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January 25, 2019

Jewell Associates Engineers, Inc
560 Sunrise Drive
Spring Green, WI 53588

NTS Project No. 17506_SBR

Attention: **Patrick Boland, PE**
patrick.boland@jewellassoc.com

Subject: **Subsurface Soil Investigation Report**
Creek Road Bridge and Approaches
(WSOR Crossing)
Project ID 3614-00-05
Town of Bradford, Rock County, WI

As requested, Nummelin Testing Services, Inc. has conducted a Geotechnical Engineering Subsurface Exploration and Report for the above-named project. We enclose our report "Subsurface Soil Investigation, Creek Road Bridge and Approaches (WSOR Crossing), Project ID 3614-00-05, Town of Bradford, Rock County, WI – NTS 175.06" which discusses our conclusions and recommendations.

If additional information or clarification is needed, or if we may be of further service during the construction phase of the project, please do not hesitate to contact our office.

The soil samples will be discarded after April 1, 2019.

Respectfully,

Benjamin K. Nummelin

Benjamin K. Nummelin, P.E.
NUMMELIN TESTING SERVICES, INC.
bkn/mn

encl. report & logs
abandonment forms
location map
winSTABL input/output

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SUBSURFACE SOIL INVESTIGATION

CREEK ROAD BRIDGE AND APPROACHES
(WSOR CROSSING)
TOWN OF BRADFORD
ROCK COUNTY
WISCONSIN

PROJECT ID 3614-00-05

NTS 175.06

PREPARED FOR:

JEWELL ASSOCIATES ENGINEERS, INC
560 SUNRISE DRIVE
SPRING GREEN, WI 53588

ATTENTION: PATRICK BOLAND, PE

FIELD INVESTIGATION BY:

NUMMELIN TESTING SERVICES, INC.
STEVENS POINT/WAUNAKEE, WI

JANUARY 25, 2019

SUBSURFACE SOIL INVESTIGATION

CREEK ROAD BRIDGE AND APPROACHES
(WSOR CROSSING)
TOWN OF BRADFORD
ROCK COUNTY
WISCONSIN

PROJECT ID 3614-00-05

1. INTRODUCTION

Nummelin Testing Services, Inc. (NTS) performed this investigation to provide design information for the proposed replacement of the bridge and approaches on Creek Road over the Wisconsin and Southern Railroad (WSOR), approximately 300 feet northeast of Hofstrom Road, in the Town of Bradford, Rock County, Wisconsin. The results and recommendations reported are based upon information obtained during a field investigation with soil borings, and the geotechnical analysis of that information.

The conclusions and recommendations reported are based on our interpretation of available subsurface and project information. The report may not represent variations that occur between or away from the boring locations.

Should the scope of this project be altered, or if subsurface variations become evident during construction, it may be necessary to modify our recommendations. See the attached Geotechnical Engineering Report Information Sheet for general information on NTS's geotechnical reports.

2. PROJECT DESCRIPTION

The proposed project is the replacement of the bridge and approaches on Creek Road over WSOR.

The new bridge is expected to be a single-span prestressed girder bridge supported by either driven piling or footings bearing below the maximum frost depth. The approaches and abutment fill are expected to be retained by Mechanically Stabilized Earth (MSE) Walls, which have an exposed wall height of up to 21 feet. The horizontal and vertical alignments of the new bridge are expected to nearly match that of the previous bridge.

At the time of the investigation, the site was that of the existing bridge over WSOR. The approaches were covered with asphaltic concrete pavement.

3. FIELD INVESTIGATION

Two standard penetration borings were performed on January 9 and 14, 2019, at the locations shown on the attached map. Jewell Associates Engineers determined the proposed boring locations and depths. NTS located the borings in the field. Both borings were drilled approximately at the proposed locations. The borings were proposed to be drilled to a depth of 65 feet but were ended at depths of 52.5 and 50.5 feet, where auger refusal occurred on less weathered bedrock.

Representative soil samples were obtained during boring using the Standard Penetration Test (SPT) method according to ASTM Test Procedure D1586 at the depths indicated on the boring logs. Sampling was continuous in the top 20 feet, then at 5-foot intervals to deeper depths. Drilling between interval samples was by the hollow-stem-auger technique. The soils were visually/manually classified by a technician at the time the borings were performed. Soil samples taken from the site have also been examined in the laboratory by this writer for verification of descriptions which appear on the logs and to classify the soils according to the USCS and AASHTO classification system. No lab testing has been performed.

An automatic-trip hammer that is assumed to have an efficiency rating of 80 percent was used to drive the split-spoon sampler. However, the program used by NTS called ‘Driven,’ which uses LRFD acceptable methods to estimate pile skin friction and end bearing, is based on standard penetration values obtained with a 60 percent efficient hammer. To estimate pile skin friction and end bearing resistances for the bridge, the automatic-trip hammer penetration values (N_{80}) have been corrected to the standard 60 percent efficiency (N_{60}) values. These N_{60} standard penetration values are shown on the boring logs.

The ground elevations at the boring locations were determined by NTS. The railroad spike in the power pole just southwest of the bridge was used as a benchmark. According to the site plan, the elevation of the benchmark is 910.78.

After completion of the borings, the boreholes were backfilled to comply with Wisconsin DNR requirements, then topped off with auger cuttings and the pavement patched with cold-mix asphaltic concrete patch.

Copies of the soil boring logs and the location map are appended to this report.

4. SUBSURFACE CONDITIONS

4. 1. Area Geology

The subsoils in this area are mapped as outwash deposits, which typically consist stratified sand and/or stratified sand and gravel. The underlying bedrock is mapped as dolomite with some shale that is present at depths of greater than 100 feet below the average surface terrain near major waterways, such as the Rock River, but at depths of less than 100 feet elsewhere. The online NRCS web soil survey maps the near-surface soils around the bridge as Plano silt loam.

4. 2. Soils at the Boring Locations

A summary of soil conditions encountered in the borings is shown in Table 4.2.

Table 4.2. Summary of soil conditions in the borings.

Boring	*Surface Elevation*	*Asphalt / Sandy Base Thicknesses*	*Very Stiff Clay (Fill)*	*Med Dense to Dense Sand / Gravel*	*Dense to Very Dense Sand / Gravel*	*Highly Weathered Dolomite*
1 (E Abut)	916.2	4.0” / 3.5”	7.5”- 13’	13’- 23.5’*	23.5’- 48’*	48’- 52.5’**
2 (W Abut)	916.8	5.5” / 5.0”	10.5”- 13’	13’- 18.5’*	18.5’- 46.5’*	46.5’- 50.5’**
*Cobbles encountered in this depth range. **Auger refusal at this depth on less weathered dolomite bedrock.						

At the surface, the borings found 4.0 and 5.5 inches of asphaltic concrete over 3.5 and 5.0 inches of sand with some gravel (sandy base course). The base course at both boring locations appeared to be too sandy to meet the current Wisconsin DOT requirements for dense-graded base course. Below the pavement section, the borings found very stiff lean clay (fill) to a depth of 13 feet. The clay fill had varying amounts of sand and several sand layers up to roughly 1.5 feet thick. Apparent native, light brown sand with gravel and cobbles was found below the clay fill in both borings to depths of 48 and 46.5 feet. The sand with gravel was medium-dense to dense to depths of 23.5 and 18.5 feet, then was dense to very dense below. Below the sand with gravel, highly weathered dolomite bedrock was encountered. The dolomite became less weathered with depth, and the borings were ended at depths of 52.5 and 50.5 feet, where auger refusal occurred on less weathered dolomite.

See individual boring logs for more detailed soil descriptions.

4. 3. Water Level and Creek Bed Measurements

The regional water table was not encountered in either boring. However, wet or saturated soils were found as a result of perched water in the abutment fill and at various deeper depths. These moisture conditions should be considered as representative of site conditions at the time of boring only.

5. DISCUSSION AND RECOMMENDATIONS

5. 1. General

Considering the railroad crossing and that dense soils were found below the abutment fill, either a deep or shallow foundation could be considered for bridge support. If a deep foundation is used, either driven H piling or driven cast-in-place piles (CIP) are expected to be the most economical choices for a deep foundation. The dense soils found below the abutment fill are expected to provide adequate support for a shallow foundation and MSE walls, and the weathered dolomite found at depths of 48.5 and 46 feet is expected to provide adequate support for driven piling.

Cobbles were encountered throughout the native soils in both borings, and these cobbles may complicate pile driving. Should 10.75-inch CIP piles be used, a shell thickness of at least 0.500 inches is recommended to avoid overstressing the piles during driving. Pile points or shoes are also recommended for both H piles and CIP piles to help protect them when driving past the cobbles.

Soils encountered in the frost zone consisted of mostly clay fill, which is a poor soil type for pavement support because of its high frost susceptibility. Soil maps of the area suggest that the native surface soils consist of Plano silt loam, which is also a poor soil type for pavement support because of its high frost susceptibility.

See below for further recommendations.

5. 2. Driven Piles

If a deep foundation is used, either 10x42 H piles or 10.75-inch CIP piles are expected to be the most economical choices for support of the new bridge. For maximum structural capacity, 10x42 H piles should be driven to a nominal axial resistance of 180 tons using the FHWA modified Gates dynamic formula, and 10.75-inch CIP piles with a shell thickness of 0.500 inches should be driven to a nominal axial resistance of 150 tons using the FHWA modified Gates dynamic formula.

At the east abutment (Boring 1 location), the required driving resistance of 180 tons for 10x42 H piles is expected to occur when the piles have been driven to the less weathered dolomite, which occurred at a depth of 52.5 feet (elevation 863.7) in Boring 1. The required driving resistance of 150 tons for maximum axial capacity of 10.75-inch CIP piles is expected to occur when the piles have been driven to a depth of 40 feet (elevation 876.2), several feet into the very dense sand and gravel found in Boring 1.

At the west abutment (Boring 2 location), the required driving resistance of 180 tons for 10x42 H piles is expected to occur when the piles have been driven to the less weathered dolomite, which occurred at a depth of 50.5 feet (elevation 866.3) in Boring 2. The required driving resistance of 150 tons for maximum axial capacity of 10.75-inch CIP piles is expected to occur when the piles have been driven to the weathered dolomite bedrock, which occurred at a depth of 46.5 feet (elevation 870.3) in Boring 2.

The soil density and degree of weathering of the bedrock varied between boring locations, and variations in pile penetrations may occur. Be prepared to cut or splice piles.

Unit skin friction and end bearing values shown in Table 5.2 may be used to estimate pile penetrations for piles driven to other resistances. Skin friction and end bearing values shown in Table 5.2 were estimated using the Alpha and Nordlund methods presented in the 7th edition of the AASHTO LRFD Bridge Design Specifications. The estimations were made with the aid of the FHWA computer software program 'Driven'. Note that unit skin friction for CIP piles in cohesionless soils increases with pile diameter. CIP piles of larger diameter will experience significantly larger skin friction, and shorter CIP pile penetrations may occur if CIP piles larger than 10.75 inches in diameter are used. For H piling, it is recommended that the steel cross section be used in the end bearing calculations.

Table 5.2. Soil Parameters, Skin Frictions, and End Bearings for Pile Design.

EAST ABUTMENT (Boring 1)

Soil Description	***Friction Angle (Deg)***	***Cohesion (psf)***	***Unit Weight (pcf)***	***Nominal Skin Friction† (psf)***	***Nominal End Bearing† (psf)***
CLAY, Sandy, V Stiff (Fill) (Elevation 916.2 to 903.2)	0	2,000	120	560	N/A
SAND & GRAVEL, Dense (Elevation 903.2 to 892.7)	39	0	130	610	69,000 - 135,000‡
SAND & GRAVEL, Very Dense (Elevation 892.7 to 868.2)	41	0	140	1,500	235,000 - 521,000‡
BEDROCK, Dolomite, Highly Weath. (Elevation 868.2 to 863.7)	43	0	150	2,200	677,000
BEDROCK, Dolomite, Less Weath. (Elevation 863.7 to 863.2)	0	High	160	1,280	Refusal*

†Skin friction and end bearing values are nominal (ultimate) values and have not been modified by a resistance factor.
‡End bearing increases linearly with depth in this range.
*Capacity of the bedrock exceeds the capacity of the piling.

Table 5.2. (cont) Soil Parameters, Skin Frictions, and End Bearings for Pile Design.

WEST ABUTMENT (Boring 2)

Soil Description	*Friction Angle (Deg)*	*Cohesion (psf)*	*Unit Weight (pcf)*	*Nominal Skin Friction† (psf)*	*Nominal End Bearing† (psf)*
CLAY, Sandy, V Stiff (Fill) (Elevation 916.8 to 903.8)	0	2,000	120	560	N/A
SAND & GRAVEL, Dense (Elevation 903.8 to 898.3)	36	0	130	370	37,000 - 55,000‡
SAND & GRAVEL, Very Dense (Elevation 898.3 to 870.3)	39	500	140	1,210	101,000 - 296,000‡
BEDROCK, Dolomite, Highly Weath. (Elevation 870.3 to 866.3)	43	0	150	2,150	677,000
BEDROCK, Dolomite, Less Weath. (Elevation 866.3 to 865.8)	0	High	160	1,280	Refusal*

†Skin friction and end bearing values are nominal (ultimate) values and have not been modified by a resistance factor.
‡End bearing increases linearly with depth in this range.
*Capacity of the bedrock exceeds the capacity of the piling.

5. 3. Pile Drivability
Drivability evaluations were performed for both 10.75-inch CIP piles and 10x42 H piles driven for maximum structural capacity at both boring locations using a Delmag D-16-32 diesel hammer. Evaluation results indicate that a shell thickness of at least 0.500 inches will be needed for 10.75-inch CIP piling driven at this site to avoid exceeding the 35 ksi compressive stress limit during driving through the dense sands at the site. Evaluation results indicated that the compressive stress limit would not be exceeded during driving of 10x42 H piles. Pile points/shoes are also recommended on the ends of the piles to help protect the piles while driving past the cobbles at the site.

5. 4. Spread Footings
Considering the native soils found below the abutment fill consisted of dense to very dense sand and gravel and the bridge does not cross a waterway, a shallow foundation could be considered for bridge support. All strip footings should have a minimum width of 18 inches, and all square footings should have a minimum width of 30 inches.

The shallow foundation should bear below the frost line to provide protection against frost heave. According to the Wisconsin Administrative Code, this site is in Zone ‘A’, where the mapped frost protection depth in the soil type in the area is approximately 4.5 feet. Be aware that frost can occur to depths significantly deeper than 4.5 feet below areas where heavy traffic occurs at the ground surface or where snow cover is frequently removed, such as below roadways or possibly below the railroad tracks.

The bearing capacity of the native sand and gravel has been calculated according to the 7th edition of the AASHTO LRFD Bridge Manual at the Strength I Limit State. Footings for the new bridge are expected to be at least 2.5 feet wide or wider and bear below the frost line. Considering the minimum footing dimensions, the unfactored soil bearing capacity of the native dense sand and gravel is approximately 33 kips per square foot (ksf). With resistance factor of 0.45, the factored soil bearing capacity for the expected shallow foundation is 15 ksf. However, the Wisconsin DOT Bridge Manual requires that settlement of the bridge be limited to 1.5 inches. Settlement of the shallow foundation has been estimated using the Hough Method at the Service I Limit State. To limit settlement of the shallow foundation to 1.5 inches, a soil bearing capacity of 6 ksf is recommended for shallow foundation design over the native, dense sand and gravel. Should it be necessary to further limit settlement, such as possibly near the railroad embankment, a soil bearing capacity of 4 ksf is recommended for shallow foundation design over the native, dense sand and gravel to limit settlement to 1.0 inch.

The above soil bearing capacities are applicable to the native, dense sand and gravel found below the abutment fill by both borings. However, these soil bearing capacities may also be used for shallow foundation design over engineered fill that is placed directly on the native dense sand and gravel and that is compacted to at least 95 percent of the maximum dry density as established by the Modified Proctor Method (ASTM D1557).

The unfactored coefficient of static friction against sliding between cast-in-place concrete and the native dense sand and gravel is at least 0.725. This friction coefficient may also be used where cast-in-place concrete is used over sand and gravel fill that is similar to the native soils and that is compacted as specified above.

The base of all shallow foundation excavations should be inspected at the time of construction to verify that adequate bearing capacity is present. Some undercutting and/or compaction of the native soils may be required to establish the required soil bearing capacity depending on site conditions or excavation techniques at the time of construction.

5. 5. Approach Pavement Design Parameters

The pavement construction should meet the requirements of the Wisconsin DOT Standard Specifications for Road and Bridge Construction.

A prime requirement for successful pavement is preparation of the subgrade soil. Prior to pavement placement, the base course should be firm and unyielding when proof-rolled. An acceptable proof-roller for silty or clayey soils would be a fully-loaded, tandem-axle dump truck. An acceptable proof-roller for granular soil (sand and/or gravel) would be a smooth-drum vibratory roller weighing at least 25,000 pounds. Any soft soils disclosed by the proof-rolling should be replaced with drier soil or stabilized with crushed rock or breaker run. Should undercutting or excavation below subgrade (EBS) be done, this applies after undercut locations are backfilled. Any breaker run or crushed rock used to stabilize a soft subgrade should not be considered as part of the base course thickness.

Assuming a stable subgrade is established prior to paving, pavement design will be controlled by the frost susceptibility of the near surface soils within the frost zone. Soils encountered in the frost zone consisted mostly of clay fill, which is a poor soil type for pavement support because of its high frost susceptibility. Soil maps of the area suggest that the native surface soils consist of Plano silt loam, which is also a poor soil type for pavement support because of its high frost susceptibility.

According to the Attachment 15.1 of Chapter 11-5 of the Facilities Design Manual (FDM), this area is mapped in the Wisconsin DOT's Standard Inclusion Area for Select Materials to improve pavement subgrade. The subgrade improvement consists of an undercut which is then filled with Select Materials. The depth of undercut depends on the type of Select Materials used to backfill the undercut. For example, if breaker run stone is used as the Select Material, the required depth of undercut is 16 inches. More guidance can be found in Attachment 15.2 of FDM Chapter 11-5. Select Materials were not observed below the pavement in the borings.

The recommended soil parameters for pavement design over the on-site soils and over the on-site soils after they have been improved with Select Materials are shown in Table 5.5, including Frost Group Designation (FGD), Design Group Index (DGI), Soil Support Value (SSV), California Bearing Ratio (CBR), modulus of subgrade reaction (k), USCS Classification, and AASHTO Classification. It is recommended that soil parameters for the on-site soils be used unless the soils below the approaches are improved with Select Materials.

Table 5.5. Recommended soil parameters for pavement design for the approaches.

Subgrade	***FGD***	***DGI***	***SSV***	***CBR***	***k (pci)***	***USCS***	***AASHTO***
On-Site Soils	F-4	16	3.6	4	125	SM / CL	A-4 / A-6
Select Materials	F-4	16	4.2	6	175	-	-

The soils encountered have a low shrink/swell potential as a result of loss/gain of moisture.

5. 6. Reuse of Existing Asphalt

The existing asphaltic concrete, if pulverized, may be reused as a form of base course. Consider the existing asphaltic concrete, when pulverized, to have a structural coefficient of not more than 0.10 and a CBR of 8.

The material found just below the pavement on both sides of the bridge appeared too sandy to meet current requirements for Wisconsin DOT base course. Use of a base course layer is recommended below the new pavement.

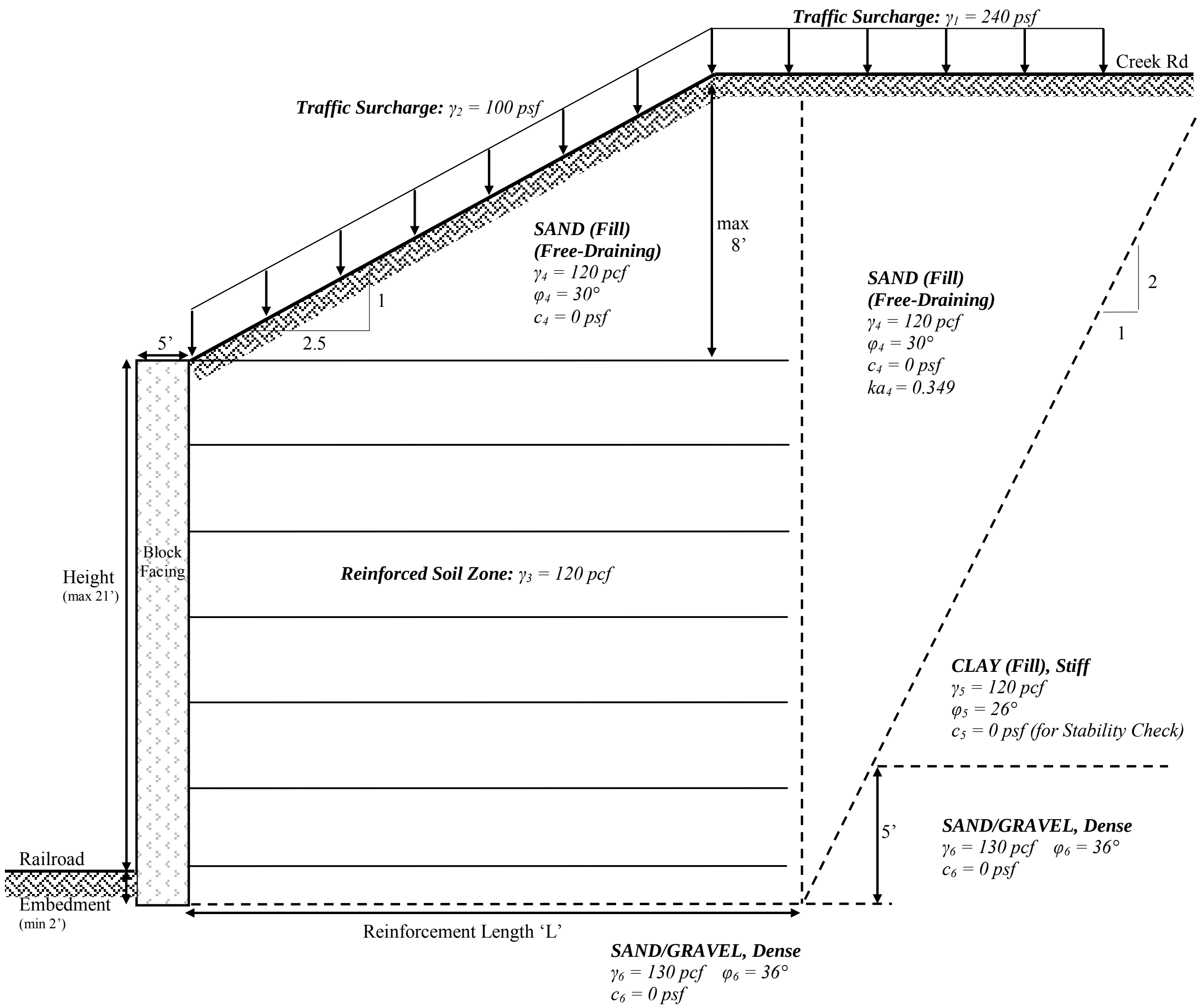
5. 7. MSE Wall External Stability and Settlement

The external stability of the proposed MSE walls was evaluated according to the 7th edition of the AASHTO Bridge Design Manual and Chapter 14 of the Wisconsin DOT Bridge Manual using Load and Resistance Factor Design (LRFD). Internal stability of the wall was not part of the scope of services provided by NTS, and it was assumed for the evaluation that the wall would remain internally stable (i.e. the soil reinforcement will not break/pullout, each reinforcement layer is a continuous sheet extending to the required lengths, etc). It is our understanding that the MSE walls will be a maximum exposed wall height of 21 feet. Should it be necessary to construct walls with a height greater than 21 feet, contact this writer for further recommendations.

It was assumed that all reinforced soil would be free-draining soil with unit weight of 120 pounds per cubic foot (pcf), and that all soil in the active zone behind the wall would be free-draining soil with unit weight of 120 pcf and friction angle of at least 30 degrees that will be drained such that no water pressure or saturated conditions will develop in the retained soil. Contributions to loads and resistances from the wall facing were not considered in the analysis. It is recommended that the wall be embedded at least 2 feet below the ground surface at the toe of the wall, and this minimum embedment was included in the external stability calculations. It was assumed that the wall would bear on the dense, native sand and gravel or on compacted fill with strength properties similar to the native sand and gravel. A traffic surcharge of 240 pounds per square foot (psf) was used to simulate vehicle loading on the soil below Creek Road, and a traffic surcharge of 100 psf was used along the face of the slope to simulate construction traffic.

A cross section of the wall/slope configuration used in the stability evaluation is shown in Figure 5.7.

Figure 5.7. Wall/Slope Cross-Section of the MSE Wall.



Each of the possible external wall/slope failure modes was considered, including failure of the overall wall/slope configuration (global stability), failure by wall sliding, failure by wall overturning, and bearing capacity failure below the wall. Settlement of the wall was also evaluated using the Hough Method assuming the wall bears on the native sands and/or compacted fill placed directly on the native sands. Failure by sliding, overturning, and bearing capacity were evaluated at the Strength I Limit State, and global stability and settlement were evaluated at the Service I Limit State. The wall stability for each failure mode is expressed using the Capacity-to-Demand Ratio (CDR), which is the ratio of the factored resistances to the factored loads. A CDR of 1.0 or greater indicates the wall will be stable considering the failure mode. A summary of the required reinforcement lengths at the max height and two lesser heights to maintain wall/slope stability are shown in Table 5.7. The CDRs for each failure mode and expected total wall settlements are also shown in Table 5.7.

Table 5.7. MSE Wall Reinforcement Lengths for External Stability and Corresponding CDRs.

Exposed MSE Wall Height	***Required Reinforcement Length***	***Global Stability CDR***	***Sliding CDR***	***Overturning CDR***	***Bearing Capacity CDR***	***Maximum Expected Settlement****
21 feet	17 feet	1.0	1.0	1.7	2.6	< 2.0 in
15 feet	14 feet	1.0	1.0	2.1	3.0	< 1.5 in
10 feet	12 feet	1.0	1.0	2.6	3.7	< 1.0 in
*Differential settlement is expected to be limited to no more than 1 inch over 500 inches of wall for all three wall heights.						

A load factor of 1.5 was used for horizontal earth loads, and a load factor of 1.35 was used for the vertical earth loads, including the weight of the wall and earth surcharge load. Resistance factors of 1.0 were used for sliding resistance and overturning resistance. A resistance factor of 0.65 was used in global stability because the slope supports a structural element. A resistance factor of 0.65 was used for soil bearing resistance. A load factor of 1.75 was used for traffic surcharge.

Reinforcement lengths and CDRs for wall heights between 10 and 21 feet which are not shown in the table may be determined by linear interpolation. No extrapolated values should be used.

Sliding resistance and global stability of the overall wall/slope determined the required reinforcement lengths. The global stability was evaluated at the Service I Limit State for the various wall heights. At the service limit state, load factors for overall stability are 1.0, and the resistance factor is 0.65 for slopes which support structural elements. The overall stability analysis was performed using the computer software program WinSTABL and the modified Bishop method. Note that WinSTABL calculates safety factors rather than CDRs. Because load factors are 1.0, the safety factor may be correlated to a CDR to determine stability using LRFD.

The safety factor against shear failure of the slope was calculated along several thousand possible failure arcs stemming from below the toe and at the toe and terminating below the wall and behind the wall. The lowest safety factor found against shear failure for each wall height was 1.5 using the reinforcement lengths shown in Table 5.7. This safety factor of 1.5 corresponds to a global stability CDR of 1.0, indicating that the proposed wall/slope geometry will be stable using those reinforcement lengths.

Graphs of the WinSTABL program inputs and outputs are appended. Output graphs show the 10 critical surfaces with the lowest factors of safety against shear failure and those corresponding factors of safety.

It is important that soils behind the wall do not become saturated. All reinforced soil and soil in the active zone behind the wall should be free-draining soil with drainage provided at the lowest point such that any water infiltrating the backfill is removed prior to saturation of the backfill. Free-draining soil is granular soil with less than 50 percent passing the US number 40 sieve along with less than 5 percent passing the US number 200 sieve. The active zone for this wall may be considered as all soil above a line drawn upward from the base of the back of the wall at an angle of 2 vertical to 1 horizontal.

It is also important that the overall wall/slope geometry be preserved. Erosion control measures should be implemented to prevent the slopes from becoming steeper. Should the geometry change, such as if the overall slope becomes steeper, the wall and slope may become unstable. After placement of the soil reinforcement, the reinforcement should not be disturbed or damaged, or the wall may become unstable. No excavation should be performed behind the wall facing, below the wall, or near the toe of the wall unless proper precautions are exercised.

5. 8. Excavation

All excavations should comply with OSHA standards. This includes reduction of excavation side slopes to 1.5 horizontal to 1 vertical or less. Where steeper slopes are necessary or more convenient, full excavation bracing should be used (not spaced braces). Design and implementation of temporary shoring is generally the responsibility of the excavating contractor.

Most common excavators (backhoes) are expected to be able to make the necessary excavations. Cobbles occurred in the native soils that may make excavation of the native soils difficult. Any cobbles disturbed during excavation should be removed, and the surrounding soil compacted.

The regional water table is not expected to be encountered in excavations, though perched water may occur. Perched water may be sump pumped from the excavation.

Undercutting may be required to remove uncontrolled fill or other soils from below the new foundations. When undercutting, use the 60-degree approximation to determine the resulting pressure at the base of the undercut. The recommended width of undercut is twice the undercut depth plus the width of the load-bearing area, measured at the bottom of cut. If the load-bearing area is accurately marked and centered in the base of the undercut, then the minimum width of the undercut is the depth of undercut plus the width of the load-bearing area, measured at the base of the undercut. If the entire building footprint is undercut, the undercut should extend outside the slab edges a distance of one-half the undercut depth. A good practice is to add at least one foot to these widths. Replace all undercut soils with properly compacted fill (see Section 5.9 “Compaction and Fill Requirements”).

Excavations should be performed with a flat plate attached to the bucket teeth of the backhoe to minimize the disturbance at the base of the excavation. Where a toothed bucket is used, the last six inches (roughly) should be excavated by turning the bucket so that the teeth are parallel to the proposed grade, thus minimizing the disturbance of footing-grade soils. Any soil loosened during excavation should be compacted or removed by hand.

5. 9. Compaction and Fill Requirements

Use of a base course layer, at least 8 inches in thickness, is recommended below all slabs-on-grade and pavement. The base course should meet the requirements for dense-graded base course of Section 305 of the Wisconsin DOT Standard Specifications.

Soils at the site which contain no deleterious materials, that are unfrozen, and that are at a moisture content appropriate for compaction at the time of construction, should be suitable for reuse as structural fill. Structural fill is any fill which must support the weight of a structure. Note that most of the fill encountered was lean clay, and obtaining adequate compaction of the clay will be more moisture dependent and may be difficult to achieve. It may be better to use granular soil as structural fill.

If imported fill is required as structural fill, NTS recommends granular soil free of deleterious materials such as organics, snow, ice, sod, frozen soil, and construction debris. Free-draining sand is recommended as backfill in the active zone against the retaining walls to prevent hydrostatic pressure from building up against the walls. Free-draining sand is sand with less than 50 percent passing the number 40 sieve and less than 5 percent passing the number 200 sieve. The active zone should be considered as the zone above a line drawn from the bottom of the backside of the retaining wall going upward into the retained fill at an angle of 60 degrees from the horizontal. A suitable outlet for water should be provided at the bottom of the sand layer against any retaining walls.

At the time of construction, NTS should verify that the proposed fill soils are acceptable. NTS will verify that the moisture content is appropriate for proper compaction and that the fill contains no deleterious materials. Frozen soil should not be used as structural fill.

Any required fill should be placed in lifts not exceeding 1 foot (uncompacted).

Compact all structural fill to at least 95 percent of the maximum dry density (modified Proctor method - ASTM D1557). The recommended compaction control method for fill placed elsewhere is the DOT standard compaction method. If the compaction of any fill is questionable, compaction tests may be performed on that fill. The compaction check should include one laboratory compaction test per field density determination. All nuclear testing should be calibrated to site soils by ASTM Method D6938. However, no work should be accepted that does not meet the requirements for standard compaction, regardless of test results. Site or soil conditions at the time of construction may warrant a change in the recommended compaction levels and/or techniques. However, no changes should be made without review by NTS.

5. 10. Soil Parameters

Table 5.10 shows the recommended soil parameters for on-site inorganic soils and for imported, free-draining sand fill. Table 5.10 includes soil unit weights for dry, moist, and submerged conditions in pounds per cubic foot (pcf), internal angle of friction in degrees (deg), and cohesion in pounds per square foot (psf). These parameters are for the soils when compacted according to Section 5.9 of this report. Free-draining sand is sand with less than 50 percent passing the number 40 sieve and less than 5 percent passing the number 200 sieve. Free-draining sand is recommended against earth-retaining structures to prevent hydrostatic pressure from building up against the walls.

Table 5.10. Estimated Soil Parameters for the Soils Encountered.

Soil Type	Unit Weights – Dry / Moist / Sub (pcf)	Friction Angle (deg)	Cohesion (psf)
Sand/Gravel, Compacted (On-Site)	135 / 145 / 85	36	0
Clay, Compacted (On-Site)	115 / 130 / 75	26	> 1,000
Sand, Free-Draining, Compacted (Imported)	110 / 120 / 60	30	0

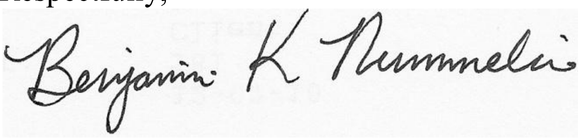
5. 11. Drainage

The site should be graded to promote drainage, and all drainage measures should be routed to a suitable outlet.

A pavement cross-slope of two percent is recommended to promote drainage in paved areas.

Soil in the active zone of the retaining walls should have drainage such that hydrostatic pressure cannot build up behind the walls.

Respectfully,



Benjamin K. Nummelin, P.E.
Nummelin Testing Services, Inc.
bkn/mw

NUMMELIN TESTING SERVICES, INC

GEOTECHNICAL ENGINEERING REPORT INFORMATION SHEET

Subsurface soil conditions are responsible for many of the construction problems encountered at building sites. In order to help you, our client, manage your risks, we offer you the following information and suggestions.

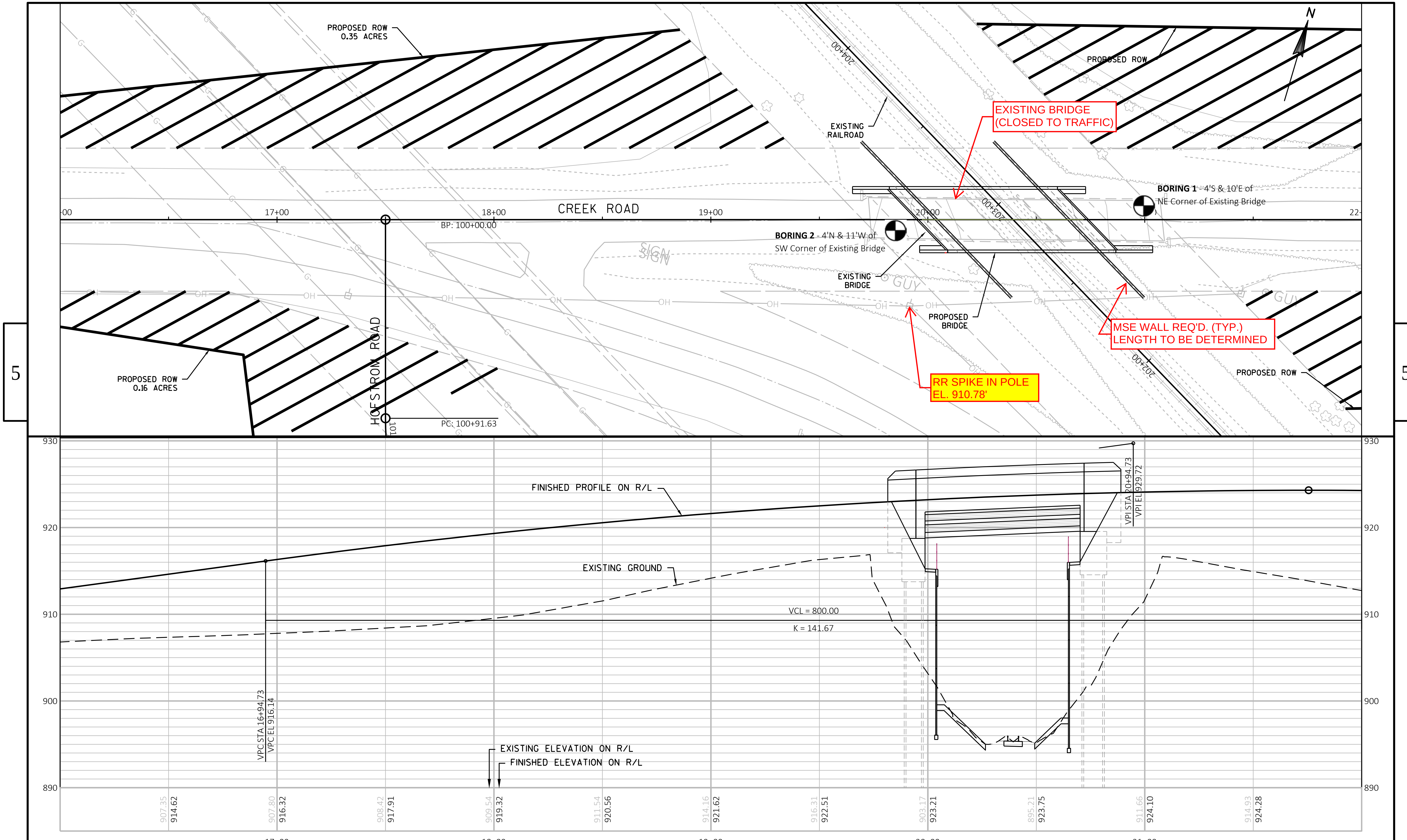
Geotechnical engineering reports are based on observations of specific soil conditions existing at the time of the subsurface soil investigation. As these conditions may change over time, construction decisions should be made with the timeliness of the report in mind. Further testing may be advisable if subsurface soil conditions are affected by natural events (flooding, spring thaws, etc.) and construction (drilling, blasting, surcharges, etc.) on-site or adjacent to it. Talking to your geotechnical professional before construction begins will help keep one informed if further tests are recommended.

The recommendations included in your geotechnical engineering report are based on a limited number of samples/tests. These recommendations assume that subsurface conditions throughout the site will be similar to those observed. As all recommendations are preliminary when based on limited testing, it is important to have your geotechnical professional observe the actual conditions during construction. This allows him/her to note any differences that may not have been revealed by the limited samples/tests and/or that are more abrupt than reported in the preliminary report. It is this geotechnical professional, using his/her knowledge and familiarity of site history, as well as construction observations, who will be able to determine if there is adequate and appropriate support to consider these recommendations final. He/she will also be able to document that the contractor is following these recommendations. Be aware that this geotechnical professional can not assume responsibility and/or liability for his/her recommendations based on observations and determinations by others.

Professional judgement, based on experience and observations, is at the heart of our geotechnical recommendations. Geotechnical reports use information from a limited number of samples/tests to predict conditions regarding your overall site. No one may say with certainty what subsurface conditions really exist without actual observation. The conditions away from sample/test areas may vary from what is predicted. It is important to identify variations as early as possible. This is why we encourage you to take advantage of our knowledge and experience during the construction phase of your project. Working together we can help minimize the impact when unexpected variations occur.

Geotechnical reports are written for a specific client, purpose, project and set of conditions. They are not intended to be a generalized, generic report for a proposed site. They are for the sole use of our client for the express purpose indicated to us. Should the scope of the project be altered, or if subsurface variations become evident during construction, it may be necessary to modify our recommendations. Early communication with your geotechnical professional can help you avoid expensive problems that may occur when changes to a project's purpose, structure, size, usage, site orientation, elevation, etc. are made after a report is written.

Following these guidelines, your geotechnical subsurface report should provide informed and accurate information to assist in the planning and construction of your project.



PROJECT NO: 3614-00-05	HWY: CREEK ROAD	COUNTY: ROCK	PLAN AND PROFILE: CREEK ROAD - 50 MPH	SHEET	E

FILE NAME : S:\PROJECTS\K51090 CREEK ROAD, ROCK COUNTY\SHEETSPLAN\PLAN & PROFILE SHEETS\36140005_050101_PP_CREEK_50 MPH WITH ELEVATION AND LAYOUT.DWG
LAYOUT NAME - Creek (1)
PLOT DATE : 11//29//2018 2:31 PM PLOT BY : BOLAND, PATRICK PLOT NAME : PLOT SCALE : 1 IN:40 FT
WISDOT//CADDS SHEET 44

NUMMELIN TESTING SERVICES, INC.

BORING LOG NOTES

DESCRIPTIVE TERM, GRANULAR SOIL (% BY DRY WEIGHT)

Trace	0% - 5%
Little	5% - 12%
Some	12% - 35%
And	35% - 50%

Q_P = Estimated Unconfined Compressive Strength (by pocket penetrometer) Expressed in tons per square foot (t/sf).

Q_U = Estimated Unconfined Compressive Strength (by ASTM 2166) Expressed in tons per square foot (t/sf).

NM = Natural Moisture

M = MOISTURE

D = Dry	F = Frozen
M = Moist	W = Wet
S = Saturated	

LOI = Loss on Ignition (Organic Content)

N (Standard Blow Count) = blows per foot, as shown. Performed in general accordance with Standard Penetration Test Specifications (ASTM 1586).

NR = No Recovery **WOH** = Weight of Hammer **#** = Sample Number

SOIL CLASSIFICATION

F = Fine	LL = Liquid Limit, percent
M = Medium	PL = Plastic Limit, percent
C = Coarse	PI = Plasticity Index (LL - PL)
W.L. = Water Level	

SOIL STRENGTH CHARACTERISTICS

CONSISTENCY (Cohesive Soils)

Term	Q_U tons/sq ft
Very Soft	0.0 to 0.25
Soft	0.25 to 0.50
Firm	0.50 to 1.0
Stiff	1.0 to 2.0
Very Stiff	2.0 to 4.0
Hard	Over 4.0

RELATIVE DENSITY (Granular Soils)

Term	"N" Value
Very Loose	0 - 4
Loose	4 – 10
Medium-Dense	10 – 30
Dense	30 - 50
Very Dense	Over 50

ORGANIC CONTENT BY COMBUSTION METHOD

Soil Description	Loss on Ignition
Non Organic	Less than 4%
Organic Silt / Clay	4 - 12%
Sedimentary Peat	12 - 50%
Fibrous & Woody Peat	More than 50%

PLASTICITY

Term	Plastic Index
None to Slight	0 - 4
Slight	5 - 7
Medium	8 - 22
High to Very High	Over 22

geotechborenotes.bor

SOIL BORING LOG

Boring By: Nummelin Testing Services, Inc.

Project: Creek Road Bridge, Project ID 3614-00-05
Location: East Abutment - See Map
WSOR Crossing, Town of Bradford, Rock County, WI

Boring: 1
Auger: HSA

Drillers: BM / NH
Date: 1/9/2019
Elevation: 916.2

Depth (ft.)		Classification/Description	#	Sample Depth (ft.)	N_{60}	Rec (in.)	M	Qp (tsf)	Notes
	-	4.0" of Asphaltic Concrete PAVEMENT							
1	-	3.5" of Brn SAND & GRAVEL (Base Course)	1	1 - 3	5	14	W		
	-								
2	-								
	-								
3	-		2	3 - 5	5	14	W		
	-	Brown Sandy CLAY							
4	-	Thin F-M Sand Layers							
	-	(Fill)							
5	-	(USCS: CL, AASHTO: A-6)	3	5 - 7	5	12	W	2.0	
	-								
6	-								
	-								
7	-		4	7 - 9	5	20	W	2.0	
	-	------ 7.5' ------							
8	-	Dark Brown / Gray Lean CLAY							
	-	Some Organics							
9	-	(Fill) (USCS: CL, AASHTO: A-6)	5	9 - 11	8	3	W		
	-	------ 8.5' ------							
10	-								
	-	Brown Lean CLAY							
11	-	(Fill)	6	11 - 13	9	12	M	2.0	
	-	(USCS: CL, AASHTO: A-6)							
12	-								
	-								
13	-	------ 13.0' ------	7	13 - 15	13	12	M		
	-	Light Brown F-M SAND							
14	-	Few Cobbles							
	-	(USCS: SP, AASHTO: A-3)							
15	-	------ 15.0' ------	8	15 - 17	67/3"	3	M		
	-								
16	-								
	-								
17	-		9	17 - 18.5	29	12	M		
	-								
18	-								
	-		10	18.5 - 20	39	12	M		
19	-	Light Brown SAND & GRAVEL							
	-	Little Silt, Cobbles							
20	-	(USCS: GP-GM, AASHTO: A-1-a)							
	-								
21	-								
22	-								
	-								
23	-								Hit Cobble
	-		11	23.5 - 25	67/3"	NR			w/ Sampler
24									@ 23.5'
	-								
25	-	(continued)							

SOIL BORING LOG

		Boring:	1
Boring By:	Nummelin Testing Services, Inc.	**Auger:**	HSA
		Page:	2 of 2
Project:	Creek Road Bridge, Project ID 3614-00-05	**Drillers:**	BM / NH
Location:	East Abutment - See Map	**Date:**	1/9/2019
	WSOR Crossing, Town of Bradford, Rock County, WI	**Elevation:**	916.2

Depth (ft.)		Classification/Description	#	Sample Depth (ft.)	N_{60}	Rec (in.)	M	Qp (tsf)	Notes
	-	(continued)							Mud
26	-								Rotary
	-								Below 25'
27	-								
	-								
28	-								
	-		12	28.5 - 30	47	12	M		
29	-								
	-								
30	-								
	-								
31	-								
	-								
32	-								
	-								
33	-								
	-		13	33.5 - 35	80	12	M		
34	-								
	-	Light Brown SAND & GRAVEL							
35	-	Little Silt, Cobbles							
	-	(USCS: GP-GM, AASHTO: A-1-a)							
36	-								
	-								
37	-								
	-								
38	-								
	-		14	38.5 - 40	56	5	M		
39	-								
	-								
40	-								
	-								
41	-								
	-								
42	-								
	-								
43	-								
	-		15	43.5 - 45	64	12	M		
44	-								
	-								
45	-								
	-								
46	-								
	-								
47	-								Hard
	-	------ 48.0' ------							Drilling
48	-	Brown SAND & GRAVEL							Below 48'
---		(Highly Weathered Dolomite Bedrock)	16	48.5 - 50	67/2"	2	M		
52		(Less Weathered Dolomite Bedrock @ 52.5')							Auger
	-	----- E.O.B. 52.5' -----							Refusal
53	-	----- Backfilled w/ Bentonite Chips -----							@ 52.5'

SOIL BORING LOG

Boring By: Nummelin Testing Services, Inc.

Project: Creek Road Bridge, Project ID 3614-00-05
Location: West Abutment - See Map
WSOR Crossing, Town of Bradford, Rock County, WI

Boring: 2
Auger: HSA
Page: 1 of 2
Drillers: BM / NH
Date: 1/14/2019
Elevation: 916.8

Depth (ft.)		Classification/Description	#	Sample Depth (ft.)	N_{60}	Rec (in.)	M	Qp (tsf)	Notes
	-	5.5" of Asphaltic Concrete PAVEMENT							
1	-	5.0" of Brn SAND, Some Gravel (Sandy Base)	1	1 - 3	5	16	W		
	-	Brown F-M SAND							
2	-	(Fill) (USCS: SP, AASHTO: A-3)							
	-	------ 2.0' ------							
3	-		2	3 - 5	5	14	W		
	-								
4	-	Dark Brown Sandy CLAY							
	-	Thin F-M Sand Layers							
5	-	(Fill)	3	5 - 7	8	6	W		
	-	(USCS: CL, AASHTO: A-6)							
6	-								
	-								
7	-	------ 7.0' ------	4	7 - 9	9	16	M		
	-	Brown F-M SAND, Little Gravel							
8	-	(Fill) (USCS: SP-SM, AASHTO: A-3)							
	-	------ 8.5' ------							
9	-	Dark Gray Lean CLAY, Some Organics	5	9 - 11	11	16	M	1.5	
	-	(Fill) (USCS: CL, AASHTO: A-6)							
10	-								
	-	------ 10.5' ------							
11	-		6	11 - 13	15	16	M	3.0	
	-	Brown Lean CLAY							
12	-	(Fill) (USCS: CL, AASHTO: A-6)							
	-								
13	-	------ 13.0' ------	7	13 - 15	15	14	M		
	-								
14	-								
	-								
15	-	Light Brown F-M SAND	8	15 - 17	24	14	M		
	-	Some Gravel, Cobbles							
16	-	(USCS: SP, AASHTO: A-1-b)							
	-								Hit Cobble
17	-		9	17 - 18.5	60	2	M		w/ Sampler
	-								@ 17'
18	-								
	-	------ 18.5' ------	10	18.5 - 20	51	10	M		
19	-								
	-								
20	-								
	-								
21	-	Light Brown SAND & GRAVEL							
		Little Silt, Cobbles							
22	-	(USCS: GP-GM, AASHTO: A-1-a)							
	-								
23	-								
	-		11	23.5 - 25	59	8	M		
24									
	-								
25	-	(continued)							

SOIL BORING LOG

Boring By: Nummelin Testing Services, Inc.

Project: Creek Road Bridge, Project ID 3614-00-05
Location: West Abutment - See Map
WSOR Crossing, Town of Bradford, Rock County, WI

Boring: 2
Auger: HSA

Drillers: BM / NH
Date: 1/14/2019
Elevation: 916.8

Depth (ft.)		Classification/Description	#	Sample Depth (ft.)	N_{60}	Rec (in.)	M	Qp (tsf)	Notes
	-	(continued)							
26	-								
	-								
27	-								
	-								
28	-								
	-		12	28.5 - 30	48	10	M		
29	-								
	-								
30	-								
	-								
31	-								
	-								
32	-								
	-								
33	-								
	-		13	33.5 - 35	27	10	M		
34	-								
	-	Light Brown SAND & GRAVEL							
35	-	Little Silt, Cobbles							
	-	(USCS: GP-GM, AASHTO: A-1-a)							
36	-								
	-								
37	-								
	-								
38	-								Hit Cobble
	-		14	38.5 - 40	73	2	W		w/ Sampler
39	-								@ 38.5'
	-								
40	-								
	-								
41	-								
	-								
42	-								
	-								
43	-								
	-		15	43.5 - 45	51	10	M		
44	-								
	-								
45	-								
	-								
46	-								Hard
	-	------ 46.5' ------							Drilling
47	-	Brown SAND & GRAVEL							Below 46.5'
	-	(Highly Weathered Dolomite Bedrock)							
48	-								
---		(Less Weathered Dolomite Bedrock @ 50.5')	16	48.5 - 50	67/2"	NR			
50		----- E.O.B. 50.5' -----							Auger
	-	----- Dry @ Completion -----							Refusal
51	-	----- Backfilled w/ Bentonite Chips -----							@ 50.5'

State of Wisconsin- Dept of Natural Resources
P.O. Box 7921, Madison WI 53707-7921

Well / Drillhole / Borehole Abandonment
Form 3300-005 (R 10/03) Page 1

Notice: Completion of this report is required by chs. 160, 281, 283, 289, 291-293, 295 and 299, Wis Stats., and ch. NR 141, Wis. Adm. Code. In accordance with chs. 281, 289, 291-293, 295, and 299, Wis. Stats., failure to file this form may result in a forfeiture of between $10-25,000, or imprisonment for up to one year, depending on the program and conduct involved. Personally identifiable information on this form is not intended to be used for any other purpose. Return form to the appropriate DNR office and bureau. See instructions for more information.

Route To:
[ ] Drinking Water [ ] Watershed Water [ ] Waste Management [ ] Remediation/Redevelopment [ ] Other: ______________

1. General Information

Boring Number	DNR Well ID No.	County
1		Rock

Common Well Name:
Gov't Lot # (if applic.):
1/4 / 1/4 | 1/4 | Section | Township N | Range [ ] E [ ] W
Grid Location: Feet [ ] N [ ] S [ ] E [ ] W
[ ] Local Grid Origin [ ] (estimated) OR [ ] Well Location
Latitude: DEG MIN SEC N
Longitude: DEG MIN SEC W
Reason For Abandonment: Borehole Termination
WI Unique Well No. of Replacement Well: ___ ___ ___ ___ ___

2. Facility / Owner Information

Facility Name: Creek Road Bridge, Project ID 3614-00-05
Facility ID: 175.06
License/Permit No.:
City, Village, or Town: Bradford Town
Street Address of Well: WSOR Crossing
Present Well Owner:
Original Well Owner:
Street Address or Route of Owner:
City | State | ZIP Code

3. Well / Drillhole / Borehole Information

[ ] Monitoring Well
[ ] Water Well
[x] Borehole / Drillhole

Original Construction Date: 1/9/2019
If a Well Construction Report is available, please attach.

Construction Type:
[x] Drilled [ ] Driven (sandpoint) [ ] Dug
[ ] Other (specify): ______________________________

Formation Type:
[x] Unconsolidated Formation [x] Bedrock

Total Well Depth From Groundsurface (ft.):
Casing Diameter (in.):
Lower Drillhole Diameter (in.):
Casing Depth (ft.):
Was Well Annular Space Grouted? [ ] Yes [ ] No [ ] Unknown
If yes, to what depth (feet)?
Depth to water (feet)

4. Pump, Liner, Screen, Casing & Sealing Material

	Yes	No	N/A
Pump and piping removed?	[ ]	[ ]	[x]
Liner(s) removed?	[ ]	[ ]	[x]
Screen removed?	[ ]	[ ]	[x]
Casing left in place?	[ ]	[ ]	[x]
Casing cut off below surface?	[ ]	[ ]	[x]
Sealing material rise to surface?	[x]	[ ]	[ ]
Material settle after 24 hrs?	[ ]	[ ]	[x]
If yes, was hole retopped?	[ ]	[ ]	[x]
If bentonite chips were used, were they hydrated with water from a known safe source?	[ ]	[ ]	[x]

Required Method of Placing Sealing Material
[ ] Conductor Pipe-Gravity [ ] Conductor Pipe-Pumped
[ ] Screened and Poured (Bentonite Chips) [ ] Other (explain): ______________

Sealing Materials
[ ] Neat Cement Grout [ ] Clay Sand Slurry (11lb/gal wt.)
[ ] Sand Cement (concrete) Grout [ ] Bentonite-Sand Slurry
[ ] Concrete [x] Bentonite Chips

For Monitoring Wells and Monitoring Well Boreholes Only:
[ ] Bentonite Chips [ ] Bentonite-Cement Grout
[ ] Granular Bentonite [ ] Bentonite-Sand Slurry

5. Material Used to Fill Well / Drillhole	From (ft.)	To (ft.)	No. Yards, Sacks Sealant or Volume (circle one)	Mix Ratio or Mud Weight
3/8" Bentonite Chips	Surface	52.5		

6. Comments

7. Supervision of Work

Name of Person or Firm Doing Sealing Work: NTS, Inc.
Date of Abandonment: 01/09/19
Street or Route: P.O. Box 127
Telephone Number: (715) 341-7974
City: Stevens Point | State: WI | ZIP Code: 54481
Signature of Person Doing Work:
Date Signed:

DNR Use Only
Date Received: | Noted By:
Comments:

State of Wisconsin- Dept of Natural Resources
P.O. Box 7921, Madison WI 53707-7921

Well / Drillhole / Borehole Abandonment

Form 3300-005 (R 10/03) Page 1

Notice: Completion of this report is required by chs. 160, 281, 283, 289, 291-293, 295 and 299, Wis Stats., and ch. NR 141, Wis. Adm. Code. In accordance with chs. 281, 289, 291-293, 295, and 299, Wis. Stats., failure to file this form may result in a forfeiture of between $10-25,000, or imprisonment for up to one year, depending on the program and conduct involved. Personally identifiable information on this form is not intended to be used for any other purpose. Return form to the appropriate DNR office and bureau. See instructions for more information.

Route To:
☐ Drinking Water ☐ Watershed Water ☐ Waste Management ☐ Remediation/Redevelopment ☐ Other: _______________

1. General Information

Boring Number	DNR Well ID No.	County
2		Rock

Common Well Name:
Gov't Lot # (if applic.):

1/4 / 1/4: | 1/4: | Section: | Township: N | Range: ☐ E ☐ W

Grid Location: Feet ☐ N ☐ S ☐ E ☐ W
☐ Local Grid Origin ☐ (estimated) OR ☐ Well Location

Latitude: DEG MIN SEC N
Longitude: DEG MIN SEC W

Reason For Abandonment: Borehole Termination
WI Unique Well No. of Replacement Well: ___ ___ ___ ___ ___

2. Facility / Owner Information

Facility Name: Creek Road Bridge, Project ID 3614-00-05

Facility ID	License/Permit No.	City, Village, or Town
175.06		Bradford Town

Street Address of Well: WSOR Crossing
Present Well Owner:
Original Well Owner:
Street Address or Route of Owner:
City: | State: | ZIP Code:

3. Well / Drillhole / Borehole Information

☐ Monitoring Well
☐ Water Well
☑ Borehole / Drillhole

Original Construction Date: 1/14/2019
If a Well Construction Report is available, please attach.

Construction Type:
☑ Drilled ☐ Driven (sandpoint) ☐ Dug
☐ Other (specify): _______________

Formation Type:
☑ Unconsolidated Formation ☑ Bedrock

Total Well Depth From Groundsurface (ft.):
Casing Diameter (in.):
Lower Drillhole Diameter (in.):
Casing Depth (ft.):

Was Well Annular Space Grouted? ☐ Yes ☐ No ☐ Unknown
If yes, to what depth (feet)?
Depth to water (feet):

4. Pump, Liner, Screen, Casing & Sealing Material

	Yes	No	N/A
Pump and piping removed?	☐	☐	☑
Liner(s) removed?	☐	☐	☑
Screen removed?	☐	☐	☑
Casing left in place?	☐	☐	☑
Casing cut off below surface?	☐	☐	☑
Sealing material rise to surface?	☑	☐	☐
Material settle after 24 hrs?	☐	☐	☑
If yes, was hole retopped?	☐	☐	☑
If bentonite chips were used, were they hydrated with water from a known safe source?	☐	☐	☑

Required Method of Placing Sealing Material
☐ Conductor Pipe-Gravity ☐ Conductor Pipe-Pumped
☐ Screened and Poured (Bentonite Chips) ☐ Other (explain): _______________

Sealing Materials
☐ Neat Cement Grout
☐ Sand Cement (concrete) Grout
☐ Concrete
☐ Clay Sand Slurry (11lb/gal wt.)
☐ Bentonite-Sand Slurry
☑ Bentonite Chips

For Monitoring Wells and Monitoring Well Boreholes Only:
☐ Bentonite Chips ☐ Bentonite-Cement Grout
☐ Granular Bentonite ☐ Bentonite-Sand Slurry

5. Material Used to Fill Well / Drillhole

Material Used to Fill Well / Drillhole	From (ft.)	To (ft.)	No. Yards, Sacks Sealant or Volume (circle one)	Mix Ratio or Mud Weight
3/8" Bentonite Chips	Surface	50.5		

6. Comments

7. Supervision of Work

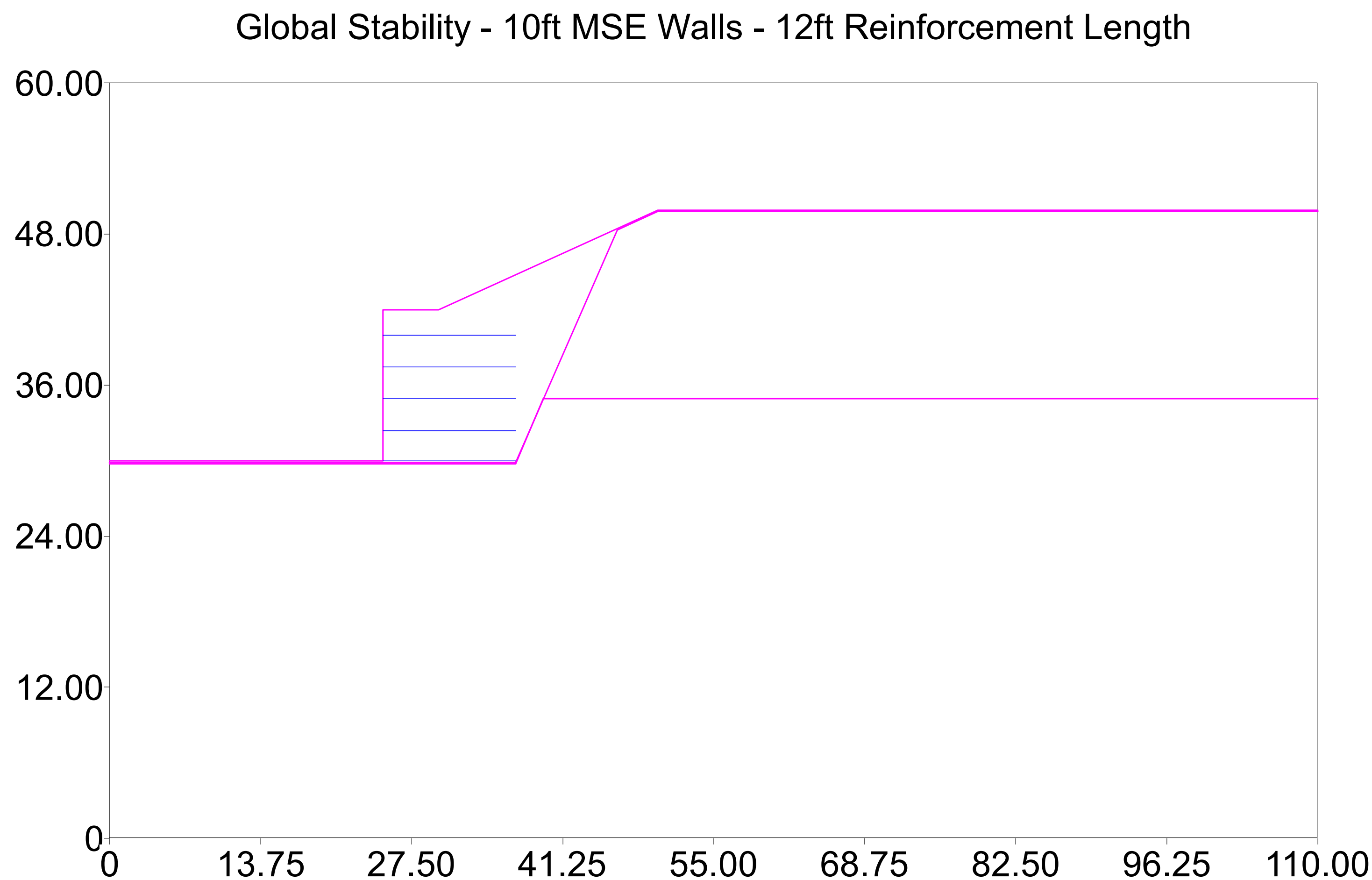
Name of Person or Firm Doing Sealing Work	Date of Abandonment
NTS, Inc.	01/14/19

Street or Route	Telephone Number
P.O. Box 127	(715) 341-7974

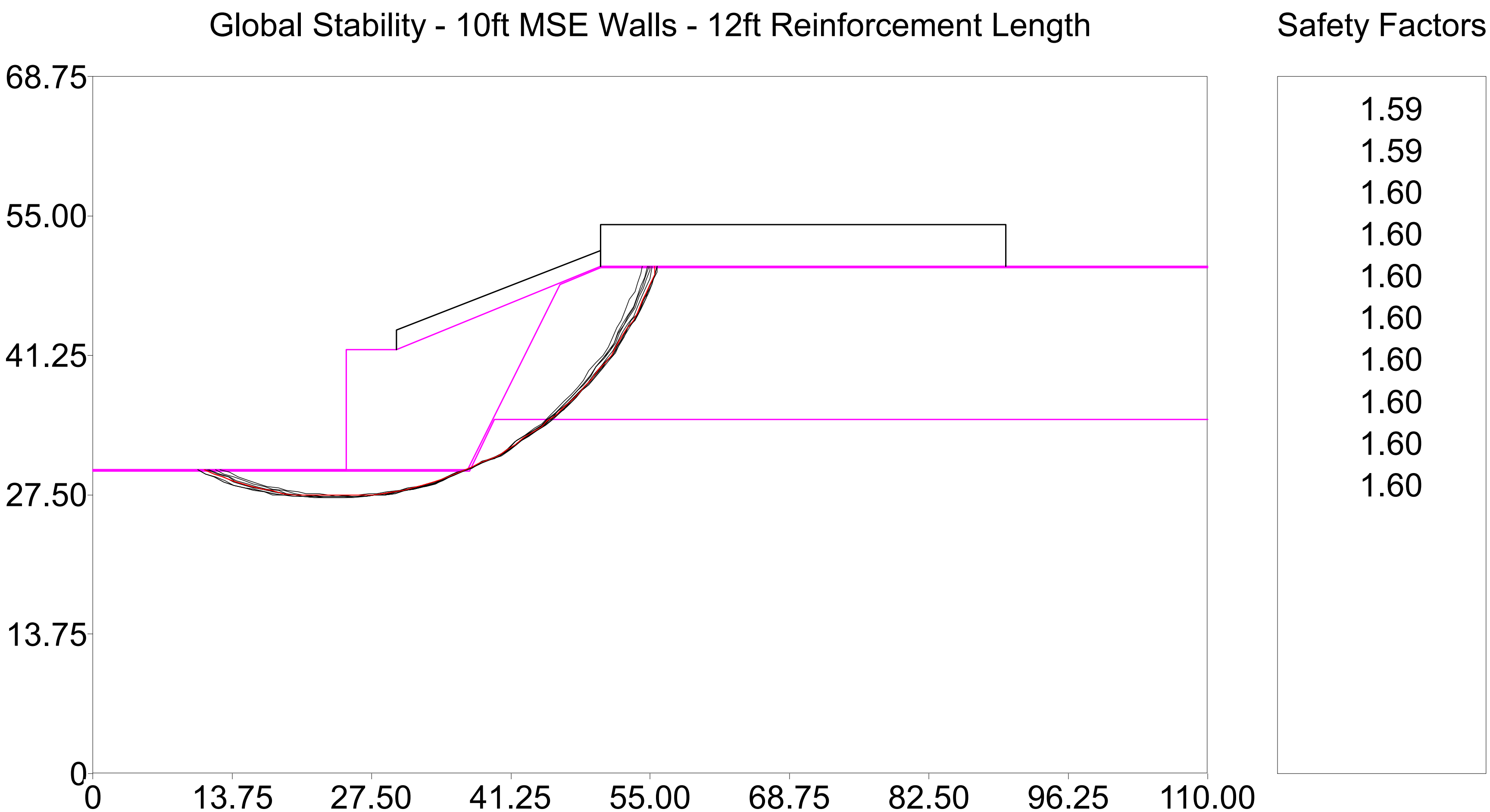
City	State	ZIP Code
Stevens Point	WI	54481

Signature of Person Doing Work: | Date Signed:

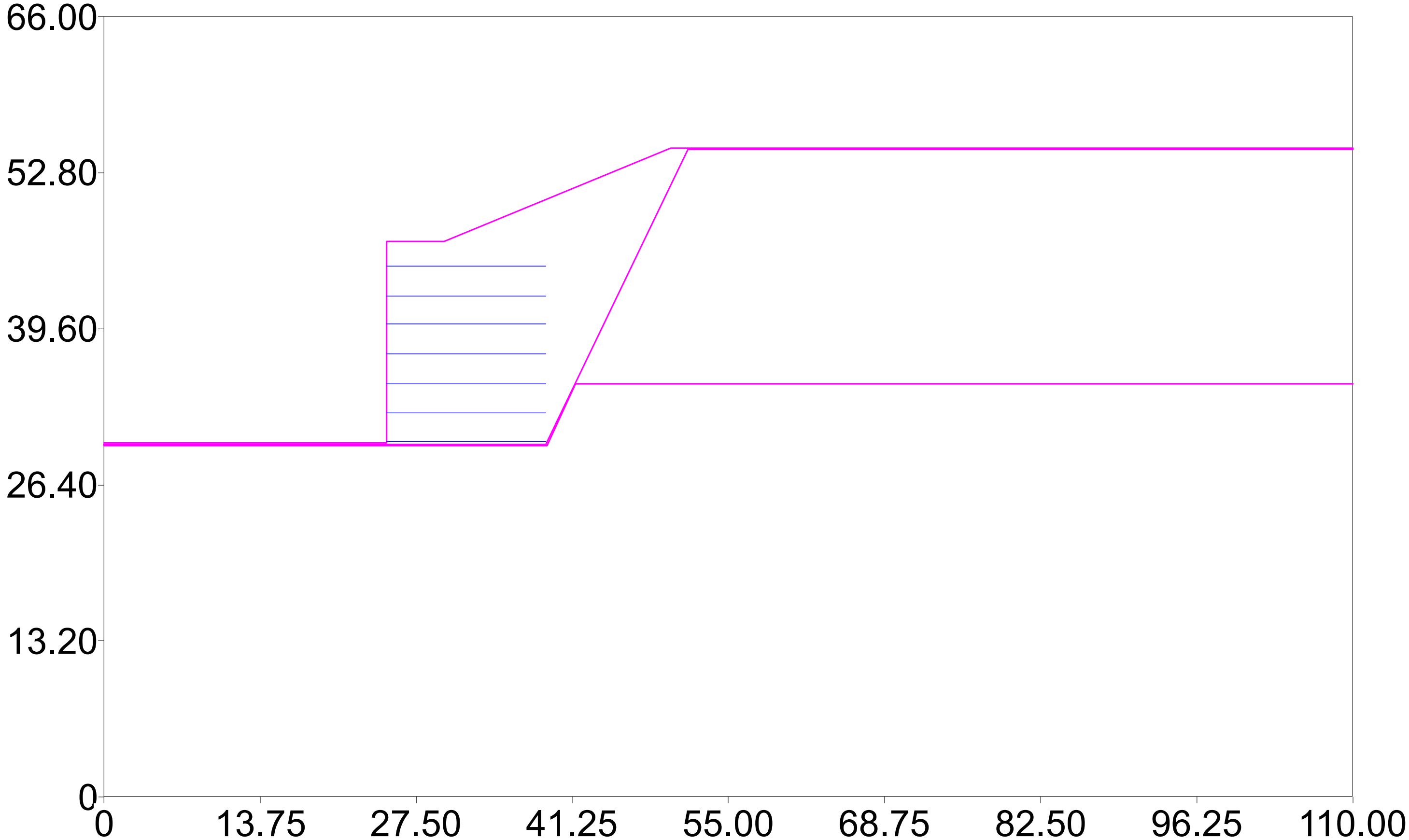
DNR Use Only
Date Received: | Noted By:
Comments:



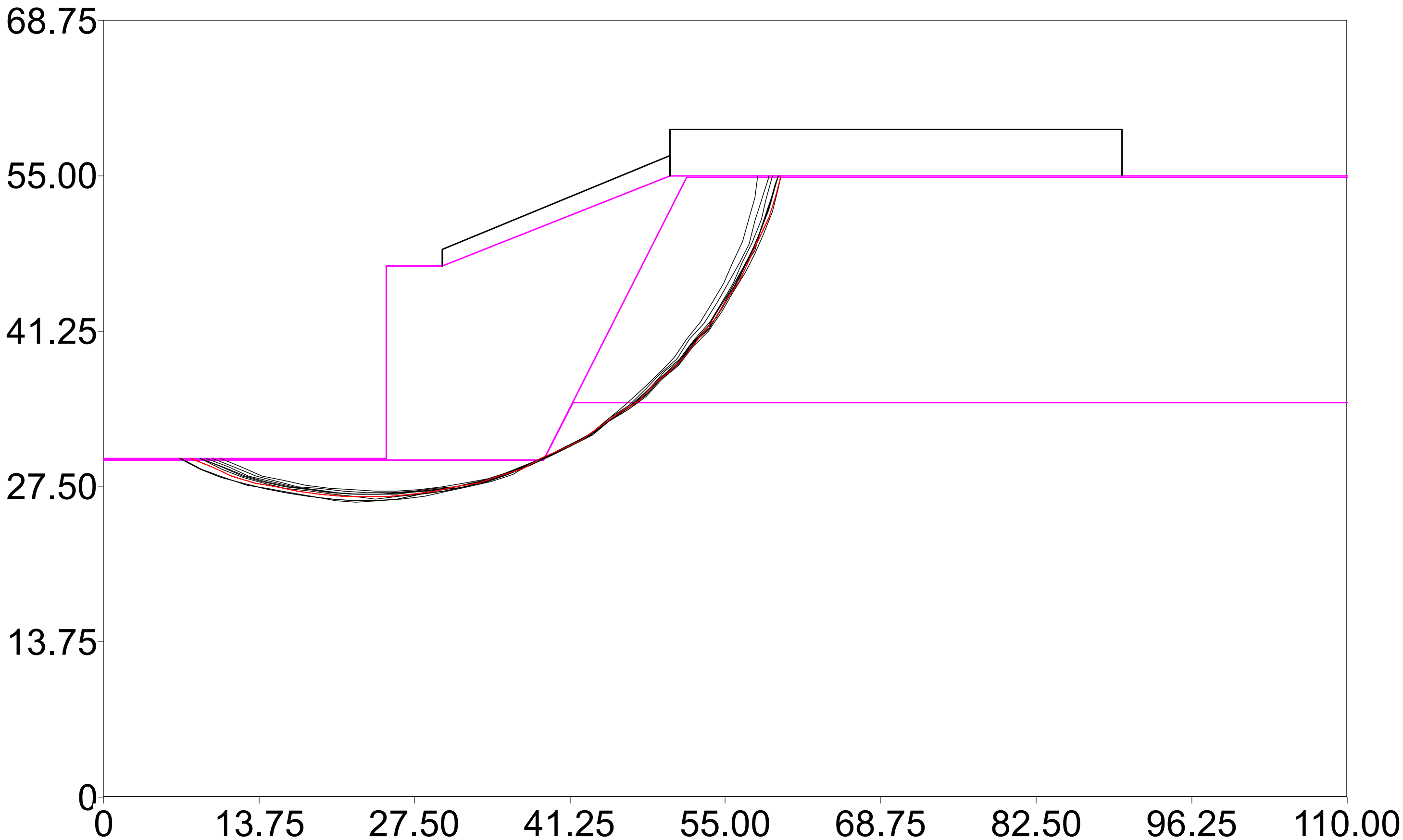
Global Stability - 10ft MSE Walls - 12ft Reinforcement Length
60.00
48.00
36.00
24.00
12.00
0
0
13.75
27.50
41.25
55.00
68.75
82.50
96.25
110.00


Global Stability - 10ft MSE Walls - 12ft Reinforcement Length
Safety Factors
68.75
55.00
41.25
27.50
13.75
0
0
13.75
27.50
41.25
55.00
68.75
82.50
96.25
110.00
1.59
1.59
1.60
1.60
1.60
1.60
1.60
1.60
1.60
1.60

Global Stability - 15ft MSE Walls - 14ft Reinforcement Length



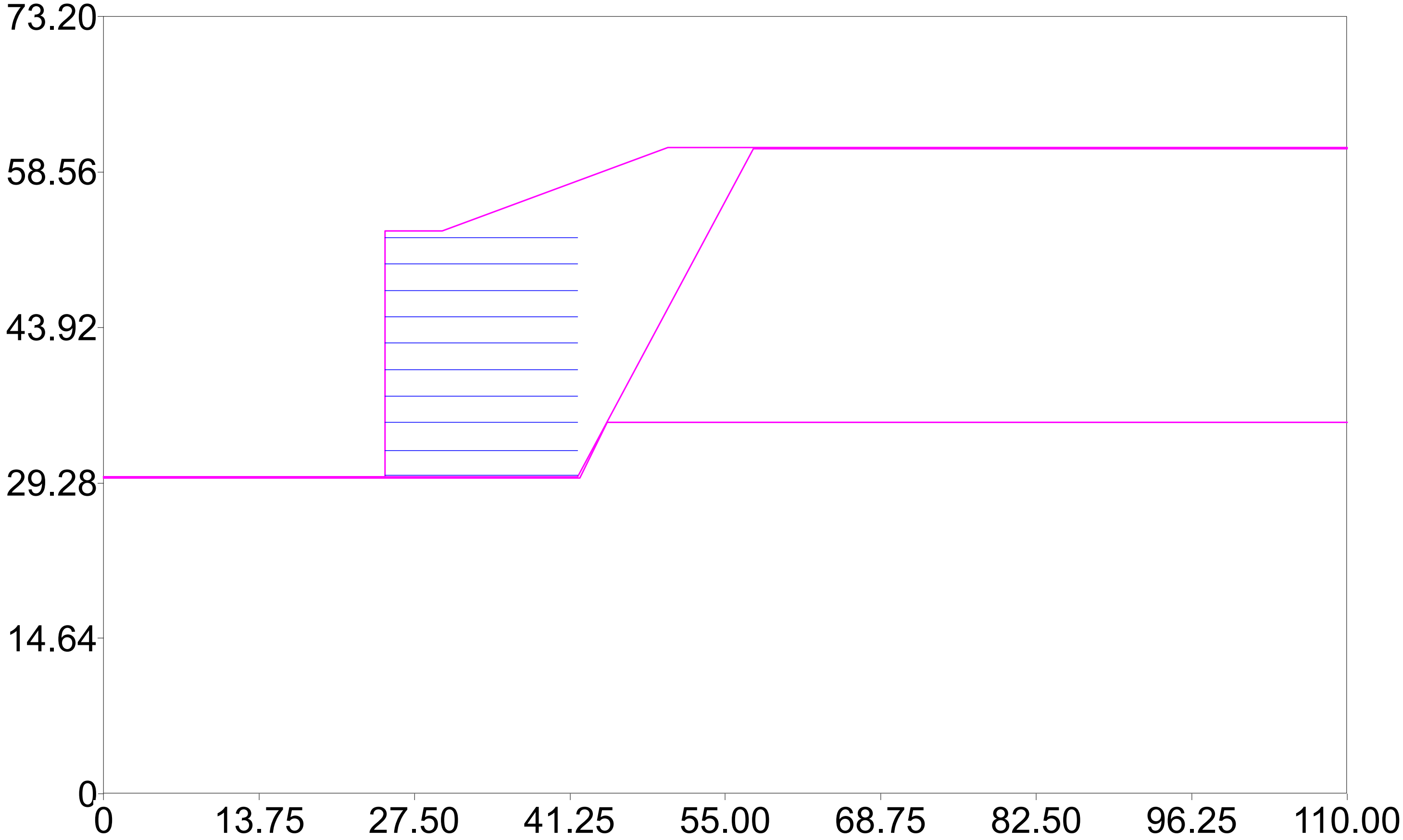
Global Stability - 15ft MSE Walls - 14ft Reinforcement Length

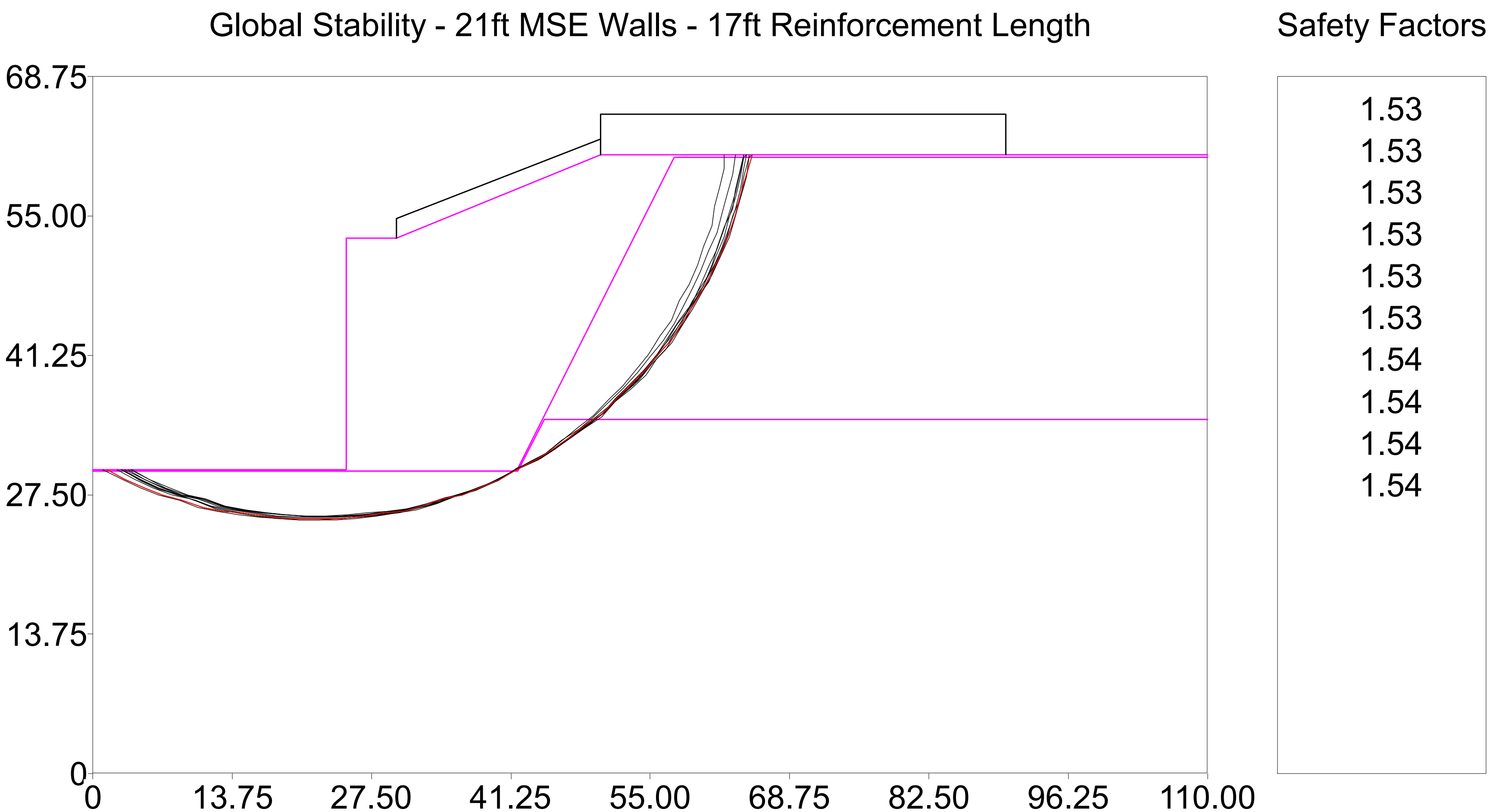


Safety Factors

1.54
1.54
1.54
1.54
1.54
1.54
1.55
1.55
1.55
1.55

Global Stability - 21ft MSE Walls - 17ft Reinforcement Length




Global Stability - 21ft MSE Walls - 17ft Reinforcement Length
Safety Factors
1.53
1.53
1.53
1.53
1.53
1.53
1.54
1.54
1.54
1.54
68.75
55.00
41.25
27.50
13.75
0
0
13.75
27.50
41.25
55.00
68.75
82.50
96.25
110.00