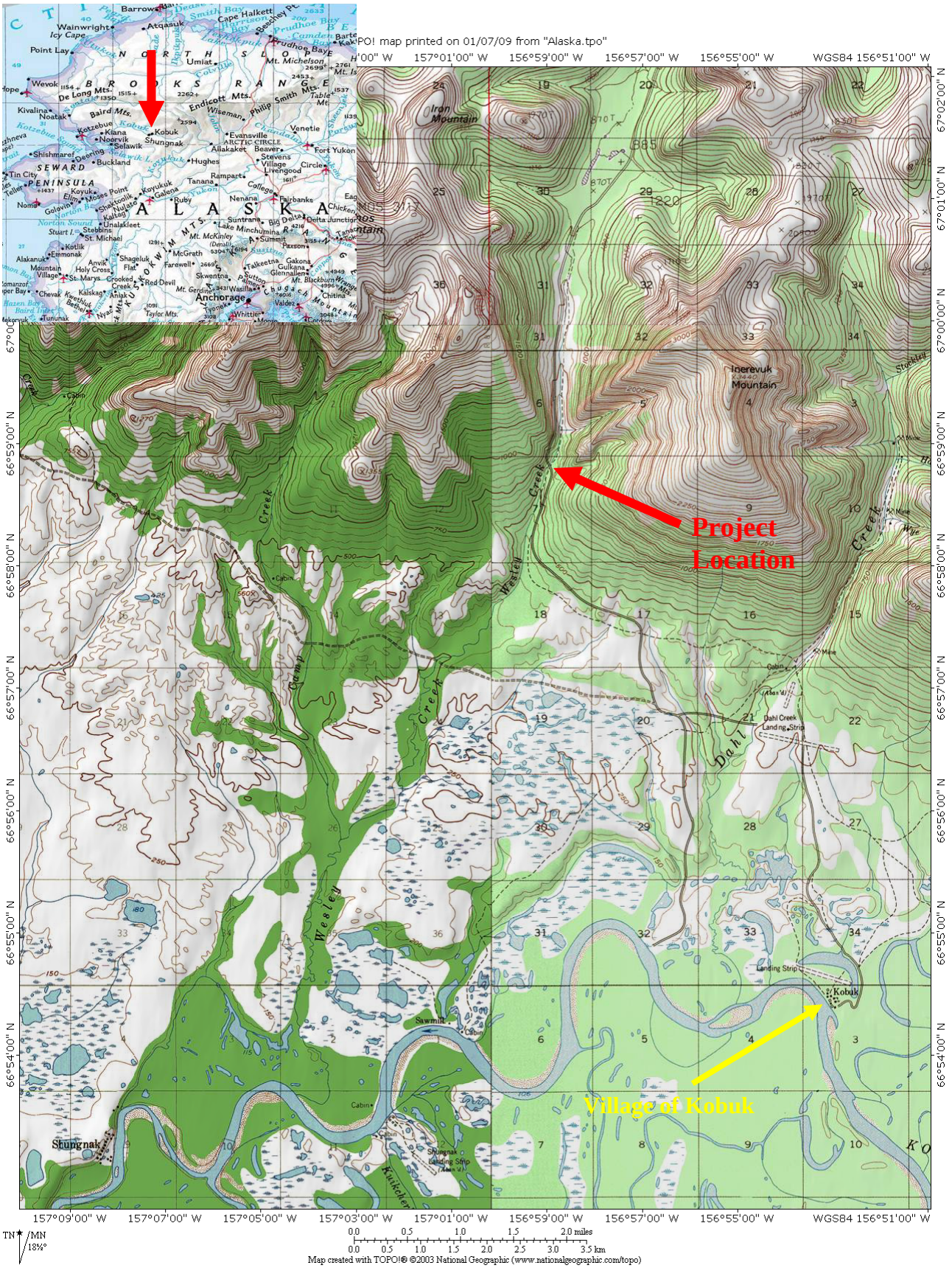

Figure 1 – Vicinity Map and Project Location

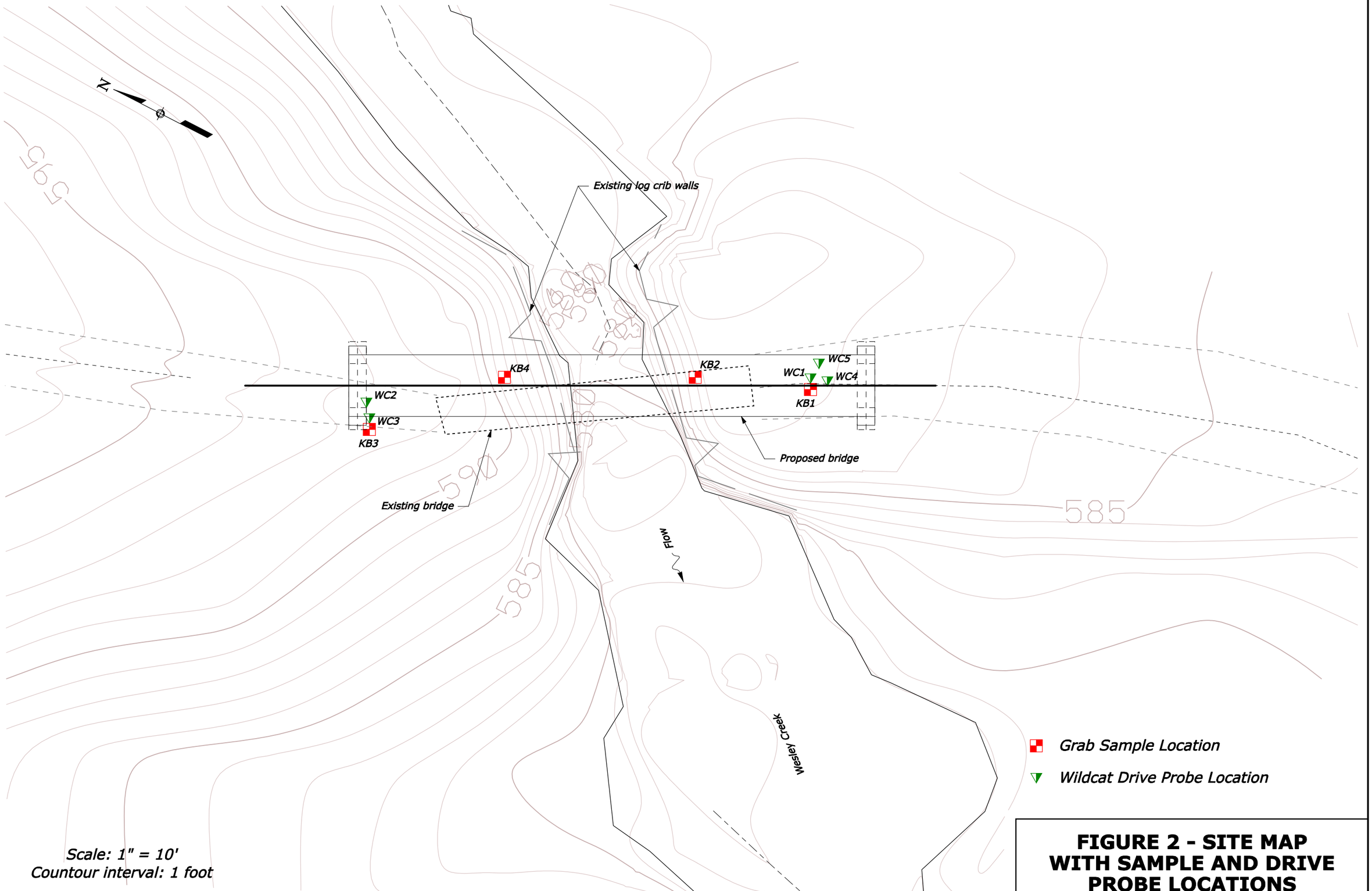

FIGURE 2 - SITE MAP WITH SAMPLE AND DRIVE PROBE LOCATIONS

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APPENDIX A

Site Photographs

In-water stream crossing just downstream of bridge location (left) and bridge approach road, looking north

Approach to bridge, looking north

Existing timber bridge, looking north

End of bridge and approach road, looking north

Existing bridge, looking upstream

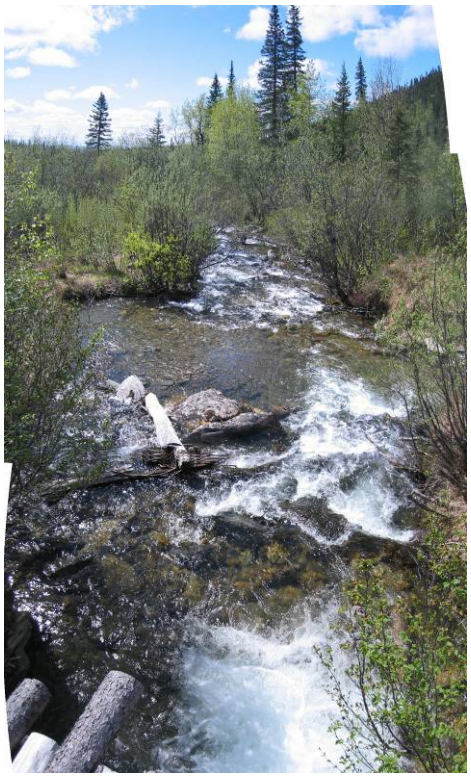
North abutment, looking upstream

South abutment, looking upstream

Existing bridge looking downstream

South abutment

North abutment



Looking downstream from bridge

Looking upstream from bridge

North abutment

Sample location KB2-GS, south abutment fill slope beneath existing bridge

Sample location KB3-HA, north abutment roadway, hand auger hole

Aerial photo looking north from near Kobuk airstrip, project location is in the valley in the upper center/left of photograph just below aircraft wing

Aerial photo of Kobuk Village and Kobuk River, looking south

Village of Kobuk

APPENDIX B

B-1 – Unified Soil Classification System
B-2 – Material Testing Results

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions			Group Symbols (†)	Typical Names	Field Identification Procedures (excluding particles larger than 75 mm and basing fractions on estimated weights)		
1	2		3	4	5		
Coarse grained Soils: More than half of material is larger than No. 200 (‡) (75 μm) sieve size. The No. 200 sieve size is about the smallest particle visible to the naked eye.	Gravels: More than half of gravel fraction is larger than No. 4 sieve size. (4.75 mm) (for visual classification, 5 mm may be used as equivalent to the No. 4 sieve size)	Clean Gravels (little or no fines)	GW	Well-graded gravels, gravel sand mixtures, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.		
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.		
		Gravels with Fines (appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).		
			GC	Clayey gravels, gravel-sand-clay mixtures.	Plastic fines (for identification procedures see CL below).		
	Sands: More than half of coarse fraction is smaller than No. 4 sieve size. (4.75 mm)	Clean Sands (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.		
			SP	Poorly graded sands, gravelly sands, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.		
		Sands with Fines (appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).		
			SC	Clayey sands, sand-clay mixtures.	Plastic fines (for identification procedures see CL below).		
Fine grained Soils: More than half of material is smaller than No. 200 (75 μm) sieve size.					Identification Procedures on Fraction Smaller than No. 40 Sieve Size		
					Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near PL)
	Silts and Clays: Liquid limit less than 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.	None to slight	Quick to slow	None
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Medium to high	None to very slow	Medium
			OL	Organic silts and organic silty clays of low plasticity.	Slight to medium	Slow	Slight
	Silts and Clays: Liquid limit greater than 50		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Slight to medium	Slow to none	Slight to medium
			CH	Inorganic clays of high plasticity, fat clays.	High to very high	None	High
			OH	Organic clays of medium to high plasticity, organic silts.	Medium to high	None to very slow	Slight to medium
Highly Organic Soils			Pt	Peat and other highly organic soils.	Readily identified by color, odor, spongy feel, and frequently by fibrous texture.		

Laboratory Classification Criteria (6)

Use grain size curve in identifying the fractions as given under field identification.

Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows:

- Less than 5%: GW, GP, SW, SP
- More than 12%: GM, GC, SM, SC.
- 5% to 12%: Borderline cases requiring use of dual symbols.

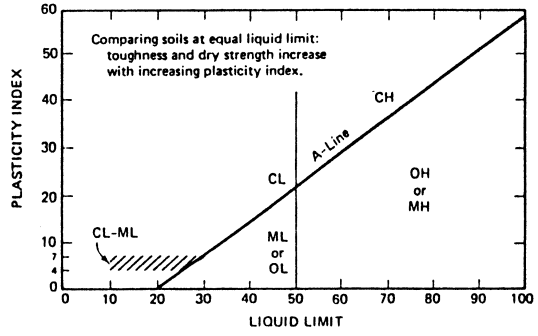
GW: $C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3 (See Sec. 2-5)

- Not meeting all gradation requirements for GW
- Atterberg limits below A-line, or PI less than 4
- Atterberg limits above A-line with PI greater than 7
- Above A-line with PI between 4 and 7 are borderline cases requiring use of dual symbols.

SW: $C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3 (See Sec. 2-5)

- Not meeting all gradation requirements for SW
- Atterberg limits below A-line, or PI less than 4
- Atterberg limits above A-line with PI greater than 7
- Limits plotting in hatched zone with PI between 4 and 7 are borderline cases requiring use of dual symbols.

Plasticity Chart
For laboratory classification of fine-grained soils



† Boundary classifications: soils possessing characteristics of two groups are designated by combinations of group symbols. For example: GW-GC, well-graded gravel sand mixture with clay binder.

‡ All sieve sizes on this chart are U.S. Standard.

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

Test Report Issued: **23 Jun 2008**
Lab Control Number: **W-08-0420-SO**

Project Name: Kobuk Bridge
Project Number: AK DEN 2007(1)
Acct. No.: 1517020200701 R10.D0.15F0.02

Sample No: KB1-HA
Sampled By: Nathan Jenks
Date Sampled: 6/11/2008

Submitted By: Nathan Jenks
Phone: (360)619-7748
Fax: (360)619-7845

Address: WFLHD, Geotechnical Branch
610 East Fifth Street
Vancouver, WA 98661

Sample of: embankment fill
Quantity Rep:

Date Received: 6/18/2008
No. & Containers: 1 bag
Dates Tested: 6/18/08 - 6/21/08

Source Name: abutment 1 - roadway
Owner:
Boring No./Test Pit: KB1-HA
Sample Location: 7' back from bridge

County: NW Artic **State:** AK
Depth: 0-10"

Sieve Analysis

Sieve Size	As Received % Passing
2"	100.0
1 1/2"	97.2
1"	86.2
3/4"	79.8
1/2"	70.7
3/8"	64.1
#4	48.8
#10	37.2
#40	22.5
#200	12.9
20µm	7.1

Soil Classification (DL145)

AASHTO	A-1-a(0)	SI-SA-GRAVEL
Unified	GM; Silty gravel with sand	

Apparent Specific Gravity (T100) 2.744

Natural Moisture (T265) (Sample dried at 140 °F), % 6.00

Atterberg Limits (T89)

Liquid Limit	NP
Plasticity Index	NP

G. R. HACKER, Materials Laboratory Chief
For: JOHN SNYDER, Materials Engineer

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

Test Report Issued: **23 Jun 2008**
Lab Control Number: **W-08-0421-SO**

Project Name: Kobuk Bridge
Project Number: AK DEN 2007(1)
Acct. No.: 1517020200701 R10.D0.15F0.02

Sample No: KB2-GS
Sampled By: Nathan Jenks
Date Sampled: 6/11/2008

Submitted By: Nathan Jenks
Phone: (360)619-7748
Fax: (360)619-7845

Address: WFLHD, Geotechnical Branch
610 East Fifth Street
Vancouver, WA 98661

Sample of: embankment fill
Quantity Rep:

Date Received: 6/13/2008
No. & Containers: 1 bag
Dates Tested: 6/18/08 - 6/21/08

Source Name: abutment 1 - fill slope
Owner:
Boring No./Test Pit: KB2-GS
Sample Location: 3' below rd level

County: NW Artic **State:** AK
Depth: 6-12"

Sieve Analysis

Sieve Size	As Received % Passing
2"	100.0
1 1/2"	94.1
1"	83.2
3/4"	68.7
1/2"	60.9
3/8"	54.9
#4	40.2
#10	33.3
#40	24.3
#200	15.9
20µm	9.2

Soil Classification (DL145)

AASHTO	A-1-b(0)	SI-SA-GRAVEL
Unified	GM; Silty gravel with sand	

Apparent Specific Gravity (T100) 2.678

Natural Moisture (T265) (Sample dried at 140 °F), % 12.6

Atterberg Limits (T89)

Liquid Limit	NP
Plasticity Index	NP

G. R. HACKER, Materials Laboratory Chief
For: JOHN SNYDER, Materials Engineer

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

Test Report Issued: **23 Jun 2008**
Lab Control Number: **W-08-0422-SO**

Project Name: Kobuk Bridge
Project Number: AK DEN 2007(1)
Acct. No.: 1517020200701 R10.D0.15F0.02

Sample No: KB3-HA
Sampled By: Nathan Jenks
Date Sampled: 6/11/2008

Submitted By: Nathan Jenks
Phone: (360)619-7748
Fax: (360)619-7845

Address: WFLHD, Geotechnical Branch
610 East Fifth Street
Vancouver, WA 98661

Sample of: embankment fill
Quantity Rep:

Date Received: 6/13/2008
No. & Containers: 1 bag
Dates Tested: 6/18/08 - 6/21/08

Source Name: abutment 2 - roadway
Owner:
Boring No./Test Pit: KB3-HA
Sample Location: 8.5' from end of bridge

County: NW Artic **State:** AK
Depth: 0-9"

Sieve Analysis

Sieve Size	As Received % Passing
1 1/2"	100.0
1"	80.5
3/4"	60.8
1/2"	48.9
3/8"	42.8
#4	31.8
#10	24.4
#40	14.5
#200	7.5
20μm	4.1

Soil Classification (DL145)

AASHTO	A-1-a(0)	SI-SA-GRAVEL
Unified	GP-GM; Poorly graded gravel with silt and sand	

Apparent Specific Gravity (T100) 2.701

Natural Moisture (T265) (Sample dried at 140 °F), % 4.70

Atterberg Limits (T89)

Liquid Limit	NP
Plasticity Index	NP

G. R. HACKER, Materials Laboratory Chief
For: JOHN SNYDER, Materials Engineer

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

Test Report Issued: **23 Jun 2008**
Lab Control Number: **W-08-0423-SO**

Project Name: Kobuk Bridge
Project Number: AK DEN 2007(1)
Acct. No.: 1517020200701 R10.D0.15F0.02

Sample No: KB4-GS
Sampled By: Nathan Jenks
Date Sampled: 6/11/2008

Submitted By: Nathan Jenks
Phone: (360)619-7748
Fax: (360)619-7845

Address: WFLHD, Geotechnical Branch
610 East Fifth Street
Vancouver, WA 98661

Sample of: embankment fill
Quantity Rep:

Date Received: 6/13/2008
No. & Containers: 1 bag
Dates Tested: 6/18/08 - 6/21/08

Source Name: abutment 2 - fill slope
Owner:
Boring No./Test Pit: KB4-GS
Sample Location: 4' below rd level

County: NW Artic **State:** AK
Depth: 4-12"

Sieve Analysis

Sieve Size	As Received % Passing
3"	100.0
2"	93.9
1 1/2"	88.5
1"	78.1
3/4"	65.6
1/2"	55.5
3/8"	51.9
#4	42.8
#10	36.1
#40	27.2
#200	20.2
20µm	12.6

Soil Classification (DL145)

AASHTO	A-1-b(0)	SA-SI-GRAVEL
Unified	GM; Silty gravel with sand	

Apparent Specific Gravity (T100) 2.709

Natural Moisture (T265) (Sample dried at 140 °F), % 9.50

Atterberg Limits (T89)

Liquid Limit	NP
Plasticity Index	NP

G. R. HACKER, Materials Laboratory Chief
For: JOHN SNYDER, Materials Engineer

APPENDIX C

Wildcat Penetrometer Results

WILDCAT DYNAMIC CONE PENETROMETER ANALYSIS

Project : Kobuk Bridge
Project Location: North Alaska
Boring Number: **WCH1**
Boring Location: South abutment - 7' Back from Start of Bridge, Center of Rd
Drill Dates : 6/11/2008
Drilled By: N Jenks
Logged By: N Jenks

*Dynamic cone resistance equation

$$q_d = \frac{M^2 \cdot H \cdot N_d}{A_p\left(M + M' + P_a\right)10}$$

M = Hammer mass = 15.89 Kg, H = Drop Height = 38.1 cm

N_d = Blows / 10 cm, A_p = Projected cone area = 10 cm^2

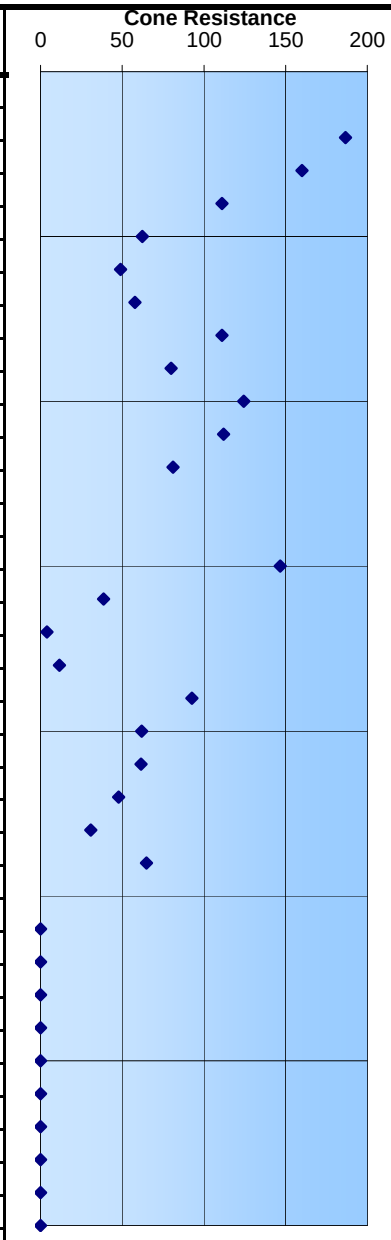
M' = Mass driven portion of hammer = 2.49 Kg

P_a = Mass of rod string = 3.26 Kg · number of rods

**Correlated SPT N Value = $N' = \frac{q_d}{3.5}$

Equations based on manufacturer's recommendations, resistance (q_d) higher than 90 have indeterminate SPT correlation values

DEPTH (m) Strt	end	D (ft)	BLOWS per 10 cm	RESIST* Kg/cm²	N'**	DENSITY/CONSIST SAND & SILT	CLAY	COMMENTS
0.0	0.1	0.3						
0.1	0.2	0.7	42	186.7	53	Very Dense	Hard	
0.2	0.3	1.0	36	160.0	46	Dense	Hard	
0.3	0.4	1.3	25	111.1	32	Dense	Hard	
0.4	0.5	1.6	14	62.2	18	Med. Dense	V. Stiff	
0.5	0.6	2.0	11	48.9	14	Med. Dense	Stiff	
0.6	0.7	2.3	13	57.8	17	Med. Dense	V. Stiff	
0.7	0.8	2.6	25	111.1	32	Dense	Hard	
0.8	0.9	3.0	18	80.0	23	Med. Dense	V. Stiff	
0.9	1.0	3.3	28	124.5	36	Dense	Hard	
1.0	1.1	3.6	29	112.0	32	Dense	Hard	
1.1	1.2	3.9	21	81.1	23	Med. Dense	V. Stiff	
1.2	1.3	4.3	62	239.5	68	Very Dense	Hard	
1.3	1.4	4.6	85	328.4	94	Very Dense	Hard	
1.4	1.5	4.9	38	146.8	42	Dense	Hard	
1.5	1.6	5.2	10	38.6	11	Med. Dense	Stiff	
1.6	1.7	5.6	1	3.9	1	Very Loose	V. Soft	void - rod dropped from
1.7	1.8	5.9	3	11.6	3	Very Loose	Soft	~68 to 71 inches
1.8	1.9	6.2	24	92.7	26	Dense	V. Stiff	
1.9	2.0	6.6	16	61.8	18	Med. Dense	V. Stiff	
2.0	2.1	6.9	18	61.5	18	Med. Dense	V. Stiff	
2.1	2.2	7.2	14	47.8	14	Med. Dense	Stiff	
2.2	2.3	7.5	9	30.7	9	Loose	Firm	
2.3	2.4	7.9	19	64.9	19	Med. Dense	V. Stiff	
2.4	2.5	8.2	100	341.6	98	Very Dense	Hard	Refusal - 100 blows for
2.5	2.6	8.5		0.0	0	---	---	3 inches, TD 8.1'
2.6	2.7	8.9		0.0	0	---	---	
2.7	2.8	9.2		0.0	0	---	---	
2.8	2.9	9.5		0.0	0	---	---	
2.9	3.0	9.8		0.0	0	---	---	
3.0	3.1	10.2		0.0	0	---	---	
3.1	3.2	10.5		0.0	0	---	---	
3.2	3.3	10.8		0.0	0	---	---	
3.3	3.4	11.2		0.0	0	---	---	
3.4	3.5	11.5		0.0				



Western Federal Lands Highway Division

WILDCAT DYNAMIC CONE PENETROMETER ANALYSIS

Project : Kobuk Bridge
Project Location: North Alaska
Boring Number: **WCH2**
Boring Location: North abutment, 8 feet back from bridge, center of road
Drill Dates : 6/11/2008
Drilled By: N Jenks
Logged By: N Jenks

*Dynamic cone resistance equation

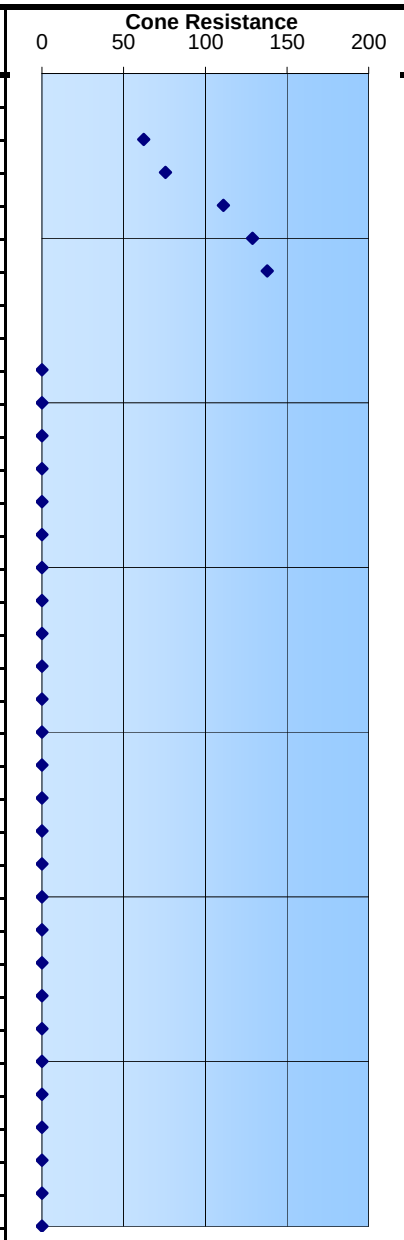
$$q_d = \frac{M^2 \cdot H \cdot N_d}{A_p\left(M + M' + P_a\right)10}$$

M = Hammer mass = 15.89 Kg, H = Drop Height = 38.1cm
N_d = Blows /10cm, A_p = Projected cone area = 10cm²
M' = Mass driven portion of hammer = 2.49 Kg
P_a = Mass of rod string = 3.26 Kg · number of rods

**Correlated SPT N Value = $N' = \frac{q_d}{3.5}$

Equations based on manufacturer's recommendations, resistance (q_d) higher than 90 have indeterminate SPT correlation values

DEPTH (m) Strt	end	D (ft)	BLOWS per 10 cm	RESIST* Kg/cm²	N'**	DENSITY/CONSIST SAND & SILT	CLAY	COMMENTS
0.0	0.1	0.3						
0.1	0.2	0.7	14	62.2	18	Med. Dense	V. Stiff	
0.2	0.3	1.0	17	75.6	22	Med. Dense	V. Stiff	
0.3	0.4	1.3	25	111.1	32	Dense	Hard	
0.4	0.5	1.6	29	128.9	37	Dense	Hard	
0.5	0.6	2.0	31	137.8	39	Dense	Hard	
0.6	0.7	2.3	56	248.9	71	Very Dense	Hard	
0.7	0.8	2.6	50	222.3	64	Very Dense	Hard	Refusal - 50 blows for
0.8	0.9	3.0		0.0	0	---	---	< 1 inch, TD 2.4'
0.9	1.0	3.3		0.0	0	---	---	
1.0	1.1	3.6		0.0	0	---	---	
1.1	1.2	3.9		0.0	0	---	---	
1.2	1.3	4.3		0.0	0	---	---	
1.3	1.4	4.6		0.0	0	---	---	
1.4	1.5	4.9		0.0	0	---	---	
1.5	1.6	5.2		0.0	0	---	---	
1.6	1.7	5.6		0.0	0	---	---	
1.7	1.8	5.9		0.0	0	---	---	
1.8	1.9	6.2		0.0	0	---	---	
1.9	2.0	6.6		0.0	0	---	---	
2.0	2.1	6.9		0.0	0	---	---	
2.1	2.2	7.2		0.0	0	---	---	
2.2	2.3	7.5		0.0	0	---	---	
2.3	2.4	7.9		0.0	0	---	---	
2.4	2.5	8.2		0.0	0	---	---	
2.5	2.6	8.5		0.0	0	---	---	
2.6	2.7	8.9		0.0	0	---	---	
2.7	2.8	9.2		0.0	0	---	---	
2.8	2.9	9.5		0.0	0	---	---	
2.9	3.0	9.8		0.0	0	---	---	
3.0	3.1	10.2		0.0	0	---	---	
3.1	3.2	10.5		0.0	0	---	---	
3.2	3.3	10.8		0.0	0	---	---	
3.3	3.4	11.2		0.0	0	---	---	
3.4	3.5	11.5		0.0				



Western Federal Lands Highway Division

WILDCAT DYNAMIC CONE PENETROMETER ANALYSIS

Project : Kobuk Bridge
Project Location: North Alaska
Boring Number: **WCH3**
Boring Location: North abutment, 8 feet back from bridge, LT side
Drill Dates : 6/11/2008
Drilled By: N Jenks
Logged By: N Jenks

*Dynamic cone resistance equation

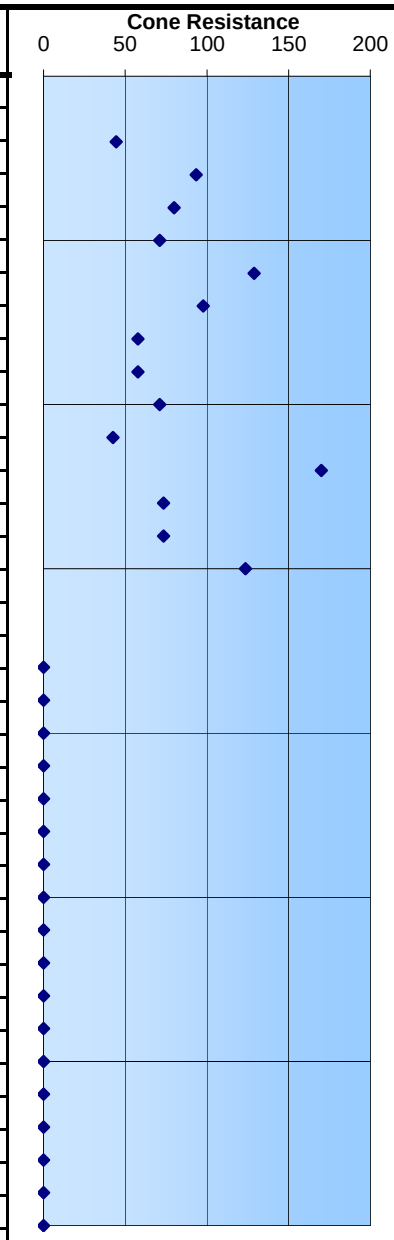
$$q_d = \frac{M^2 \cdot H \cdot N_d}{A_p\left(M + M' + P_a\right)10}$$

M = Hammer mass = 15.89 Kg, H = Drop Height = 38.1cm
N_d = Blows /10cm, A_p = Projected cone area = 10cm²
M' = Mass driven portion of hammer = 2.49 Kg
P_a = Mass of rod string = 3.26 Kg · number of rods

**Correlated SPT N Value = $N' = \frac{q_d}{3.5}$

Equations based on manufacturer's recommendations, resistance (q_d) higher than 90 have indeterminate SPT correlation values

DEPTH (m) Strt	end	D (ft)	BLOWS per 10 cm	RESIST* Kg/cm²	N'**	DENSITY/CONSIST SAND & SILT	CLAY	COMMENTS
0.0	0.1	0.3						
0.1	0.2	0.7	10	44.5	13	Med. Dense	Stiff	
0.2	0.3	1.0	21	93.4	27	Dense	V. Stiff	
0.3	0.4	1.3	18	80.0	23	Med. Dense	V. Stiff	
0.4	0.5	1.6	16	71.1	20	Med. Dense	V. Stiff	
0.5	0.6	2.0	29	128.9	37	Dense	Hard	
0.6	0.7	2.3	22	97.8	28	Dense	V. Stiff	
0.7	0.8	2.6	13	57.8	17	Med. Dense	V. Stiff	
0.8	0.9	3.0	13	57.8	17	Med. Dense	V. Stiff	
0.9	1.0	3.3	16	71.1	20	Med. Dense	V. Stiff	
1.0	1.1	3.6	11	42.5	12	Med. Dense	Stiff	
1.1	1.2	3.9	44	170.0	49	Dense	Hard	
1.2	1.3	4.3	19	73.4	21	Med. Dense	V. Stiff	
1.3	1.4	4.6	19	73.4	21	Med. Dense	V. Stiff	
1.4	1.5	4.9	32	123.6	35	Dense	Hard	
1.5	1.6	5.2	61	235.7	67	Very Dense	Hard	
1.6	1.7	5.6	100	386.3	110	Very Dense	Hard	Refusal - 50 blows for < 1 inch, TD 5.3'
1.7	1.8	5.9		0.0	0	---	---	
1.8	1.9	6.2		0.0	0	---	---	
1.9	2.0	6.6		0.0	0	---	---	
2.0	2.1	6.9		0.0	0	---	---	
2.1	2.2	7.2		0.0	0	---	---	
2.2	2.3	7.5		0.0	0	---	---	
2.3	2.4	7.9		0.0	0	---	---	
2.4	2.5	8.2		0.0	0	---	---	
2.5	2.6	8.5		0.0	0	---	---	
2.6	2.7	8.9		0.0	0	---	---	
2.7	2.8	9.2		0.0	0	---	---	
2.8	2.9	9.5		0.0	0	---	---	
2.9	3.0	9.8		0.0	0	---	---	
3.0	3.1	10.2		0.0	0	---	---	
3.1	3.2	10.5		0.0	0	---	---	
3.2	3.3	10.8		0.0	0	---	---	
3.3	3.4	11.2		0.0	0	---	---	
3.4	3.5	11.5		0.0				



Western Federal Lands Highway Division

WILDCAT DYNAMIC CONE PENETROMETER ANALYSIS

Project : Kobuk Bridge
Project Location: North Alaska
Boring Number: **WCH4**
Boring Location: South abutment, 9 feet back from bridge, CL of road
Drill Dates : 6/11/2008
Drilled By: N Jenks
Logged By: N Jenks

*Dynamic cone resistance equation

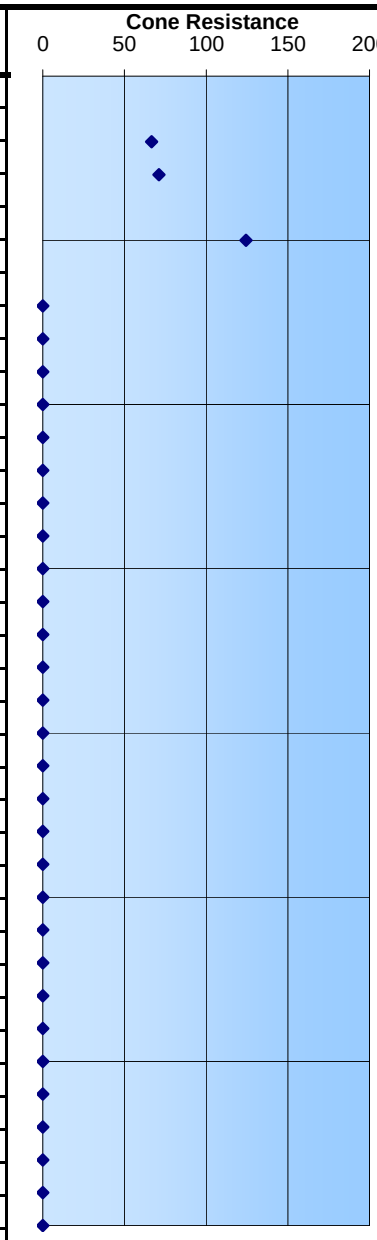
$$q_d = \frac{M^2 \cdot H \cdot N_d}{A_p\left(M + M' + P_a\right)10}$$

M = Hammer mass = 15.89 Kg, H = Drop Height = 38.1cm
N_d = Blows /10cm, A_p = Projected cone area = 10cm²
M' = Mass driven portion of hammer = 2.49 Kg
P_a = Mass of rod string = 3.26 Kg · number of rods

**Correlated SPT N Value = $N' = \frac{q_d}{3.5}$

Equations based on manufacturer's recommendations, resistance (q_d) higher than 90 have indeterminate SPT correlation values

DEPTH (m) Strt	end	D (ft)	BLOWS per 10 cm	RESIST* Kg/cm²	N'**	DENSITY/CONSIST SAND & SILT	CLAY	COMMENTS
0.0	0.1	0.3						
0.1	0.2	0.7	15	66.7	19	Med. Dense	V. Stiff	
0.2	0.3	1.0	16	71.1	20	Med. Dense	V. Stiff	
0.3	0.4	1.3	48	213.4	61	Very Dense	Hard	
0.4	0.5	1.6	28	124.5	36	Dense	Hard	
0.5	0.6	2.0	60	266.7	76	Very Dense	Hard	Refusal - 60 blows for
0.6	0.7	2.3		0.0	0	---	---	< 1 inch, TD 1.7'
0.7	0.8	2.6		0.0	0	---	---	
0.8	0.9	3.0		0.0	0	---	---	
0.9	1.0	3.3		0.0	0	---	---	
1.0	1.1	3.6		0.0	0	---	---	
1.1	1.2	3.9		0.0	0	---	---	
1.2	1.3	4.3		0.0	0	---	---	
1.3	1.4	4.6		0.0	0	---	---	
1.4	1.5	4.9		0.0	0	---	---	
1.5	1.6	5.2		0.0	0	---	---	
1.6	1.7	5.6		0.0	0	---	---	
1.7	1.8	5.9		0.0	0	---	---	
1.8	1.9	6.2		0.0	0	---	---	
1.9	2.0	6.6		0.0	0	---	---	
2.0	2.1	6.9		0.0	0	---	---	
2.1	2.2	7.2		0.0	0	---	---	
2.2	2.3	7.5		0.0	0	---	---	
2.3	2.4	7.9		0.0	0	---	---	
2.4	2.5	8.2		0.0	0	---	---	
2.5	2.6	8.5		0.0	0	---	---	
2.6	2.7	8.9		0.0	0	---	---	
2.7	2.8	9.2		0.0	0	---	---	
2.8	2.9	9.5		0.0	0	---	---	
2.9	3.0	9.8		0.0	0	---	---	
3.0	3.1	10.2		0.0	0	---	---	
3.1	3.2	10.5		0.0	0	---	---	
3.2	3.3	10.8		0.0	0	---	---	
3.3	3.4	11.2		0.0	0	---	---	
3.4	3.5	11.5		0.0				



Western Federal Lands Highway Division

WILDCAT DYNAMIC CONE PENETROMETER ANALYSIS

Project : Kobuk Bridge
Project Location: North Alaska
Boring Number: **WCH5**
Boring Location: South abutment, 8 feet back from start of bridge RT side
Drill Dates : 6/11/2008
Drilled By: N Jenks
Logged By: N Jenks

*Dynamic cone resistance equation

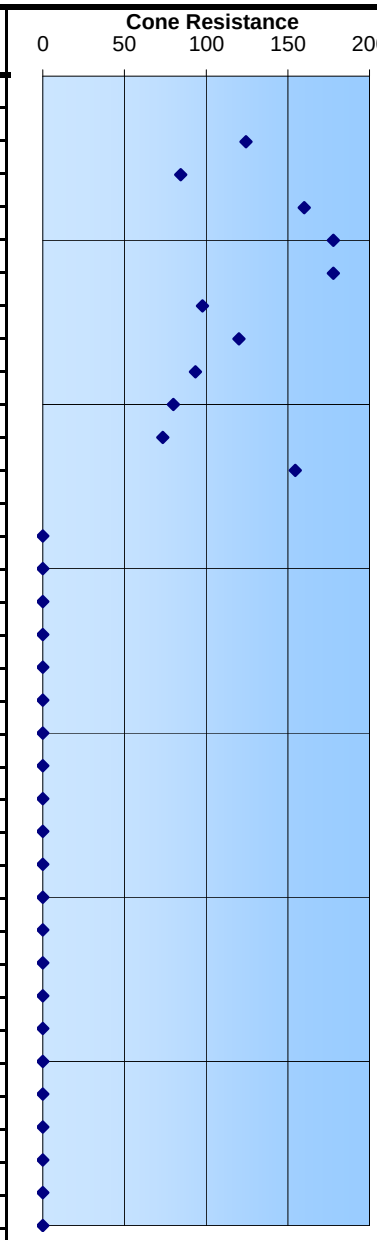
$$q_d = \frac{M^2 \cdot H \cdot N_d}{A_p\left(M + M' + P_a\right)10}$$

M = Hammer mass = 15.89 Kg, H = Drop Height = 38.1cm
N_d = Blows /10cm, A_p = Projected cone area = 10cm²
M' = Mass driven portion of hammer = 2.49 Kg
P_a = Mass of rod string = 3.26 Kg · number of rods

**Correlated SPT N Value = $N' = \frac{q_d}{3.5}$

Equations based on manufacturer's recommendations, resistance (q_d) higher than 90 have indeterminate SPT correlation values

DEPTH (m) Strt	end	D (ft)	BLOWS per 10 cm	RESIST* Kg/cm²	N'**	DENSITY/CONSIST SAND & SILT	CLAY	COMMENTS
0.0	0.1	0.3						
0.1	0.2	0.7	28	124.5	36	Dense	Hard	
0.2	0.3	1.0	19	84.5	24	Med. Dense	V. Stiff	
0.3	0.4	1.3	36	160.0	46	Dense	Hard	
0.4	0.5	1.6	40	177.8	51	Very Dense	Hard	
0.5	0.6	2.0	40	177.8	51	Very Dense	Hard	
0.6	0.7	2.3	22	97.8	28	Dense	V. Stiff	
0.7	0.8	2.6	27	120.0	34	Dense	Hard	
0.8	0.9	3.0	21	93.4	27	Dense	V. Stiff	
0.9	1.0	3.3	18	80.0	23	Med. Dense	V. Stiff	
1.0	1.1	3.6	19	73.4	21	Med. Dense	V. Stiff	
1.1	1.2	3.9	40	154.5	44	Dense	Hard	
1.2	1.3	4.3	60	231.8	66	Very Dense	Hard	Refusal - 60 blows for <2 inches, TD 4.4'
1.3	1.4	4.6		0.0	0	---	---	
1.4	1.5	4.9		0.0	0	---	---	
1.5	1.6	5.2		0.0	0	---	---	
1.6	1.7	5.6		0.0	0	---	---	
1.7	1.8	5.9		0.0	0	---	---	
1.8	1.9	6.2		0.0	0	---	---	
1.9	2.0	6.6		0.0	0	---	---	
2.0	2.1	6.9		0.0	0	---	---	
2.1	2.2	7.2		0.0	0	---	---	
2.2	2.3	7.5		0.0	0	---	---	
2.3	2.4	7.9		0.0	0	---	---	
2.4	2.5	8.2		0.0	0	---	---	
2.5	2.6	8.5		0.0	0	---	---	
2.6	2.7	8.9		0.0	0	---	---	
2.7	2.8	9.2		0.0	0	---	---	
2.8	2.9	9.5		0.0	0	---	---	
2.9	3.0	9.8		0.0	0	---	---	
3.0	3.1	10.2		0.0	0	---	---	
3.1	3.2	10.5		0.0	0	---	---	
3.2	3.3	10.8		0.0	0	---	---	
3.3	3.4	11.2		0.0	0	---	---	
3.4	3.5	11.5		0.0				



Western Federal Lands Highway Division

APPENDIX D

Hydraulics Bridge Design Report

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
WESTERN FEDERAL LANDS HIGHWAY DIVISION
610 east Fifth street
Vancouver Washington, 98661

HYDRAULICS BRIDGE DESIGN REPORT

To: Michael Traffalis, WFLHD Project Manager
From: Sven Leon, P.E., WFLHD Hydraulics Engineer
Date: August 11, 2008
Project: Kobuk Trail Bridge Replacement – AK DEN (1)

Project Description

The Northwest Artic Borough and the Bureau of Land Reclamation (BLM) have determined a log stringer trail bridge crossing the upper Wesley Creek and serving the village of Kobuk, Alaska, is in poor condition and should be replaced. The Federal Highway Administration (FHWA), Western Federal Lands Highway Division (WFLHD) will design the new trail bridge. Hydrologic, geomorphologic, hydraulic, and scour analysis is presented along with recommendations for bridge low chord elevations and scour protection.

The existing log stringer trail bridge is perched on sloped soil abutments. Toe erosion is controlled by log cribbing walls, approximately 15 years old. To reduce project costs, the existing log cribbing walls will be used as toe erosion control for the new bridge abutments.

Conclusions and Recommendations

The following conclusions are made:

- Based on observed conditions and hydraulic modeling, the log cribbing walls can be expected to provide cost-effective erosion control for at least 15 to 20 year service life.
- Scour predicted for flood events greater than the 100-year may undermine the existing log cribbing walls causing them to fail.
- The log cribbing walls will continue to deteriorate and eventually fail.
- Failure of the cribbing walls will allow erosion to eventually undermine the perched bridge footings.
- Flow alignment will result in lateral stream bank migration towards the southeast abutment.

The following recommendations are made:

- To reduce project costs, the existing log cribbing walls may be used for the new trail bridge.
- Minimum bridge low chord elevation should be 588 feet (project datum) for providing hydraulic flow and debris conveyance.

- Minimum bridge length should be 60 feet for accommodating 1.75(h):1(v) sloped abutments and reducing the risk of the bridge footings from being undermined by erosion.
- To offset expected lateral stream bank migration, set the southeast end at least 20 feet from the southeast end of the existing bridge.
- Monitor the log cribbing walls for structurally threatening deterioration and scour.
- Before the log cribbing walls fail, replace them with appropriately designed riprap revetments.

Site Conditions

The existing vertical log cribbing extends approximately 6 feet above the channel bottom and constricts the floodway to approximately 12 feet. No evidence of floodwater overtopping and eroding the sloped embankments was observed. The bridge crossing is skewed approximately 10 degrees relative to the stream. Channel alignment results in flow impingement on the southeast abutment. Minor scour under limited sections of both walls was observed. No evidence of the walls being immediately undermined and failing was observed. The log cribbing shows light deterioration, but is in generally serviceable condition.

Average stream gradient is approximately 3 percent. Bankfull width is approximately 15 to 20 feet. Bankfull depth is approximately 2.5 feet. The stream is confined in a moderately wide glacial-fluvial valley with flood prone area widths between 40 and 60 feet. Depositional soils appear to be generally glacial-fluvial. The stream is perennial. Spring and storm flow are the dominant stream-flow control.

Riffles are the dominant channel bed morphology. Sediment observed both upstream and downstream of the crossing appears to be predominantly cobble and gravel. The d5 (particle size with 50 percent smaller) is estimated to be 3 inches. The largest particle size observed was 12 inches. Depositional features include cobble clusters and transverse ribs. Bank materials appear predominantly cobble and gravel. Floodplains are poorly developed. Riparian vegetation is predominantly dense underbrush and small spruce and poplar trees. No bedrock was observed in the stream banks or bottom near the crossing.

The small supply of large woody debris available to the stream suggests a low potential of the stream transporting large woody debris. The banks are generally well vegetated and appear stable. Stable banks and cobble depositional features suggest a moderate sediment supply. Based on the generally stable banks, the stream does not appear to be rapidly degrading or aggrading.

Peak Discharge Estimates

The drainage area was determined using a U.S.G.S. quadrangle map to be approximately 9.6 square miles. The drainage area is mountainous, with elevations from 500 to 2,000 feet (msl). The drainage area is thickly timbered with small spruce and poplar. According to mapping in U.S.G.S. Report 03-4188 mean annual rainfall is approximately 25 inches. Peak flood discharges at the crossing site were estimated using drainage-area based statewide regression equations (U.S.G.S. Report 03-4188, 2003). The results are presented in Table 1.

Water Surface Elevations and Flow Velocities

Water surface elevations and flow velocities were estimated using the Hydrologic Engineering Center River Analysis System (HEC-RAS), a computer program that performs one-dimensional steady flow calculations (see attached calculations). Cross sections across the channel and flood plain parallel to expected flow direction were generated using ground topographic survey data. Manning's roughness coefficients for the channel and flood plain were estimated using guidelines in FHWA-TS-84-204. Proposed bridge geometry based on July 8, 2008 TSL. Flow was assumed subcritical. The energy slope at the downstream station was assumed to equal the water surface slope. The results are presented in Table 1.

Scour

A HEC-18 based scour analysis was performed. Results are presented in Table1. Stream bed sediment particle size was visually estimated from channel deposits immediately downstream and upstream of the crossing. Bedrock was not observed.

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Table 1. Peak Discharge Estimates and Hydraulic Model Results

	Recurrence Interval						
	2-Year			**25-Year**	**50-Year**	**100-Year**	**500-Year**
Peak Discharge (ft3/s)	212			537	617	695	875
Flow Velocity (ft/sec)	8.7			11.6	12.0	12.4	13.3
Flow Depth (ft)	3.8			5.8	6.2	6.6	7.5
Water Surface Elev.(1)(ft)	583.6			585.8	586.2	586.6	587.5
Total Scour (ft)	NA			NA	NA	1.3	1.8

(1) Water surface at upstream face of proposed bridge, assumed channel elevation 580.1 feet (msl).

(2) Not analyzed.