Geotechnical Evaluation Report

64th Avenue South Overpass of Interstate I-29
City of Fargo Project Number PN-19-A0
Fargo, North Dakota

Prepared for

KLJ

Professional Certification:
I hereby certify that this plan, specification, or report was prepared by me or under my direct supervision and that I am a duly Registered Professional Engineer under the laws of the State of North Dakota.

Ezra Ballinger, PE
Business Unit Leader, Senior Engineer
Registration Number: PE-7328
August 26, 2020

Project B1600465

Braun Intertec Corporation
August 26, 2020

Scott Middaugh, PE
KUJ
3203 32nd Avenue South, Suite 102
Fargo, ND 58103

Re: Geotechnical Evaluation Report
   64th Avenue South Overpass of Interstate I-29
   City of Fargo Project Number PN-19-A0
   Fargo, North Dakota

Dear Mr. Middaugh:

We are pleased to present this Geotechnical Evaluation Report for the proposed 64th Avenue South overpass of Interstate I-29. The results of our fieldwork and analyses are presented below.

Braun Intertec previously published a Geotechnical Evaluation Report, and has since published three Addendum reports to address various design changes made over the past four years. This report reflects the sum of those design changes and, referencing the project’s final design plans, thus supersedes our earlier reports.

Thank you for making Braun Intertec your geotechnical consultant for this project. If you have questions about this report, or if there are other services that we can provide in support of our work to date, please contact Ezra Ballinger at 701.492.5872 (eballinger@braunintertec.com)

Sincerely,

BRAUN INTERTEC CORPORATION

[Signature]
Ezra Ballinger, PE
Business Unit Leader, Senior Engineer

[Signature]
Charles D. Hubbard, PE
Technical Leader, Principal Engineer/Geologist

AA/EOE
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A. Introduction

A.1. Project Description

This Geotechnical Evaluation Report addresses the design and construction of a new overpass of Interstate I-29 at 64th Avenue South in Fargo, North Dakota. The project is illustrated in a series of documents provided to us by KLJ and contained within Appendix A, and includes a Bridge Layout (plan, elevation and section), approach embankment cross sections (for the west embankment), and a project plan/profile of the overpass. Figure 1, using a Google Earth aerial background, illustrates the project setting.

Figure 1. Site Layout.

The bridge will consist of a 2-span structure with a single pier in the median between north and southbound I-29, and abutments to the east and west. The bridge will be 400 feet long, its deck supported on 7 concrete girders spanning between the abutments and piers. The bridge deck will support the 2-lane roadway as well as a multi-use path, making the bridge nearly 52 feet wide.

The proposed finished grades at the east and west ends of the bridge will be approximately 935 feet. Existing grades along the alignment centerline vary from approximately 907 to 909 feet from west to east. The approach embankments will ascend toward the bridge at a 3 percent slope. The maximum grade raise will be approximately 26 1/2 feet.
A.2. Site Conditions and History

Currently, 64th Avenue South exists as a gravel road terminating west and east of I-29. Within the interstate right of way, shallow ditches and a median convey water away from either side of the existing northbound and southbound lanes.

A.3. Purpose

The purpose of our geotechnical evaluation is to provide KLJ with the geotechnical information required to complete the design and prepare plans and specifications for construction of the bridge foundations and approach embankments.

A.4. Background Information and Reference Documents

Our geotechnical evaluation was based primarily on the following documents:

- Geologic maps from Bulletin 47, County Groundwater Studies 8, Part I, Plates 1, 2 and 3, by the United States Geological Survey (USGS).
- PS&E plan set titled City of Fargo Improvement District BN-21-A1, Cass County, 64th Ave S – 38th Street S to 33rd Street South, Project No. SU-8-984(153)156, PCN 21564, by KLJ and dated July 29, 2020.

We have described our understanding of the proposed construction and site to the extent others reported it to us. Depending on the extent of available information, we may have made assumptions based on our experience with similar projects. If we have not correctly recorded or interpreted the project details, the project team should notify us. New or changed information could require additional evaluation, analyses and/or recommendations.

A.5. Scope of Services

We performed our scope of services for the project in accordance with our Proposal QTB028125 to KLJ, dated October 12, 2015, which KLJ authorized on November 3, 2015. Tasks completed in accordance with our authorized scope of services included:

- Reviewing the previously cited background information and reference documents.
• Selecting and clearing exploration location of underground utilities. We selected boring locations based on the preliminary bridge plans and provided those locations to KLJ who staked them. KLJ provided us the horizontal coordinates and surface elevations of the borings after staking.

• Performing eight standard penetration test (SPT) borings, denoted ST-01 to ST-08, to nominal depths of 25 to 100 feet below existing grades.

• Performing laboratory tests on selected penetration test and thin-walled tube samples to help classify the materials encountered and estimate or measure their engineering properties.

• Performing engineering analyses, including approach embankment settlement and stability, and bridge foundation capacity.

• Preparing this report containing a Soil Boring Location Sketch, logs of the penetration test borings, a summary of the materials encountered, results of our laboratory tests, and recommendations for bridge foundation design, approach embankment construction, and pavement design.

Our scope of services did not include environmental services or testing, nor did we train the personnel performing this evaluation to provide environmental services or testing.

B. Results

B.1. Geologic Overview

Geologically, the area is dominated by glacial lake deposits (lacustrine soils) consisting mainly of fat clays that are strength sensitive and compressible. These soils are underlain at an appreciable depth by glacial till, also consisting mainly of clay but of greater strength and limited compressibility.

The lacustrine soils, from the ground surface down, generally belong to the Sherack, Brenna, and Argusville Formations. These soils are locally overlain with existing fill from historic grading and construction. While consisting mainly of fat clay, the existing fill and lake-deposited soils also consist locally of lean clay and silt. Silt in particular is often prevalent near the lower boundary of the Sherack Formation, and slickensides are often prevalent in the underlying Brenna Formation, helping reveal the boundary between the two formations.
The Sherack and upper portion of the Brenna Formations are generally over-consolidated, while the lower portion of the Brenna and the Argusville Formations are normally consolidated. Shear strength typically falls to a minimum below the zone of over-consolidation but rises again with depth. The soils are typically saturated or nearly so, even above the hydrostatic groundwater surface, and possess low to very low hydraulic conductivities.

We based the geologic origins used in this report on the soil types, laboratory testing, and available common knowledge of the geological history of the site. Because of the complex depositional history, geologic origins can be difficult to ascertain. We did not perform a detailed investigation of the geologic history for the site.

B.2. Subsurface Geologic Profile

Table 1 summarizes the depths and thicknesses of the various materials encountered by the borings, which are described below in more detail below.

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Topsoil/Fill</th>
<th>Sherack Formation</th>
<th>Ox. Brenna Formation</th>
<th>Brenna Formation</th>
<th>Argusville Formation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Depth^A</td>
<td>Thickness</td>
<td>Depth^A</td>
<td>Thickness</td>
<td>Depth^A</td>
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<tr>
<td>ST-01</td>
<td>2 1/2</td>
<td>2 1/2</td>
<td>9</td>
<td>6 1/2</td>
<td>18</td>
</tr>
<tr>
<td>ST-02</td>
<td>3</td>
<td>3</td>
<td>10</td>
<td>7</td>
<td>18</td>
</tr>
<tr>
<td>ST-03</td>
<td>2</td>
<td>2</td>
<td>9</td>
<td>7</td>
<td>18</td>
</tr>
<tr>
<td>ST-04</td>
<td>3</td>
<td>3</td>
<td>8</td>
<td>5</td>
<td>19</td>
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<tr>
<td>ST-05</td>
<td>5</td>
<td>5</td>
<td>12 1/2</td>
<td>6 1/2</td>
<td>18</td>
</tr>
<tr>
<td>ST-06</td>
<td>4</td>
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<td>12 1/2</td>
<td>7 1/2</td>
<td>23</td>
</tr>
<tr>
<td>ST-07</td>
<td>4 1/2</td>
<td>4 1/2</td>
<td>11 1/2</td>
<td>7</td>
<td>18</td>
</tr>
<tr>
<td>ST-08</td>
<td>4 1/2</td>
<td>4 1/2</td>
<td>12 1/2</td>
<td>8</td>
<td>18</td>
</tr>
</tbody>
</table>

^A Depth to bottom of stratum, in feet.

B.2.a. Topsoil and Existing Fill

From the surface, the borings encountered 2 1/2 to 5 feet of topsoil and/or existing fill (for simplicity in this report, we define existing fill to mean uncontrolled or undocumented – not engineered – fill). The
average thickness was just over 3 1/2 feet. The topsoil and fill generally contained some organic root matter, were black, and were moist.

The moisture contents of these materials ranged from 14 to 29 percent, with an average of about 22 percent.

B.2.b. Sherack Formation
Below the topsoil and/or fill, the borings typically encountered native fat clays (ASTM symbol "CH") associated with the Sherack Formation. The clays of the Sherack Formation are distinguishable from underlying formations due to the presence of silt and/or sand lenses and seams, and lower moisture contents and plasticity. The lower boundary of the Sherack Formation was penetrated at depths of approximately 8 to 12 1/2 feet.

The moisture contents of the Sherack Formation clays ranged from 16 to 39 percent; from our experience, these values indicate the clays were near to well above their optimum moisture contents.

B.2.c. Brenna Formation
Below the Sherack Formation, the borings encountered fat clays associated with the Brenna Formation. The Brenna Formation’s clays have few if any silt/sand lenses and seams, and have much higher moisture contents and plasticity. The upper portion of the Brenna Formation is referred to as the "Oxidized Brenna" and is identified by its mixed gray and brown color. The "Oxidized Brenna" terminates where the soils become more uniformly gray. The bottom of the Oxidized Brenna appeared to extent to depths of approximately 18 to 23 feet; the Brenna Formation on the whole appeared to continue to a depth of approximately 60 feet at each boring location.

The moisture contents of the Brenna Formation ranged from 38 to 63 percent, indicating they, too, were well above their optimum moisture contents. The liquid limit (LL) of the Brenna varied from 65 to 116.

B.2.d. Argusville Formation
Below the Brenna, the borings encountered fat clays of the Argusville Formation. The Argusville clays have higher sand contents than the Brenna, and slightly lower plasticity. Typically, these differences are difficult through visual classification and require laboratory testing to confirm the depth of the boundary between the Brenna and Argusville. The bottom of the Argusville Formation appeared to extent to depths of approximately 78 to 79 feet.

The moisture contents of the Argusville Formation ranged from 32 to 63 percent. The liquid limit (LL) of the Argusville clays ranged from 70 to 80.
B.2.e. Glacial Till

Below the Argusville, and extending to the termination depths of the borings, sandy lean clay glacial till was encountered. The glacial till became very hard below a depth of about 85 feet, corresponding to an elevation of about 820 feet.

B.2.f. Standard Penetration Resistances

Penetration resistance values recorded in the Sherack, Brenna, and Argusville Formations ranged from 1 to 12 blows per foot (BPF), with an average between 4 and 5 BPF, indicating they were generally soft to rather stiff in consistency.

Penetration resistance values recorded in the glacial till ranged from 12 to 50 BPF within the upper 5 to 10 feet of the till. At greater depths, values increased from about 50 to in excess of 100 BPF.

B.2.g. Groundwater

Groundwater was not observed in any of the borings during advancement. The lack of a measurable groundwater level within an open borehole is not uncommon in the area, and is typically due to the low permeability of the area clays. Groundwater levels can take hours or days to stabilize in open boreholes.

To gather groundwater level information, three vibrating wire (VW) piezometers were installed in the borehole left by Boring ST-03. The VW piezometers were installed on September 8, 2016 at depths of about 15, 35 and 60 feet. Graph 1, below, shows measured groundwater elevations at each of the VW piezometers (note the piezometers are identified by depth in the graph legend).
As indicated in Graph 1, the hydrostatic groundwater elevation at Boring ST-03 decreased with depth, indicating downward groundwater flow. For near-surface excavations, the results indicate the groundwater elevation has been relatively steady at about 890 feet, approximately 18 feet below the ground surface, with an approximate 1-foot “bump” in late winter and early spring.

B.3. Laboratory Test Results

In addition to tests performed on material samples obtained from the borings performed for this project, we referred to our database of laboratory testing data from the numerous projects performed for the FMM Diversion and other Fargo-Moorhead area flood control projects.

B.3.a. Classification and Index Testing

Moisture content tests (per ASTM D2216), dry and wet density tests, Atterberg limits tests (per ASTM D4318), and sieve-hydrometer analyses (per ASTM D422 and D1140) were performed to help classify the materials encountered, estimate formation transition depths, and help estimate or measure directly the engineering properties of the materials encountered. Table 2, below, summarizes the results of our tests.

The majority of the results shown in Table 2 are also provided on the Log of Boring Sheets included in Appendix A. The moisture content tests are listed within the column labeled “MC”, the dry density (DD), wet density (WD), and Atterberg limit results are listed in the column labeled “Tests or Notes”. The
Atterberg limits are identified in the column with the symbols LL (liquid limit), PL (plastic limit), and PI (plastic index). Test reports for the sieve-hydrometer analyses are included in Appendix B.

### Table 2. Summary of Classification and Index Testing

<table>
<thead>
<tr>
<th></th>
<th>Moisture Content (%)</th>
<th>Wet Unit Weight (pcf)</th>
<th>Dry Unit Weight (pcf)</th>
<th>Atterberg limits</th>
<th>Sieve-Hydrometer Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Liquid Limit (%)</td>
<td>Plastic Limit (%)</td>
</tr>
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<td>Topsoil/Fill</td>
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<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>Range 14 to 29</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>Average 22</td>
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<td>--</td>
<td>--</td>
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<tr>
<td>Sherack Formation</td>
<td># of Tests 23</td>
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<td>--</td>
<td>--</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Range 16 to 39</td>
<td>114</td>
<td>83</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>Average 29</td>
<td>114</td>
<td>83</td>
<td>--</td>
<td>--</td>
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<tr>
<td>Oxidized Brenna Formation</td>
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<td>1</td>
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<td>3</td>
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<tr>
<td></td>
<td>Range 38 to 63</td>
<td>105 to 110</td>
<td>67 to 76</td>
<td>105</td>
<td>25</td>
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<tr>
<td></td>
<td>Average 50</td>
<td>108</td>
<td>72</td>
<td>105</td>
<td>25</td>
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<tr>
<td>Brenna Formation</td>
<td># of Tests 46</td>
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<td>9</td>
<td>3</td>
<td>3</td>
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<tr>
<td></td>
<td>Range 42 to 58</td>
<td>104 to 109</td>
<td>66 to 71</td>
<td>65 to 116</td>
<td>20 to 33</td>
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<td></td>
<td>Average 52</td>
<td>106</td>
<td>68</td>
<td>94</td>
<td>24</td>
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<tr>
<td>Argusville Formation</td>
<td># of Tests 13</td>
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<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Range 32 to 63</td>
<td>100 to 111</td>
<td>65 to 75</td>
<td>70 to 80</td>
<td>22 to 24</td>
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<tr>
<td></td>
<td>Average 48</td>
<td>106</td>
<td>70</td>
<td>75</td>
<td>23</td>
</tr>
<tr>
<td>Glacial Till</td>
<td># of Tests 10</td>
<td>2</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>Range 12 to 24</td>
<td>127 to 134</td>
<td>109 to 118</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>Average 16</td>
<td>131</td>
<td>114</td>
<td>--</td>
<td>--</td>
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</table>
**B.3.b. Shear Strength Testing**

*Unconfined Compressive Strength Testing*

Eight unconfined compressive strength \((Q_u)\) tests (per ASTM D2166) were performed on selected thin-walled tube samples to aid in estimating the soils’ undrained shear strength. The results are summarized below in Table 3.

**Table 3. Summary of Unconfined Compressive Strength Testing**

<table>
<thead>
<tr>
<th>Soil Formation</th>
<th>Number of Tests</th>
<th>Range of Test Results (psf)</th>
<th>Average Unconfined Compressive Strength (psf)</th>
<th>Average Estimate Undrained Shear Strength (psf)&lt;sup&gt;a&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sherack</td>
<td>1</td>
<td>1,400</td>
<td>1,400</td>
<td>700</td>
</tr>
<tr>
<td>Oxidized Brenna</td>
<td>3</td>
<td>1,420 to 2,470</td>
<td>1,880</td>
<td>940</td>
</tr>
<tr>
<td>Brenna</td>
<td>5</td>
<td>2,090 to 2,780</td>
<td>2,556</td>
<td>1,278</td>
</tr>
<tr>
<td>Argusville</td>
<td>0</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Glacial Till</td>
<td>0</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
</tbody>
</table>

<sup>a</sup> Undrained shear strength is determined as half the unconfined compressive strength, representing a “peak” undrained shear strength.

Pocket penetrometer tests \((q_p)\) were also performed on all cohesive penetration test samples to provide an additional estimation of the soils’ unconfined compressive strength. These tests results were not considered in our direct calculations, but were rather reviewed in conjunction with the standard penetration test results to evaluate the soils’ consistencies and changes with depth. The penetrometer recorded unconfined compressive strengths from 1/4 to 4 ½-plus tons per square foot (tsf) and are presented in the column labeled “\(q_p\)” on the Log of Boring Sheets in Appendix A.

*Consolidated-Undrained Triaxial Shear Strength Testing*

We performed three consolidated-undrained (CU) triaxial shear strength tests with porewater pressure measurements (per ASTM D4767) to measure the soils’ shear strength properties and for the development of curvilinear shear strength envelopes. Shear strength parameters were measured at strains of 15%. The CU test results for the tests we performed are summarized below in Table 4.
Table 4. Summary of Triaxial Shear Strength Testing

<table>
<thead>
<tr>
<th>Boring</th>
<th>Depth</th>
<th>Formation</th>
<th>Total Stress Properties</th>
<th>Effective Stress Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Friction Angle</td>
<td>Cohesion</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(degrees)</td>
<td>(psf)</td>
</tr>
<tr>
<td>ST-03</td>
<td>39 1/2 - 41 1/2</td>
<td>Brenna</td>
<td>6.2</td>
<td>535</td>
</tr>
<tr>
<td>ST-04</td>
<td>14 1/2 – 16 1/2</td>
<td>Oxidized Brenna</td>
<td>8.7</td>
<td>550</td>
</tr>
<tr>
<td>ST-05</td>
<td>24 1/2 – 26 1/2</td>
<td>Brenna</td>
<td>5.7</td>
<td>500</td>
</tr>
</tbody>
</table>

B.3.c. Consolidation Tests

Compression Characteristics

We performed three (3) time-rate consolidation tests to determine the soils’ settlement characteristics for settlement calculations. The results of our tests are provided below in Table 5.

Table 5. Summary of Consolidation Test Results

<table>
<thead>
<tr>
<th>Boring</th>
<th>Test Depth (ft)</th>
<th>Formation</th>
<th>WD (pcf)</th>
<th>MC (%)</th>
<th>Atterberg Limits (LL/PI)</th>
<th>s_c (tsf)</th>
<th>OCR</th>
<th>C_c</th>
<th>C_r</th>
<th>e_o</th>
</tr>
</thead>
<tbody>
<tr>
<td>ST-04</td>
<td>30</td>
<td>Brenna</td>
<td>105</td>
<td>54</td>
<td>87 / 54</td>
<td>3.63</td>
<td>3.83</td>
<td>0.84</td>
<td>0.18</td>
<td>1.475</td>
</tr>
<tr>
<td>ST-04</td>
<td>50</td>
<td>Brenna</td>
<td>103</td>
<td>60</td>
<td>116 / 91</td>
<td>3.64</td>
<td>2.62</td>
<td>1.19</td>
<td>0.18</td>
<td>1.623</td>
</tr>
<tr>
<td>ST-05</td>
<td>60</td>
<td>Brenna</td>
<td>107</td>
<td>51</td>
<td>85 / 60</td>
<td>3.09</td>
<td>1.70</td>
<td>0.73</td>
<td>0.15</td>
<td>1.379</td>
</tr>
</tbody>
</table>

Abbreviations: WD = total density, MC = moisture content, LL = liquid limit, PI = plastic index, s_c = preconsolidation pressure, s_o’ = initial effective overburden pressure, OCR = overconsolidation ratio, C_c = compression index, C_r = recompression index, e_o = initial void ratio.

Time-Rate of Compression Characteristics

For each of the time-rate consolidation tests we measured the soils’ coefficient of consolidation ($c_v$), which was used to calculate the rate at which the soils will consolidate. Since 2012, we have performed consolidation testing for about 10 bridges that are anticipated to be constructed over the proposed Fargo-Moorhead Metro Diversion channel. As the settlement mechanism is very similar for the proposed bridge over I-29, in an effort to select appropriate values based on confining pressures, the values of the all testing we have performed for the Diversion project to date were compared to the tests for the current project. The individual test reports for the tests used are available in our Diversion reports previously submitted or are individually available upon request.
Graph 2, below summarizes the $c_v$ values calculated at their corresponding pressures. Note the $c_v$ values are presented in units of ft$^2$/day. Trend-lines were applied to the data sets for each of the formations. Since only a single time-rate consolidation test was performed for the Argusville Formation, (near the boundary of the Brenna Formation) we considered this value in our analysis of the Brenna Formation and used the same value for the Argusville Formation.

**Graph 2. Coefficient of Consolidation Curves**

**C. Recommendations**

**C.1. Design and Construction Discussion**

**C.1.a. Settlement and Stability**

The major geotechnical consideration for this project is the potential for unfavorable approach embankment settlement relative to the bridge structure. We originally calculated that if construction proceeded without settlement mitigation, the roadway adjacent to the bridge abutments could settle well over a foot, and risk precipitating a bearing capacity failure if not staged. We therefore completed our analysis assuming the approach abutments would be pre-loaded in advance of bridge construction.
Through discussion with KLJ, the City of Fargo and the NDDOT, the project team elected to use a combination of pre-load, or surcharge, and lightweight embankment fill (geofoam), augmented by wick drains penetrating the lacustrine soils in advance of fill placement to allow the soils to drain and cause settlement to occur within a reasonable amount of time.

A geofoam section includes stacked blocks typically 3 feet thick and arranged to create a stepped versus curvilinear surface, a polyethylene cover to guard against deterioration from exposure to oil-based products, and a soil cover up to approximately 4 feet thick (including pavement). Approximately 35,000 cubic yards of geofoam were used in the stabilization of the Veterans Boulevard crossing of I-94.

The project approach will be as follows:

- Strip subgrades beneath the entire embankment
- Place drainage sand
- Install wick drains
- Place roadway embankment fill to elevation 915 and wait 2 weeks
- Place a first stage of surcharge fill 4 feet thick to a maximum elevation of 919 feet and wait 2 weeks (stage 1 surcharge fill)
- Place a second stage of surcharge fill 4 feet thick to a maximum elevation of 923 feet and wait 2 weeks (stage 2 surcharge fill)
- Place the last 6 feet of surcharge fill to a maximum elevation of 929 feet near the bridge abutments (stage 3 surcharge fill)
- Settlement monitoring and construction delay of at least 180 days
- Surcharge and earth fill removal to an elevation of 915 feet
- Placement of up to 15 feet of foam fill and earth cover to a maximum elevation of 930 feet
- Placement of embankment fill to account for a minimum 4 feet of cover over the geofoam
- Pavement subgrade preparation and paving

C.1.b. Analytical Demonstrations

We utilized SLOPE/W, SEEP/W and SIGMA/W, finite element programs by GeoStudio, to evaluate surcharge geometry and extent, and wick drain geometry, and also qualified the results in terms not only of total, differential and time-rate of settlement, but also in terms of slope stability.
Stability

In communication with the design team, we developed a profile and section including geofoam placed between a base elevation of 915 feet and a maximum of 930 feet, resulting in an average thickness of earth fill equal to 8 feet being placed beneath the foam, and 4 feet of earth fill and pavement (combined) being placed atop the foam. The earth fill was further assumed placed atop 2 feet of drainage sand into which the wick drains extend. Placement of the surcharge from a profile perspective is shown in Figures 2 and 4. Placement of the geofoam after construction is completed is shown in Figures 3 and 5.

Figure 2. West Profile Details, West End of Profile, Surcharge Stage
Figure 3. West Profile Details, West End of Profile, Post-Surcharge Placement of Geofoam

Figure 4. West Profile Details, East End of Profile, Surcharge Stage
Additional analytical graphics are included in Appendix C. The geotechnical parameters used in our analyses are published on the first graphic in Appendix C, which is followed by a graphic reduction of the triaxial shear strength test results for the project.

As discussed in Section C.1.a, we simulated embankment construction as occurring in a single stage for placement to an elevation of 915 feet. For the surcharge we modeled placement in 4-foot stages, except for the last stage, which will be up to 6 feet thick. It will be necessary to wait about two weeks between each stage of embankment and surcharge fill in order for excess pore water pressures to dissipate so that a bearing capacity failure is not induced. This will result in four stages of filling to reach design grades for the surcharge period.

From the base of preparatory excavations, our analytical models reflected:

- A 2-foot thick sand drainage fill between elevations 905 and 907 feet
- Earth fill from the top of the drainage layer upwards, between elevations 907 and 915
- Geofoam fill from elevation 915 to no less than 4 feet below the proposed pavement surface
- An effective “internal” foam prism with 3:1 (horizontal:vertical) slopes
- An “external” earth fill cap with 4:1 slopes
The profile shown in Figures 2 to 5 was used to evaluate stability along the bridge approaches. Figures 6 and 7 illustrate the analysis conditions and results while the surcharge is in place (Figure 6), and post-construction (Figure 7). As noted in both figures, the factor of safety against failure remains above 1.3 during construction and 1.5 post-construction, and increases with time post-construction. The total stress factor of safety (shown in Figure 8) is above 2 upon completion of surcharge placement.

Figure 6. Effective Stress Factor of Safety versus Time, Surcharge Period, West Profile
Figure 7. Effective Stress Factor of Safety versus Time, Post-Construction, West Profile

Figure 8. Total Stress Factor of Safety, Surcharge Period, West Profile
The cross section geometry from Station 126+48 was used to evaluate stability perpendicular to the roadway centerline as it appeared to represent the local maximum against the west abutment. Figures 9 and 10 illustrate the analysis conditions and results while the surcharge is in place. As noted in Figure 10, the factor of safety against failure remains above 1.5 and increases with time from surcharge placement.

Figure 9. West Section Details, East End of Profile, Surcharge Stage

Figure 10. Effective Stress Factor of Safety versus Time, Surcharge Period, West Cross Section
Figures 11 and 12 illustrate the analysis conditions and results after the completion of construction when the geofoam is in place. As noted in Figure 12, the factor of safety against failure remains above 2.0 and increases with time from surcharge placement. The total stress factor of safety (shown in Figure 13) is also above 2 at completion of the surcharge construction.

Figure 11. West Section Details, East End of Profile, Post-Surcharge Placement of Geofoam

![Figure 11](image1)

Figure 12. Effective Stress Factor of Safety versus Time, Post-Construction, West Cross Section

![Figure 12](image2)
Figure 13. Total Stress Factor of Safety, Surcharge Period, West Cross Section

Settlement
We analyzed surcharge-induced time-rate of settlement, and slope stability, based on the following assumptions and parameters:

- Wick drain installation will be performed by a mandrel, with the drains installed to the bottom of the Argusville Formation near elevation 830
- Wick drains will be placed in a triangular array at 8-foot centers
- The wick drains will be 100 millimeters (mm) wide by 3 to 4 mm thick (4 inches x 1/8-inch with an equivalent diameter of 0.175 foot)
- The coefficient of consolidation of the soils in the horizontal direction ($c_h$) will be twice the coefficient of consolidation in the vertical direction ($c_v$)
- The ratio of horizontal soil permeability between the undisturbed and disturbed zones around the drains will be about 2
- The diameter ratio of the disturbed zone to effective wick drain diameter will be 5 or less (this should be kept to a minimum by utilizing a mandrel with a minimum cross-sectional area)
We analyzed settlement at several different points throughout construction. Figures 14 and 15 show predicted settlement at the east end of the west embankment during the construction of the surcharge, through the surcharge period, removal of surcharge, placement of geofoam and construction of roadway. Settlement during the surcharge period is anticipated to be about 1 foot.

Figure 14. Time Rate of Settlement, West Profile Foundation Grade through construction

Figure 15. Time Rate of Settlement, West Cross Section Foundation Grade through construction
Provided that the wick drains, embankment and surcharge are constructed as detailed herein, we estimate that embankment construction will precipitate between 1 and 1 ½ feet of foundation compression (pertaining to overbuild quantities), and that the surcharge will precipitate approximately ½ foot of grade settlement (pertaining to overbuild elevations). We also anticipate that surcharge removal will cause 1 to 2 inches of rebound, though that movement is likely to cease within approximately one to two weeks of removal. With rebound complete, we estimate that longer-term settlements at the bridge abutments will be on the order of ½ inch, and that pile foundations will not be subject to drag loads.

C.2. Embankment and Surcharge Construction

C.2.a. Clearing and Grubbing
Clearing and grubbing should be performed below all areas that will receive new embankment fills in accordance with Specification 201.04 of the current North Dakota Department of Transportation (NDDOT) Standard Specifications for Road and Bridge Construction.

C.2.b. Treatment of Organic Soils
All vegetation and root zones should be removed from below all areas that will receive new fills. Topsoil and organic soil should be removed from within 2 vertical feet below the design pavement subgrade elevation. These removals should be performed from the areas located between the grading points of intersection (PI's) and their 1 horizontal to 1 vertical (1H:1V) oversize areas. Where topsoil and organic soil are encountered at depths greater than 2 feet below the design pavement subgrade elevation, they may be left in place provided they are stable enough to support overlying compaction. Beyond these oversize areas, removals are not required.

Organic soils that are removed should not be reused as embankment fill but instead stockpiled for reuse as dressing on the approach embankment slopes.

C.2.c. Treatment of Existing Pavements
Existing pavements should be removed from below all new pavement areas. Following the removal of the pavements the remaining roadway surface should be scarified to a depth of at least 6 inches and recompacted to the specified density. Where the design pavement subgrade elevation is more than 3 feet above the existing roadway surface, this scarification is not required.
C.2.d. Subgrade Preparation

After vegetation and topsoils have been removed, we recommend 12 inches of subgrade preparation in cut areas (if any), and in fill areas where less than 18 inches of fill will be placed. These subgrade preparations need only be performed below the proposed pavements and the 1H:1V oversize areas extending beyond the grading PI's. In fill areas where more than 18 inches of fill will be placed, it is not necessary to perform subgrade preparation beyond topsoil stripping. Subgrade preparation should comply with NDDOT Specification 230.04D (Type A).

Compaction control for subgrade preparation should be in accordance with AASHTO T-99 (standard Proctor) and Specification 203.04 E.2b Type A.

If unstable soils are present below the topsoil, scarification and drying or overexcavation and replacement of the unsuitable soils could be considered.

C.2.e. Subexcavation

Based on the conditions encountered in our borings, we did not encounter specific areas that would require subexcavation of mineral (non-organic) soils. Should localized soft or weak soils be encountered during subgrade preparation, however, we recommend those soils be removed as necessary, in accordance with NDDOT Specification 203.04 C.

C.2.f. Excavation Support

Based on the borings, we anticipate on-site soils in excavations will consist of predominantly native fat clay. These soils are typically considered Type B Soil under OSHA (Occupational Safety and Health Administration) guidelines. Where groundwater is observed to be freely seeping from the clay sidewalls, or where clays’ unconfined compressive strength (pocket penetrometer resistance) 0.5 tons per square foot (tsf) or less, they will be Type C soils. OSHA guidelines indicate unsupported excavations in Type B soils should have a gradient no steeper than 1:1 (horizontal:vertical); in Type C soils, no steeper than 1.5:1. Slopes constructed in this manner may still exhibit surface sloughing. OSHA requires an engineer to evaluate slopes or excavations over 20 feet in depth.

An OSHA-approved qualified person should review the soil classification in the field. Excavations must comply with the requirements of OSHA 29 CFR, Part 1926, Subpart P, “Excavations and Trenches.” This document states excavation safety is the responsibility of the contractor. The project specifications should reference these OSHA requirements.
C.2.g. Dewatering

We recommend removing groundwater from the excavations. Sumps and pumps can be considered for excavations in low-permeability clay-rich soils, or where groundwater can be drawn down 2 feet below the bottoms of excavations in more permeable sands. In large excavations, or where groundwater must be drawn down more than 2 feet, a well contractor should review our logs to determine if wells are required, how many will be required, and to what depths they will need to be installed.

C.2.h. Vertical Wick Drain Drainage Layer

A layer of drainage sand with a nominal thickness of 1 1/2 to 2 feet should be placed over the areas that will be treated with vertical wick drains. As consolidation occurs, the excess porewater will transmit upward through the vertical wick drains to the surface. The vertical wick drains will terminate in this sand layer, which will act as a medium through which porewater will drain from the embankment. Around the perimeter of the sand layer, drainage outlets consisting of 6-inch diameter perforated pipes should be placed, with their inverts located at least ½-foot below the bottom of the sand layer elevation. These pipes should be placed to divert drainage away from the embankment.

Settlement will decrease with distance measured away from the embankment centerline. We have calculated that this decrease will occur at an average rate of 1 to 1 ½ percent within the vertical wick drain areas (measured away from and perpendicular to the embankment centerline). If the clay subgrades below the drainage layer are not crowned, the settlement of the embankment will cause a “dishing” effect within the embankment, creating a bathtub below the roadway.

If the drainage sand will be designed to drain water away to both sides of the embankment, we recommend crowning the clay subgrade (below the drainage sand layer) at a gradient of at least 1 ½ percent. If the drainage sand will drain water to only one side of the embankment, we recommend the clay subgrade be graded to provide a minimum 1 ½ percent rise from the drain pipe to the embankment centerline, then continuing upward beyond the centerline at ½ percent or higher.

Prior to the placement of the sand, the clay subgrade immediately below the sand should be smoothed. The sand layer thickness may be reduced to 1 foot, where necessary, to accommodate crowning of the underlying clay subgrades.

Wick drain installation will create a surface with numerous depressions. We therefore recommend initially placing only between 50 and 75 percent of the sand prior to wick drain installation, and placing the balance after wick drain installation. Specified compaction of the sand layer is not required. We
recommend the sand, however, be placed at a moisture content within +/- 3 percentage points of the optimum moisture (standard Proctor).

The drainage sand should meet the following requirements:

- Mineral soil free of organic or foreign materials
- Free of rocks smaller than 2 inches in its longest dimension
- Containing less than 50 percent of the particles by weight passing a #40 sieve
- Containing less than 5 percent of the particles by weight passing a #200 sieve

This drainage layer sand will need to be imported.

**C.2.i. Vertical Wick Drains**

In order to achieve the majority of the predicted settlement within the proposed surcharge period, we recommend that vertical wick drains be placed in 8-foot triangular spacing beneath the entirety of the area receiving surcharge fill and extending for 40 feet outwards from the toe of the surcharge.

The vertical wick drains will need to be installed to the bottom, or to within a few feet of the bottom, of the Argusville Formation. The Argusville Formation terminated at approximate elevation 830 feet. For quantity calculation, the vertical wick drain lengths should be measured from the top of the sand drainage layer to elevation 830 feet.

**C.2.j. Instrumentation Types and Locations**

Instrumentation will be implemented to monitor performance of bridge approach embankments and engineered subgrades, using automated pore water pressure and settlement measurements (via Vibrating Wire (VW) Piezometers and ShapeArrays, respectively), as well as manual confirmation of settlement (via settlement plates).

The purpose of the VW piezometers is to monitor excess pore water pressures in the soil to confirm primary consolidation is complete, the rate has stabilized, and that clay has gained enough shear strength to allow staged construction. The ShapeArray is used to monitor settlement (downward elevation change) and shape of deformation, to confirm primary consolidation is complete, confirm rate has stabilized, isolate zones of instability, and corroborate VW piezometer data to allow staged construction. Settlement plates will be manually surveyed at specified intervals, to measure settlement (downward elevation change) near the top of the constructed embankment, to confirm the rate has
stabilized to permit road construction and corroborate ShapeArray and VW piezometer measurements. Automated instrumentation will be used throughout embankment construction and could remain in place long afterward, at the discretion of the Owner, to monitor long-term embankment performance.

**Vibrating Wire Piezometers**

We recommend installing multi-level (nested) VW piezometers below both embankments to monitor pore water pressures during and following the embankment fill placement. Evaluating pore water pressures should be performed for (1) monitoring for slope stability concerns and clearance to place the next stage of material, and (2) monitoring of the dissipation of pore water pressures over time to correlate to the settlements that will be occurring. The VW piezometers should be connected to an on-site automated data logger.

Vibrating wire piezometers shall be standard, groutable, unvented, with stainless steel filters, and shall be Model 4500S as manufactured by Geokon, Inc., Model VW2100 as manufactured by RST Instruments, Ltd., Part number 52611030 as manufactured by Durham Geo Enterprises, Inc., or approved equal. The vibrating wire piezometer manufacturer shall have ISO 9001:208 certification.

The VW piezometers shall meet the following minimum requirements:

<table>
<thead>
<tr>
<th>Requirement</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Range</td>
<td>350 kPa (50 psi)</td>
</tr>
<tr>
<td>Over Range</td>
<td>2 x rated pressure</td>
</tr>
<tr>
<td>Resolution</td>
<td>0.025% F.S.</td>
</tr>
<tr>
<td>Accuracy</td>
<td>+/- 0.1% F.S.</td>
</tr>
<tr>
<td>Nonlinearity</td>
<td>&lt; 0.5% F.S.</td>
</tr>
<tr>
<td>Temperature Range</td>
<td>-20°C to +80°C</td>
</tr>
<tr>
<td>Length x Diameter</td>
<td>133 x 19.1 mm (maximum size)</td>
</tr>
</tbody>
</table>

Vibrating wire piezometer communication cables shall consist of shielded cable with four 22-gauge tinned-copper conductors within a polyurethane jacket, as recommended, supplied, and connected to the sensor by the vibrating wire tiltmeter manufacturer.

The VW piezometers should be mounted on a tremie pipe to be grouted in place at the required elevations using a grout mix designed for soft soils. Tremie pipe shall consist of PVC (1-inch minimum diameter) with factory threaded flush joints, and will be used to both grout the boreholes from the bottom up and set the VW piezometers tip-down (adhered to the outside of the tremie pipe) at their specified depths. In order to support the “fully grouted” method for vibrating wire piezometer installation, cement-bentonite grout shall be composed of a mixture of water, Portland cement, and bentonite. Based on the “Grout for Soft Soils” details in Table 1, of P.E. Mikkelson and G.E. Green’s
FMGM paper entitled “Piezometers in Fully Grouted Boreholes”, the grout mixture shall meet the following minimum requirements:

<table>
<thead>
<tr>
<th>Materials</th>
<th>Weight</th>
<th>Ratio by Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td>75 gallons</td>
<td>6.6</td>
</tr>
<tr>
<td>Portland Cement</td>
<td>94 pounds (1 sack)</td>
<td>1</td>
</tr>
<tr>
<td>Bentonite</td>
<td>39 pounds (as required)</td>
<td>0.4</td>
</tr>
</tbody>
</table>

Notes: When placed, cement-bentonite grout shall be in colloidal form attained by high speed mechanical mixing. Borehole grouted from bottom, filled to top.

The VW piezometers (and SAA’s discussed below) should be connected to a monitoring station, consisting of a pole-mounted assembly of hardware for automated embankment monitoring. Each monitoring station shall consist of three major monitoring-related products (and related appurtenances): a solar panel, a VW datalogger, and a THREAD.

- **Solar Panel**: The ready-to-mount solar panel, supplied by the THREAD manufacturer, shall utilize renewable energy to power the THREAD (which will in turn power the VW datalogger) at each monitoring station.

- **VW Datalogger**: The VW datalogger, manufactured and quality tested through ISO 9001:2015, shall comprise a self-contained, 4-channel logger, connecting to the vibrating wire instrumentation installed beneath the embankment via their communication cable leads routed to the monitoring station. The VW datalogger shall be manufacturer-modified to connect to, and immediately transmit vibrating wire measurements to, the THREAD. The VW datalogger must meet the following minimum requirements:

<table>
<thead>
<tr>
<th>Measurement Accuracy</th>
<th>±0.05% F.S. (450-4000 Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measurement Resolution</td>
<td>1 part in 20,000</td>
</tr>
<tr>
<td>Program Memory</td>
<td>24K Flash</td>
</tr>
<tr>
<td>Data Memory</td>
<td>320K EEPROM</td>
</tr>
<tr>
<td>Temperature Range</td>
<td>-30°C to +50°C</td>
</tr>
<tr>
<td>L x W x H</td>
<td>260 x 160 x 91 mm</td>
</tr>
<tr>
<td>VW Channels</td>
<td>4</td>
</tr>
<tr>
<td>Power</td>
<td>Direct connect to THREAD</td>
</tr>
<tr>
<td>Notes:</td>
<td>Must be customized by manufacturer to connect to THREAD</td>
</tr>
</tbody>
</table>

- **THREAD**: The THREAD shall connect to the VW datalogger—and power the datalogger—while also automatically collecting the vibrating wire measurements immediately. Utilizing its second port, the THREAD shall also connect directly to the ShapeArray, and automatically collect those measurements immediately as well. Next, the THREAD, through its wireless/cellular antennas, shall transmit instrumentation data through a wireless mesh and/or cellular network to the...
cloud, whereby data is automatically, immediately, and remotely reduced, presented, and accessible via a secure, web interface hosted by Sensemetrics and managed by the Geotechnical Engineer.

With the monitoring station set up, the Sensemetrics cloud-based platform shall automatically retrieve the measurements collected on site by the VW Datalogger and THREAD, and immediately populate the online interface with near-real time data. The browser-based software interface shall be configured to communicate with each THREAD and calibrated according to sensor calibration certificates, baseline readings, and measured sensor elevations, such that measurements will be automatically reduced and presented in graphical and tabular form.

**Shape Arrays**

We recommend the installation of both shape arrays and settlement plates along the embankment for redundancy (in the event one of the systems is damaged). The shape arrays should also be connected to an on-site automated data logger.

ShapeArrays (SAA) shall consist of rigid segments separated by flexible joints comprised of triaxial MEMS gravity sensors. The SAA manufacturer shall have ISO 9001:208 certification.

The SAA’s shall meet the following minimum requirements:

<table>
<thead>
<tr>
<th>Specification</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Segment Length</td>
<td>500 mm</td>
</tr>
<tr>
<td>Joint diameter</td>
<td>19 mm</td>
</tr>
<tr>
<td>Waterproof</td>
<td>200 kPa (200 m water)</td>
</tr>
<tr>
<td>Temperature Range</td>
<td>-35°C to +60°C</td>
</tr>
<tr>
<td>Annular range of MEMS sensors</td>
<td>± 360°</td>
</tr>
<tr>
<td>Deformation Accuracy</td>
<td>±1.5 mm for 32m SAA</td>
</tr>
<tr>
<td>Long-term reliability MTBF</td>
<td>38 years for 32 m SAA</td>
</tr>
<tr>
<td>Notes:</td>
<td>Horizontal Installation, housed within 2-inch, Schedule 80, PVC conduit</td>
</tr>
</tbody>
</table>

SAA communication cables shall be shielded with waterproof jacket, as manufactured and supplied by the SAA manufacturer. SAA installation kit shall be supplied by the SAA manufacturer, including adapters for 2-inch conduit installation, and reference end assembly (near-end) to permit secure mounting and survey confirmation.

Conduit that the SAA will be inserted horizontally within shall consist of 2-inch, Schedule 80, gray PVC electrical conduit with bell ends. End caps shall consist of gray PVC, designed to cap 2-inch, Schedule 80 gray PVC electrical conduit. Conduit, including end caps, shall be connected using PVC primer and
cement (or approved combination) manufactured for gray PVC electrical conduit, and allowed proper set
times prior to SAA installation. Conduit shall be placed according to the Drawings and manufacturer
recommendations.

The SAA’s should be installed at the bottom of the drainage sand layer over a level spot, from the east to
west grading PI’s of the northbound and southbound roadways. The elevation of the SAA’s should be
surveyed prior to placing backfill over it.

Settlement Plates
Settlement plates should be placed near the grading points of intersection (PI’s) for both embankments
at the locations shown in the following section. The settlement plates should be installed below the
bottom of the drainage sand layer.

The settlement plates should consist of steel or wooden plates with a dimension of 2 feet square, fitted
with a floor flange able to fit a ¾-inch diameter steel pipe. Once set over a level spot, the top of the plate
should be staked into the ground and surveyed. Riser pipe in 3- to 6-foot section lengths should be fitted
to the settlement plate, with additional sections added with subsequent lifts of fill. A 2-inch diameter
PVC pipe should be placed around the riser pipe to protect the pipe and reduce friction along the sides of
the pipe. We recommend the PVC pipe extend a minimum of 3 vertical feet above grades at all times and
be painted and flagged to notify equipment operators of their presence. Survey measurements must be
taken of the steel riser pipe upon attaching each lead section in order to back-calculate the

The settlement plates should be surveyed according to the following schedule:

- At installation and with each additional section of steel riser
- At completion of each stage of fill and surcharge
- Immediately after the surcharge has reached finished grade
- 1, 2, 4, 7, 10, 14 and 21 days after surcharge installation, and bi-weekly to follow
- At monthly intervals only with the direction of the Geotechnical Engineer

Instrument Locations
Table 6, below, presents the recommended locations for the instrumentation.
Table 6. Summary of Instrumentation Locations

<table>
<thead>
<tr>
<th>Instrument</th>
<th>Location</th>
<th>Elevation/Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>West Embankment</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>VW Piezometers (2)</td>
<td>127+00 (CL)</td>
<td>Elevation 840’ and 880’</td>
</tr>
<tr>
<td>Shape Array (1)</td>
<td>(between grading PI’s right and left of CL)</td>
<td>Top of Drainage Sand</td>
</tr>
<tr>
<td>Settlement Plates (10)</td>
<td>126+00 (20’ Rt &amp; Lt)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>124+50 (20’ Rt &amp; Lt)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>123+00 (20’ Rt &amp; Lt)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>121+00 (20’ Rt &amp; Lt)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>119+00 (20’ Rt &amp; Lt)</td>
<td></td>
</tr>
<tr>
<td><strong>East Embankment</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>VW Piezometers (2)</td>
<td>130+65 (CL)</td>
<td>Elevation 840’ and 880’</td>
</tr>
<tr>
<td>Shape Array (1)</td>
<td>(between grading PI’s right and left of CL)</td>
<td>Top of Drainage Sand</td>
</tr>
<tr>
<td>Settlement Plates (10)</td>
<td>131+65 (20’ Rt &amp; Lt)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>133+15 (20’ Rt &amp; Lt)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>134+65 (20’ Rt &amp; Lt)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>136+50 (20’ Rt &amp; Lt)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>138+50 (20’ Rt &amp; Lt)</td>
<td></td>
</tr>
</tbody>
</table>

The VW sensors should be connected to an automated data logger capable of reading the data on an inputted schedule. The data logger shall be programmable to take readings at a specific time interval. The readings shall be stored within the data logger and capable of connecting to a laptop to download the data or to transmit the signal via modem. Appropriate interface cables shall be supplied with the data logger. A power source will be required for the data logger.

Buried wiring and tubing that will connect the VW sensors to the data logger and SAA’s must be flagged to ensure that the wires and tubing are not damaged after installation. The earthwork contractor should have the exact locations of the wiring and tubing surveyed, and have the flagging refreshed periodically. The wiring and tubing are typically irreparable once they are damaged.
Instrumentation Data Review
The readings from the VW piezometers, SAA’s and the settlement plates should be provided to the Geotechnical Engineer for review on a weekly basis, at a minimum, during the course of the surcharge. The VW settlement cell and settlement plate readings should be plotted on a semi-logarithmic graph, with settlement on the arithmetic scale and time on the logarithmic scale. The Geotechnical Engineer must compare the actual settlements to the predicted settlements at each of the instrument locations to determine when the surcharge may be removed. If the readings indicate the surcharging will not be complete within the allowable time frame, it may be necessary to add an additional stage of surcharge material.

The combined surcharge and wick drains will result in the overconsolidation of the upper Sherack soils, meaning that the surcharge and wick drains will cause those layers to experience more settlement than they would if the surcharge was not placed. If this “overconsolidation” is not taken into consideration, it may appear that the required settlements are completed when there is still remaining settlement that needs to occur within the Brenna and Argusville Formations. The target settlements for the surcharge must include this “overconsolidation” settlement.

The VW piezometer data should be monitored in conjunction with the settlement data to confirm the dissipation of porewater pressure. In the event excavations are planned adjacent the surcharge that may have questionable stability, this data may also be used to support any necessary stability calculations.

Pre-Installation Review
Prior to installation of the instrumentation, we recommend there be a meeting held between the earthwork contractor, surveyor, instrumentation installer, geotechnical engineer, and other necessary parties, to clarify roles, responsibilities and schedules.

C.2.k. Embankment Construction
After the drainage layer and instrumentation is placed, embankment construction to proceed to elevation 915 across the site. Then embankment construction will pause until after completion of the surcharge, removal of the surcharge and placement of geofoam. Above the geofoam, embankment will continue to the bottom of the proposed pavement section.

The placement and construction of the embankments should be performed in accordance with NDDOT Specifications 203.02 E.1 and 203.02 E.2b (Compaction Control, Type A). The maximum dry densities and optimum moisture contents of the embankment materials should be determined in accordance with the standard Proctor (AASHTO T99).
Embankment fill materials placed below the new pavements (including shoulders) and also within the areas extending out to a 1H: 1V slope down and away from the grading PI's should meet the following requirements:

- Mineral soil with an organic content of less than 3 percent relative to dry weight
- Free of rocks larger than 4 inches in its longest dimension where placed within the top 1 foot of the finished subgrade
- Classified in accordance with ASTM guidelines with a prefix letter of S, C or G (e.g. SP, SC, CH, GP, etc.), with exception to materials classified as CL-ML (silty clay) and SC-SM (silty clayey sand) which may only be used more than 4 vertical feet below the design pavement subgrade elevation
- Liquid limit (LL) ≤ 90

Materials that do not meet the above requirements may be placed outside of the 1H:1V areas (described above) provided they are able to be placed and compacted to the required grades per the project requirements.

**C.2.l. Surcharge**

Surcharge thickness should be measured relative to finished pavement surface elevations. The surcharge should be placed in lifts not exceeding 1 foot in loose thickness. The surcharge material should be compacted to achieve a moist unit weight on the order of 115 pcf. Compaction of the material to achieve this moist unit weight can likely be performed through repetitive wheel loading from equipment traffic. If the selected material cannot be compacted to this unit weight, the surcharge thickness should be adjusted accordingly to provide a surcharge pressure between 880 and 960 psf.

Perpendicular to the roadway, the full surcharge height should be extended to the top outside edges of the embankment, then graded down and away at 3:1 (horizontal:vertical) slopes or flatter. Parallel to the roadway and near the bridge abutment, the full surcharge height should be placed to the inside face of the bridge abutment, then graded down and away at a slope of 3:1 or flatter. Parallel to the roadway and away from the abutment, the full surcharge height should be placed to Station 118+20 on the west side of the bridge and to Station 138+00 on the east side and then be sloped down and away at a slope of 3:1 or flatter.

We estimate the surcharge will need to remain in place for approximately 6 months. The surcharge may not be removed until the monitoring results indicate that the required settlements are completed.
Surcharge materials placed over the embankments to accelerate settlements may consist of any on-site materials that are free of vegetation. The surcharge materials should consist of materials that are able to be placed and compacted to a moist unit weight of 115 pounds per cubic foot (pcf).

C.2.m. Geofoam

After removal of the surcharge to elevation 915, the surface of the earth should be covered with a nominal 6 inches of clean sand. The surface of the compacted fill upon which the sand is placed need not be particularly smooth; the thickness of the sand placed atop the earth fill can vary by 1 to 2 inches, as long as the surface of the sand can be made uniform for geofoam placement. The geofoam should then be placed in accordance with manufacturer’s recommendations, including the use of a geomembrane where recommended. The foam should be placed to an elevation that is no closer than 4 feet to top of the pavement elevations along the roadway centerline (it will be a little less than 4 feet at the edges of the roadway due to the crown across the road).

Along the road, as the embankment rises, we recommend 3 foot steps in the geofoam such that one side of the step will be about 7 feet below the centerline top of pavement grade and the other will be about 4 feet below it. Perpendicular to the roadway the outside edge of the geofoam at the bottom should be found by extending a line horizontally outward from the top of roadway at centerline to the back of curb, then vertically downward 4 feet, and then outward at a slope of 3H:1V to the intersection with elevation 915. The geofoam will then be placed from that point upward and inward so that the bottom outward corners of the blocks, when connected, create a 3H:1V sloped line upward (i.e. if the blocks are 3 feet high, the second row of geofoam will terminate nine feet inward from the bottom block).

C.2.n. Bridge Abutment Wall Backfill

We understand that the abutment wall design will include the placement of a granular backfill behind the abutment walls. The contractor may place this material during initial embankment construction or after the surcharge and overbuilt embankment has been removed from the abutment area. In order to use the soil design parameters associated with a granular material, the granular material should be placed within a zone behind the wall extending upward and outward from the base of the wall at a 1:2 (horizontal:vertical) slope. To allow for adequate compaction of this material we recommend the minimum width of sand layer at the base of the wall be 5 feet. Placement and compaction of the sand fill should be performed in accordance with NDDOT Specifications 203.02 E.1 and 203.02 E.2b (Compaction Control, Type A).
Granular material placed as bridge abutment backfill should meet the following requirements:

- Mineral soil free of organic or foreign materials
- Free of rocks larger than 4 inches in its longest dimension where placed within the top 1 foot of the finished subgrade
- Classified in accordance with ASTM guidelines with a prefix letter of S or G (e.g. SP, SC, GP, etc.)
- Plasticity index (PI) ≤ 15
- Containing less than 15 percent of the particles by weight passing a #200 sieve

The materials specified in this section will also need to be imported.

For lateral earth pressure design, abutment backfill can be assumed to have a moist unit weight of 125 pcf and a drained friction angle ($\phi^'$) of 34 degrees. These values do not include a factor of safety, and assume that drainage for the granular materials will be provided.

**C.3. Driven Piles**

**C.3.a. Pile Types**
We evaluated design requirements for a deep foundation system consisting of driven steel piling. We performed our analyses and calculations based on driven HP10x42 piles at the approach slabs, HP14x73 piles at the abutments, and HP14x102 piles at the pier.

**C.3.b. Calculation Method**
We used the computer program, UNIPILE®, to estimate the nominal geotechnical vertical compressive resistance of the piles. UNIPILE® is a static pile analysis software program. We used an effective stress analysis with estimated values of end bearing and skin friction parameters.

There are numerous methods of predicting the static resistance of piles based on the result of borings, and the results of the various methods often differ by a factor of two or more. Furthermore, measuring the nominal resistance of the pile during or after installation is also subject to variability. The measured resistance depends on the method used (e.g., dynamic formula, wave equation, high-strain dynamic testing or static load test) and the criteria used with each method.
Our scope of services did not include performing a wave equation analysis to evaluate drivability. We recommend requiring the contractor to submit wave equation analyses to analyze drivability of the proposed pile driving system.

C.3.c. Assumptions

We based the total unit weight inputs into UNIPILE® on laboratory testing results. We based the inputs for undrained shear strengths of fine-grained soils on laboratory testing results and correlated that to empirical values based on an average of the penetration resistances.

Based on the preliminary bridge layout, we estimated the bottom-of-pile-cap elevations as 922 feet and 906 feet for the abutments and the pier, respectively. We used a composite soil profile to develop the geotechnical resistance for the bridge.

C.3.d. Geotechnical Resistances and Estimated Lengths

The nominal geotechnical resistance required during driving \( R_n \) is obtained by dividing the factored load per pile \( (\Sigma \gamma Q_n) \) by the appropriate pile driving resistance factor \( \phi \) \( (R_n = \Sigma \gamma Q_n / \phi) \). The American Association of State Highway and Transportation Officials (AASHTO) recommends relating \( \phi \) to the degree of construction control. Table 7, below, summarizes AASHTO recommendations.

<table>
<thead>
<tr>
<th>Field Control Method</th>
<th>( \phi )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Variations of Engineering New Record (ENR) pile driving formula</td>
<td>0.10</td>
</tr>
<tr>
<td>FHWA-Modified Gates Formula</td>
<td>0.40</td>
</tr>
<tr>
<td>High-strain dynamic testing</td>
<td>0.65</td>
</tr>
</tbody>
</table>

\(^{A}\) Based on Table 10.5.5.2.3-1 of AASHTO LRFD Bridge Design Specifications 7th Edition with 2015 and 2016 Interim Revisions.

NDDOT practice is to use a resistance factor of 0.40 with a modified-ENR formula. Based on experience with projects of this size in the region and on our estimated pile lengths, we anticipate the shorter pile lengths from high-strain dynamic testing will not exceed the cost of performing high-strain dynamic testing.

As indicated above, we utilized UNIPILE® to estimate the nominal geotechnical vertical compressive resistance \( R_n \) for the HP14x73 H-piles. Appendix D includes results from UNIPILE® in a graphical format for the abutments and pier. The calculated resistances indicated on the graph are an estimate of driving
conditions. Using the equation \( R_n = \sum \gamma Q_n \phi \), the pile designer can plot the required values of \( R_n \) on the graphs provided to evaluate the required pile length.

**C.3.e. Downdrag**

If the project schedule and/or construction sequence does not allow the surcharging and wick drains discussed above to reduce the embankment settlement to less than about 1/2 inch after pile installation, the abutment piles will be subject to a downdrag load. We can be contacted to provide downdrag loads for a specific construction sequence if the methods discussed above are not used.

**C.3.f. Pile Settlement**

We anticipate total and differential deformation of the pile heads will be less than 1-inch under the assumed loads. The majority of deformation at the pile head is due to elastic shortening of the pile under the design loads.

**C.3.g. Pile Specifications**

The piles should meet the requirements of NDDOT Specification Section 840.01. The contractor should follow NDDOT Specification Section 622 for pile construction.

**C.3.h. Pile Driving System**

Using an under or oversized pile-driving hammer can be detrimental to the successful installation of piling. Prior to the driving system acceptance, we recommend performing a wave equation analysis modeling prospective contractors’ pile installation systems. The wave equation analysis is used to estimate probable driving stresses and pile penetration resistance based on the type of hammer proposed, the specified pile type/size and the site-specific material conditions which, when combined, help evaluate system suitability. Our firm can discuss the requirements and limitations of wave equation analyses and, if needed, perform them.

**C.3.i. Production Pile Monitoring**

A qualified bridge inspector should observe the installation of all piles. The inspector should document pertinent pile information such as lengths, elevations, and driving resistances, as well as note that the driving/length criteria has been achieved for each of the piles for satisfactory load-carrying capacities.

After the piles are driven to adequate bearing, we recommend inspecting the piles for damage and plumbness/batter. The geotechnical and structural engineers should review the load-carrying capability of any pile that is damaged during driving, or at an angle outside the plumbness or batter specification.
We recommend including contingencies in the project budget for additional piles and additional pile lengths below the predicted pile tip elevations.

C.4. Pavements

Subgrades should be prepared in accordance with Section C.2 above.

C.4.a. Subgrade Proof-Roll
Prior to placing aggregate base material, we recommend proof-rolling pavement subgrades to determine if the subgrade materials are loose, soft or weak, and in need of further stabilization, compaction, or subexcavation and recompaction or replacement.

C.4.b. Design Section
It is our understanding the typical section will be evaluated by others.

D. Procedures

D.1. Penetration Test Borings

We drilled the penetration test borings with a truck-mounted core and auger drill equipped with hollow-stem auger. We performed the borings in general accordance with ASTM D6151 taking penetration test samples at 2 1/2- or 5-foot intervals in general accordance to ASTM D1586. We collected thin-walled tube samples in general accordance with ASTM D1587 at selected depths. The boring logs show the actual sample intervals and corresponding depths. We also collected bulk samples of auger cuttings at selected locations for laboratory testing.

D.2. Exploration Logs

D.2.a. Log of Boring Sheets
Appendix A includes Log of Boring sheets for our penetration test borings. The logs identify and describe the penetrated geologic materials, and present the results of penetration resistance tests performed. The logs also present the results of laboratory tests performed on penetration test samples, and groundwater measurements. Appendix A also includes a Fence Diagram intended to provide a summarized cross-sectional view of the soil profile across the site.
We inferred strata boundaries from changes in the penetration test samples and the auger cuttings. Because we did not perform continuous sampling, the strata boundary depths are only approximate. The boundary depths likely vary away from the boring locations, and the boundaries themselves may occur as gradual rather than abrupt transitions.

D.2.b. Geologic Origins
We assigned geologic origins to the materials shown on the logs and referenced within this report, based on: (1) a review of the background information and reference documents cited above, (2) visual classification of the various geologic material samples retrieved during the course of our subsurface exploration, (3) penetration resistance testing performed for the project, (4) laboratory test results, and (5) available common knowledge of the geologic processes and environments that have impacted the site and surrounding area in the past.

D.3. Vibrating Wire Piezometer Installations

A nested set of three VW piezometers were installed in Boring ST-03. The VW piezometers were purchased from Geokon and had Model Number 4500S. The individual calibration sheets for the VW piezometers will be retained by Braun Intertec. The VW piezometers were rated for pressures of 350 kPa.

The VW piezometers were installed in the open borehole by attaching them to a 1-inch grout pipe at the prescribed depths, through which the borehole was backfilled with tremied grout. The 1-inch grout pipe and the VW piezometers were left in the borehole. The grout mix used to backfill the borehole was prepared per the specifications listed by Geokon within their instrument manual.

Upon completion of the installation, a steel protective casing was installed around the top of the borehole and was covered with a screw-on cap. The top of the cap was set below the top of the topsoil in order to allow mowers to run over the cover without risk of damage to the mower or cover. As of the date of this report, the VW piezometers remain in place.

D.4. Material Classification and Testing

D.4.a. Visual and Manual Classification
We visually and manually classified the geologic materials encountered based on ASTM D2488. When we performed laboratory classification tests, we used the results to classify the geologic materials in accordance with ASTM D2487. Appendix A includes a chart explaining the classification system we used.
D.4.b. Laboratory Testing

The exploration logs in Appendix A note most of the results of the laboratory tests performed on geologic material samples. The remaining laboratory test results are presented in Appendix B. We performed the tests in general accordance with ASTM or AASHTO procedures.

D.5. Groundwater Measurements

The drillers checked for groundwater while advancing the penetration test borings, and again after auger withdrawal. Some of the boreholes were advanced using mud rotary wash methods, which do not allow for the rechecking of groundwater upon completion. We also have referenced the on-site VW piezometers for groundwater level measurements.

E. Qualifications

E.1. Variations in Subsurface Conditions

E.1.a. Material Strata

We developed our evaluation, analyses and recommendations from a limited amount of site and subsurface information. It is not standard engineering practice to retrieve material samples from exploration locations continuously with depth. Therefore, we must infer strata boundaries and thicknesses to some extent. Strata boundaries may also be gradual transitions, and project planning should expect the strata to vary in depth, elevation and thickness, away from the exploration locations. Variations in subsurface conditions present between exploration locations may not be revealed until performing additional exploration work, or starting construction. If future activity for this project reveals any such variations, you should notify us so that we may reevaluate our recommendations. Such variations could increase construction costs, and we recommend including a contingency to accommodate them.

E.1.b. Groundwater Levels

We made groundwater measurements under the conditions reported herein and shown on the exploration logs, and interpreted in the text of this report. Note that the observation periods were relatively short, and project planning can expect groundwater levels to fluctuate in response to rainfall, flooding, irrigation, seasonal freezing and thawing, surface drainage modifications and other seasonal and annual factors.
E.2. Continuity of Professional Responsibility

E.2.a. Plan Review
We based this report on a limited amount of information, and we made a number of assumptions to help us develop our recommendations. We should be retained to review the geotechnical aspects of the designs and specifications. This review will allow us to evaluate whether we anticipated the design correctly, if any design changes affect the validity of our recommendations, and if the design and specifications correctly interpret and implement our recommendations.

E.2.b. Construction Observations and Testing
We recommend retaining us to perform the required observations and testing during construction as part of the ongoing geotechnical evaluation. This will allow us to correlate the subsurface conditions exposed during construction with those encountered by the borings and provide professional continuity from the design phase to the construction phase. If we do not perform observations and testing during construction, it becomes the responsibility of others to validate the assumption made during the preparation of this report and to accept the construction-related geotechnical engineer-of-record responsibilities.

E.3. Use of Report
This report is for the exclusive use of the addressed parties. Without written approval, we assume no responsibility to other parties regarding this report. Our evaluation, analyses and recommendations may not be appropriate for other parties or projects.

E.4. Standard of Care
In performing its services, Braun Intertec used that degree of care and skill ordinarily exercised under similar circumstances by reputable members of its profession currently practicing in the same locality. No warranty, express or implied, is made.
Appendix A

Bridge Layout
Excerpt, Approach Embankment Cross Section, vic. Station 126+50
64th Avenue Improvements
Boring Location Sketch
Log of Boring Sheets
Fence Diagram
Descriptive Terminology
Surcharge Cross Sections
Preliminary - Not for Construction
This document is preliminary and not for construction or implementation purposes.

64th Avenue South - 38th St S to 33rd St S

Plan & Profile

64th Avenue South
Sta 114+00 to 120+00 (PR64)
DENOTES APPROXIMATE LOCATION OF STANDARD PENETRATION TEST BORING
<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>WL</th>
<th>MC%</th>
<th>qpsf</th>
<th>Tests or Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>907.6</td>
<td>0.0</td>
<td>CH</td>
<td>FAT CLAY, with roots and organics, black, moist. (Topsoil)</td>
<td>8</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>905.1</td>
<td>2.5</td>
<td>CH</td>
<td>FAT CLAY, with Silt lenses and laminations, brown, moist, soft to rather soft. (Glacial Lake Deposit)</td>
<td>12</td>
<td>16</td>
<td>4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-with iron-staining at 5 feet.</td>
<td>10</td>
<td>27</td>
<td>2 1/2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-layer of SILTLY SAND at 7 1/2 feet.</td>
<td>6</td>
<td>27</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-brown and gray below 10 feet.</td>
<td>5</td>
<td>44</td>
<td>1 3/4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>889.6</td>
<td>18.0</td>
<td>CH</td>
<td>FAT CLAY, gray, wet, soft. (Glacial Lake Deposit)</td>
<td>3</td>
<td>51</td>
<td>1/2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>881.6</td>
<td>26.0</td>
<td></td>
<td>END OF BORING.</td>
<td>3</td>
<td>50</td>
<td>1/4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

END OF BORING.

Water not observed with 24 1/2 feet of hollow stem auger in the ground.

Boring immediately backfilled with bentonite grout.
### Description of Materials

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>WL</th>
<th>MC%</th>
<th>qptsf</th>
<th>Tests or Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>907.5</td>
<td>0.0</td>
<td>CH</td>
<td>FAT CLAY, with roots and organics, black, moist. (Topsoil)</td>
<td>11</td>
<td>23</td>
<td>1/2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>904.5</td>
<td>3.0</td>
<td>CH</td>
<td>FAT CLAY, brown, moist, rather soft to rather stiff. (Glacial Lake Deposit)</td>
<td>8</td>
<td>18</td>
<td>4 1/4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-with iron-staining at 7 1/2 feet.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-layer of SANDY SILT at 8 feet.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-brown and gray below 10 feet.</td>
<td>11</td>
<td>30</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>TW*</td>
<td>45</td>
<td>1 1/2</td>
<td>&quot;21 inch recovery. Qu=1750 psf WD=110 pcf, DD=76 pcf</td>
<td></td>
<td></td>
</tr>
<tr>
<td>889.5</td>
<td>18.0</td>
<td>CH</td>
<td>FAT CLAY, gray, wet, soft. (Glacial Lake Deposit)</td>
<td>3</td>
<td>48</td>
<td>1/2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Tests or Notes:**
- WD=106 pcf, DD=71 pcf
**LOCATION:** N430866.808, E2884836.803  See sketch.

### Description of Materials

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>875.5</td>
<td>32.0</td>
<td></td>
<td>FAT CLAY, gray, wet, soft. (Glacial Lake Deposit) (continued)</td>
</tr>
<tr>
<td>866.5</td>
<td>41.0</td>
<td></td>
<td>END OF BORING.</td>
</tr>
</tbody>
</table>

- Water not observed with 39 1/2 feet of hollow stem auger in the ground.
- Water not observed to cave-in depth of 32 feet immediately after withdrawal of auger.
- Boring immediately backfilled with bentonite grout.

**Tests or Notes**

<table>
<thead>
<tr>
<th>BPF</th>
<th>WL</th>
<th>MC</th>
<th>qP</th>
<th>TSF</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>52</td>
<td>1/4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>1/4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**LOCATION:** N430866.808, E2884836.803  See sketch.

### Description of Materials

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>875.5</td>
<td>32.0</td>
<td></td>
<td>FAT CLAY, gray, wet, soft. (Glacial Lake Deposit) (continued)</td>
</tr>
<tr>
<td>866.5</td>
<td>41.0</td>
<td></td>
<td>END OF BORING.</td>
</tr>
</tbody>
</table>

- Water not observed with 39 1/2 feet of hollow stem auger in the ground.
- Water not observed to cave-in depth of 32 feet immediately after withdrawal of auger.
- Boring immediately backfilled with bentonite grout.
<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>WL</th>
<th>MC</th>
<th>qpsf</th>
<th>Tests or Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>908.2</td>
<td>0.0</td>
<td>CH</td>
<td>FAT CLAY, with roots and organics, black, moist. (Topsoil)</td>
<td>9</td>
<td>22</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>906.2</td>
<td>2.0</td>
<td>CH</td>
<td>FAT CLAY, with Silt lenses and laminations, brown and gray, moist, soft to rather stiff. (Glacial Lake Deposit)</td>
<td>10</td>
<td>19</td>
<td>4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-with iron-staining at 5 feet.</td>
<td>8</td>
<td>31</td>
<td>2 1/4</td>
<td></td>
<td>Mud rotary methods used below 5 feet.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-FAT CLAY with SAND layer at 8 feet.</td>
<td>12</td>
<td>31</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>890.2</td>
<td>18.0</td>
<td>CH</td>
<td>FAT CLAY, gray, wet, very soft to soft. (Glacial Lake Deposit)</td>
<td>5</td>
<td>46</td>
<td>1 1/2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>TW*</td>
<td>56</td>
<td>1/2</td>
<td></td>
<td></td>
<td>*24 inch recovery. Qu=2590 psf WD=105 pcf, DD=67 pcf</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>54</td>
<td>1/2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>56</td>
<td>1/2</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
**BRAUN™ INTERTEC**

**BRAUN Project B1600465**
Geotechnical Evaluation
64th Ave S Roadway and Bridge
64th Avenue South and I-29
Fargo, North Dakota

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>WL</th>
<th>MC</th>
<th>qptsf</th>
<th>Tests or Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>876.2</td>
<td>32.0</td>
<td>FAT CLAY, gray, wet, very soft to soft.</td>
<td>(Glacial Lake Deposit) (continued)</td>
<td>1</td>
<td>56</td>
<td>1/2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TW*</td>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>57</td>
<td>1/2</td>
<td>*24 inch recovery. WD=105 pcf, DD=67 pcf</td>
<td></td>
</tr>
<tr>
<td>WH</td>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>55</td>
<td>1/2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WH</td>
<td></td>
<td></td>
<td></td>
<td>WH</td>
<td>54</td>
<td>1/4</td>
<td></td>
<td></td>
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<tr>
<td>WH</td>
<td></td>
<td></td>
<td></td>
<td>WH</td>
<td>56</td>
<td>1/4</td>
<td>WD=100 pcf, DD=65 pcf</td>
<td></td>
</tr>
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</table>

**LOG OF BORING**

**LOCATION:** N430877.124, E2885126.615  See sketch.

**BORING:** ST-03 (cont.)

**DRILLER:** K. Miller
**METHOD:** 3 1/4" HSA, Autohammer
**DATE:** 9/8/16
**SCALE:** 1" = 4'
### Description of Materials

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>WL</th>
<th>MC</th>
<th>qp</th>
<th>Tests or Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>844.2</td>
<td>64.0</td>
<td>FAT CLAY, gray, wet, very soft to soft. (Glacial Lake Deposit) <em>(continued)</em></td>
<td>WH</td>
<td>54</td>
<td>1/4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>837.2</td>
<td>71.0</td>
<td>END OF BORING. Water not observed with 4 1/2 feet of hollow stem auger in the ground. Water level not determined at termination due to the use of mud rotary drilling fluids. Vibrating Wire Piezometers installed at depths of 15 feet (Serial No. 1626716), 35 feet (Serial No. 1626715) and 60 feet (Serial No. 1626714). The borehole was backfilled with tremied grout (bentonite and cement) to the ground surface. An above grade protective cover was placed over the top of the borehole.</td>
<td>2</td>
<td>51</td>
<td>1/4</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
**LOG OF BORING**

**Braun Project B1600465**  
Geotechnical Evaluation  
64th Ave S Roadway and Bridge  
64th Avenue South and I-29  
Fargo, North Dakota  

**DRILLER:** J. Brooks  
**METHOD:** 3 1/4" HSA, Autohammer  
**DATE:** 9/9/16  
**SCALE:** 1" = 4'

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>WL</th>
<th>MC%</th>
<th>qp</th>
<th>tsf</th>
<th>Tests or Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>907.1</td>
<td>0.0</td>
<td>FILL</td>
<td>FILL: Fat Clay, trace Sand and Gravel, occasional roots, brown and black, moist.</td>
<td>4</td>
<td>19</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>904.1</td>
<td>3.0</td>
<td>CH</td>
<td>FAT CLAY, with Silt lenses and laminations, brown, moist, soft to medium.</td>
<td>7</td>
<td>29</td>
<td>2 1/4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(Glacial Lake Deposit)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- with iron-staining at 5 feet.</td>
<td>6</td>
<td>33</td>
<td>1 1/4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- 1 inch SAND layer at 7 1/2 feet.</td>
<td>6</td>
<td>39</td>
<td>1 3/4</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>904.1</td>
<td>3.0</td>
<td>TW*</td>
<td></td>
<td>7</td>
<td>46</td>
<td>1 1/4</td>
<td></td>
<td></td>
<td>LL=105, PL=25, PI=80</td>
</tr>
<tr>
<td>888.1</td>
<td>19.0</td>
<td>CH</td>
<td>FAT CLAY, gray, wet, very soft to rather soft.</td>
<td>3</td>
<td>53</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(Glacial Lake Deposit)</td>
<td>3</td>
<td>52</td>
<td>1/2</td>
<td></td>
<td></td>
<td>LL=96, PL=23, PI=73</td>
</tr>
<tr>
<td></td>
<td></td>
<td>TW*</td>
<td></td>
<td>3</td>
<td>53</td>
<td>1/2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>58</td>
<td>54</td>
<td>3/4</td>
<td></td>
<td></td>
<td>*24 inch recovery. LL=87, PL=33, PI=54 Qu=2090 psf</td>
</tr>
</tbody>
</table>

(See Descriptive Terminology sheet for explanation of abbreviations)
<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>WL</th>
<th>MC%</th>
<th>qp</th>
<th>tsf</th>
<th>Tests or Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>875.1</td>
<td>32.0</td>
<td></td>
<td>FAT CLAY, gray, wet, very soft to rather soft. (Glacial Lake Deposit) (continued)</td>
<td>3</td>
<td>54</td>
<td>3/4</td>
<td></td>
<td></td>
<td>WD=104 pcf, DD=66 pcf</td>
</tr>
<tr>
<td>3</td>
<td>56</td>
<td>1/2</td>
<td>LL=91, PL=21, PI=70</td>
<td>3</td>
<td>47</td>
<td>1/2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>53</td>
<td>1/4</td>
<td>Mud rotary methods used below 52 feet.</td>
<td>1</td>
<td>49</td>
<td>1/4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-trace Sand at 60 feet.</td>
<td>1</td>
<td>49</td>
<td>1/4</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*24 inch recovery. Qu=2560 psf WD=105 pcf, DD=67 pcf LL=116, PL=25, PI=91
<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>843.1</td>
<td>64.0</td>
<td>FAT CLAY, gray, wet, very soft to rather soft. (Glacial Lake Deposit) (continued)</td>
<td></td>
</tr>
<tr>
<td>829.1</td>
<td>78.0</td>
<td>CL</td>
<td>SANDY LEAN CLAY, a little Gravel, gray, moist, hard. (Glacial Till)</td>
</tr>
</tbody>
</table>

**Properties**

- **BPF**: 1, 2, 4, 50, 44, 48, 96
- **WL**: 63, 45, 33, 14, 12, 21, 24
- **MC%**: 1/4, 45, 1/2, 14, 12, 4, 21
- **Tests or Notes**: LL=70, PL=22, PI=48, WD=134 pcf, DD=118 pcf

**Additional Information**

- **Date**: 9/9/16
- **Scale**: 1" = 4'
- **Location**: N430884.627, E2885376.507 See sketch.
- **Driller**: J. Brooks
- **Method**: 3 1/4" HSA, Autohammer
<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>WL</th>
<th>MC</th>
<th>qp</th>
<th>tsf</th>
<th>Tests or Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>811.1</td>
<td>96.0</td>
<td></td>
<td>SANDY LEAN CLAY, a little Gravel, gray, moist, hard. (Glacial Till) (continued)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-SAND layer at 100 feet.</td>
<td></td>
<td>15</td>
<td>*</td>
<td></td>
<td></td>
<td>*117/9 inch.</td>
</tr>
<tr>
<td>801.5</td>
<td>105.6</td>
<td></td>
<td>-SANDY SILT layer at 105 feet.</td>
<td></td>
<td>12</td>
<td>*</td>
<td></td>
<td></td>
<td>*180/7 inch.</td>
</tr>
</tbody>
</table>

END OF BORING.

- Water not observed with 49 1/2 feet of hollow stem auger in the ground.
- Water level not determined at termination due to the use of mud rotary drilling fluids.
- Boring immediately backfilled with bentonite grout.
### Description of Materials

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>WL</th>
<th>MC%</th>
<th>qpsf</th>
<th>Tests or Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>909.3</td>
<td>0.0</td>
<td>FILL</td>
<td>FILL: Fat Clay, trace Sand and Gravel, occasional roots, brown, moist.</td>
<td>5</td>
<td>28</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0</td>
<td>4.0</td>
<td>FILL</td>
<td>FILL: Fat Clay, trace Sand and Gravel, occasional roots, brown, moist.</td>
<td>5</td>
<td>29</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>904.3</td>
<td>5.0</td>
<td>CH</td>
<td>FAT CLAY, with roots and organics, black, moist. (Buried Topsoil)</td>
<td>10</td>
<td>24</td>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>FAT CLAY, with Silt lenses and laminations, brown, moist, soft to rather stiff.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(Glacial Lake Deposit)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-with iron-staining at 7 1/2 feet.</td>
<td>7</td>
<td>33</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-SANDY SILT layer at 11 feet.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-brown and gray below 121/2 feet.</td>
<td>4</td>
<td>46</td>
<td>1 1/4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>TW*</td>
<td>37</td>
<td></td>
<td></td>
<td></td>
<td>&quot;21 inch recovery. Qu=1400 psf, WD=114 pcf, DD=83 pcf</td>
</tr>
<tr>
<td>891.3</td>
<td>18.0</td>
<td>CH</td>
<td>FAT CLAY, gray, wet, very soft to soft. (Glacial Lake Deposit)</td>
<td>3</td>
<td>52</td>
<td>3/4</td>
<td></td>
<td>LL=101, PL=22, PI=79</td>
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<td></td>
<td></td>
<td></td>
<td>TW*</td>
<td>55</td>
<td></td>
<td>1/2</td>
<td></td>
<td>&quot;24 inch recovery. WD=106 pcf, DD=68 pcf</td>
</tr>
</tbody>
</table>

| Method: 3 1/4" HSA, Autohammer | Scale: 1" = 4' | Date: 9/6/16

**LOCATION:** N430889.974, E2885701.498 See sketch.

**LOG OF BORING (See Descriptive Terminology sheet for explanation of abbreviations)**

**METHOD:**
- **BORING:**
  - Braun Project B1600465
  - Geotechnical Evaluation
  - 64th Ave S Roadway and Bridge
  - 64th Avenue South and I-29
  - Fargo, North Dakota

**DATE:** 9/6/16

**SCALE:** 1" = 4'

**Tests or Notes:**
- TW* 24 inch recovery.
- Qu=1400 psf, WD=114 pcf, DD=83 pcf
- LL=101, PL=22, PI=79
- WD=106 pcf, DD=68 pcf
**ST-05 (cont.)**

**LOCATION:** N430889.974, E2885701.498  See sketch.

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>WL</th>
<th>MC%</th>
<th>qp</th>
<th>Tests or Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>877.3</td>
<td>32.0</td>
<td></td>
<td>FAT CLAY, gray, wet, very soft to soft. <em>(Glacial Lake Deposit) (continued)</em></td>
<td>3</td>
<td>55</td>
<td>1/4</td>
<td>LL=93, PL=22, PI=71</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>51</td>
<td>1/4</td>
<td>WD=106 pcf, DD=70 pcf</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-with Silt lenses and laminations at 50 feet.</td>
<td>1</td>
<td>48</td>
<td>1/4</td>
<td>LL=65, PL=20, PI=45</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-trace Sand at 61 feet.</td>
<td>2</td>
<td>46</td>
<td>3/4</td>
<td>LL=116, PL=25, PI=91</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>55</td>
<td>1/2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>TW*</td>
<td>53</td>
<td></td>
<td>*24 inch recovery. LL=85, PL=25, PI=60 Qu=2780 psf, WD=106 pcf, DD=69 pcf</td>
<td>Mud rotary</td>
</tr>
</tbody>
</table>

(See Descriptive Terminology sheet for explanation of abbreviations)
**LOG OF BORING**

**Braun Project B1600465**  
Geotechnical Evaluation  
64th Ave S Roadway and Bridge  
64th Avenue South and I-29  
Fargo, North Dakota

**DRILLER:** K. Miller  
**METHOD:** 3 1/4” HSA, Autohammer  
**DATE:** 9/6/16  
**SCALE:** 1” = 4'

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>WL</th>
<th>MC %</th>
<th>qp</th>
<th>Tests or Notes</th>
</tr>
</thead>
</table>
| 845.3      | 64.0       |        | FAT CLAY, gray, wet, very soft to soft.  
(Glacial Lake Deposit) (continued)  
-trace Gravel at 65 feet. |     |     |     | 3  | 32 | 1/2 | methods used below 61 feet. |
| 830.3      | 79.0       | CL     | -a little Sand at 75 feet. |     | 1   | 52   | 1/4 | LL=75, PL=22, PI=53 |
| 76         |            |        | SANDY LEAN CLAY, a little Gravel, gray, moist, rather stiff to hard.  
(Glacial Till) | 12+  |     |     | 14  | *   | *No recovery. |
|            |            |        |                           |     |     |     | 12  | 4.5+ | *100/10 inch. |
|            |            |        |                           |     |     |     | 76  | 17  | WD=127 pcf, |

(See Descriptive Terminology sheet for explanation of abbreviations)
Braun Project B1600465  
Geotechnical Evaluation  
64th Ave S Roadway and Bridge  
64th Avenue South and I-29  
Fargo, North Dakota

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>WL</th>
<th>MC %</th>
<th>qpsf</th>
<th>Tests or Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>813.3</td>
<td>96.0</td>
<td></td>
<td>SANDY LEAN CLAY, a little Gravel, gray, moist, rather stiff to hard. (Glacial Till) (continued)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>DD=109 pcf</td>
</tr>
<tr>
<td>808.7</td>
<td>100.6</td>
<td></td>
<td>END OF BORING.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Water not observed with 59 1/2 feet of hollow stem auger in the ground.

Water level not determined at termination due to the use of mud rotary drilling fluids.

Boring immediately backfilled with bentonite grout.
**Braun Project B1600465**  
Geotechnical Evaluation  
64th Ave S Roadway and Bridge  
64th Avenue South and I-29  
Fargo, North Dakota  

**BORING:** ST-06  
**LOCATION:** N430893.496, E2885916.433  
See sketch.  

**DRILLER:** K. Miller  
**METHOD:** 3 1/4" HSA, Autohammer  
**DATE:** 9/1/16  
**SCALE:** 1" = 4'  

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials (Soil-ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)</th>
<th>BPF</th>
<th>WL</th>
<th>MC%</th>
<th>qptsf</th>
<th>Tests or Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>910.8</td>
<td>0.0</td>
<td>FILL</td>
<td>FILL: Fat Clay, trace Sand and Gravel, brown, moist.</td>
<td>10</td>
<td>19</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| 908.3      | 2.5        | CH     | FAT CLAY, with roots and organics, black, moist.  
(Buried Topsoil)                     | 10  | 27 |      |       |               |
| 906.8      | 4.0        | CH     | FAT CLAY, with Silt lenses and laminations, brown, moist, rather soft to rather stiff.  
(Glacial Lake Deposit)                | 12  | 23 | 3 1/2 |      |               |
|            |            |        | -with iron-staining at 7 1/2 feet.                                             | 6   | 33 | 2 1/2 |      |               |
|            |            |        | -SANDY SILT layer at 10 feet.                                                   | 6   | 29 | 1 1/4 |      |               |
|            |            |        | -brown and gray below 12 1/2 feet.                                             | 6   | 41 | 2     |      |               |
| 887.8      | 23.0       | CH     | FAT CLAY, gray, wet, soft to rather soft.  
(Glacial Lake Deposit)                | 3   | 50 | 1/2   |      |               |

**TW**  
56 1  
*24 inch recovery.  
Qu=2470 psf,  
WD=105 pcf,  
DD=67 pcf
**Boring:** ST-06 (cont.)

**Location:** N430893.496, E2885916.433  See sketch.

**Driller:** K. Miller  
**Method:** 3 1/4" HSA, Autohammer  
**Date:** 9/1/16  
**Scale:** 1" = 4'

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>WL</th>
<th>MC %</th>
<th>qpsf</th>
<th>Tests or Notes</th>
</tr>
</thead>
</table>
| 878.8      | 32.0       |        | FAT CLAY, gray, wet, soft to rather soft.  
(Glacial Lake Deposit) (continued) |     |    |      |      |               |
|            |            | TW*    | *24 inch recovery.  
Qu=2760 psf,  
WD=105 pcf,  
DD=67 pcf |     |    |      |      |               |
|            |            | 3      |                           | 57  | 1/2|      |      |               |
|            |            | 3      |                           | 49  | 1/2|      |      |               |
|            |            | 2      |                           | 42  | 1/2|      |      |               |
|            |            | 2      |                           | 53  | 1/4|      |      | WD=109 pcf,  
DD=71 pcf |
|            |            | 2      |                           | 47  | 1/2|      |      |               |
|            |            | 4      |                           | 47  | 1/4|      |      | -trace Sand below 60 feet. |

(See Descriptive Terminology sheet for explanation of abbreviations)
**Location:** N43°08'93.496, E28°55'916.433, See sketch.

**Description of Materials**

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>WL</th>
<th>MC %</th>
<th>qpsf</th>
<th>Tests or Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>846.8</td>
<td>64.0</td>
<td></td>
<td>FAT CLAY, gray, wet, soft to rather soft. (Glacial Lake Deposit) (continued)</td>
<td>3</td>
<td>46</td>
<td>1/4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>839.8</td>
<td>71.0</td>
<td></td>
<td>END OF BORING.</td>
<td>4</td>
<td>42</td>
<td>1/2</td>
<td></td>
<td>WD=111 pcf, DD=78 pcf</td>
</tr>
</tbody>
</table>

**Notes:**

- Water not observed with 69 1/2 feet of hollow stem auger in the ground.
- Boring immediately backfilled with bentonite grout.
**LOG OF BORING**

**Braun Project B1600465**  
Geotechnical Evaluation  
64th Ave S Roadway and Bridge  
64th Avenue South and I-29  
Fargo, North Dakota

**DRILLER:** K. Miller  
**METHOD:** 3 1/4" HSA, Autohammer  
**DATE:** 9/16  
**SCALE:** 1" = 4'

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>MC%</th>
<th>qpsf</th>
<th>Tests or Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>910.8</td>
<td>0.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>908.8</td>
<td>2.0</td>
<td>FILL</td>
<td>FILL: Fat Clay, a little Sand and Gravel, brown, moist.</td>
<td>12</td>
<td>20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>906.3</td>
<td>4.5</td>
<td>CH</td>
<td>FAT CLAY, with roots and organics, black, moist. (Buried Topsoil)</td>
<td>11</td>
<td>25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>892.8</td>
<td>18.0</td>
<td>CH</td>
<td>FAT CLAY, with Silt lenses and laminations, brown, moist, medium. (Glacial Lake Deposit)</td>
<td>6</td>
<td>20</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-with iron-staining below 7 1/2 feet.</td>
<td>7</td>
<td>28</td>
<td>2 1/4</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-brown and gray without iron-staining below 12 feet.</td>
<td>TW*</td>
<td>46</td>
<td>1 1/2</td>
<td>*24 inch recovery. Qu=1420 psf, WD=110 pcf, DD=75 pcf</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-SANDY SILT layer at 15 feet.</td>
<td>6</td>
<td>38</td>
<td>3/4</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>CH</td>
<td>FAT CLAY, gray, wet, soft to rather soft. (Glacial Lake Deposit)</td>
<td>4</td>
<td>51</td>
<td>1/2</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4</td>
<td>49</td>
<td>1/2</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4</td>
<td>50</td>
<td>1/2</td>
<td>WD=104 pcf, DD=75 pcf</td>
</tr>
</tbody>
</table>
### Description of Materials

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>878.8</td>
<td>32.0</td>
<td>3</td>
<td>FAT CLAY, gray, wet, soft to rather soft. (Glacial Lake Deposit) (continued)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(continued)</td>
</tr>
<tr>
<td>869.8</td>
<td>41.0</td>
<td>3</td>
<td>END OF BORING.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Water not observed with 39 1/2 feet of hollow stem auger in the ground.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Boring immediately backfilled with bentonite grout.</td>
</tr>
</tbody>
</table>
### Braun Project B1600465

**Geotechnical Evaluation**

64th Ave S Roadway and Bridge  
64th Avenue South and I-29  
Fargo, North Dakota

**BORING:** ST-08  
**LOCATION:** N430900.653, E2886356.449  
See sketch.

**DRILLER:** K. Miller  
**METHOD:** 3 1/4" HSA, Autohammer  
**DATE:** 9/1/16  
**SCALE:** 1" = 4'  

---

<table>
<thead>
<tr>
<th>Elev. feet</th>
<th>Depth feet</th>
<th>Symbol</th>
<th>Description of Materials</th>
<th>BPF</th>
<th>WL</th>
<th>MC%</th>
<th>qpsf</th>
<th>Tests or Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>910.8</td>
<td>0.0</td>
<td>FILL</td>
<td>FILL: Fat Clay, a little Sand and Gravel, brown, moist.</td>
<td>12</td>
<td>14</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>908.8</td>
<td>2.0</td>
<td>CH</td>
<td>FAT CLAY, with roots and organics, black, moist. (Buried Topsoil)</td>
<td>14</td>
<td>21</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>906.3</td>
<td>4.5</td>
<td>CH</td>
<td>FAT CLAY, with Silt lenses and laminations, brown, moist, rather soft to rather stiff. (Glacial Lake Deposit)</td>
<td>9</td>
<td>21</td>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-with iron-staining at 7 1/2 feet.</td>
<td>7</td>
<td>27</td>
<td>3 1/2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-with iron-staining at 10 feet.</td>
<td>7</td>
<td>36</td>
<td>2 1/4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-brown and gray below 12 1/2 feet.</td>
<td>5</td>
<td>41</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>892.8</td>
<td>18.0</td>
<td>CH</td>
<td>FAT CLAY, gray, wet, rather soft. (Glacial Lake Deposit)</td>
<td>4</td>
<td>53</td>
<td>1/2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| 884.8      | 26.0       |        | END OF BORING.  
Water not observed with 24 1/2 feet of hollow stem auger in the ground.  
Boring immediately backfilled with bentonite grout. | 4   | 47 | 1/2 |      |               |
### Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests

<table>
<thead>
<tr>
<th>Gravels (More than 50% of coarse fraction retained on No. 200 sieve)</th>
<th>Clean Gravels (Less than 5% fines)</th>
<th>Dew Gravels</th>
<th>Clean Sands (Less than 5% fines)</th>
<th>Dew Sands</th>
<th>Silts and Clays (Liquid limit 50% or more)</th>
<th>Inorganic</th>
<th>Organic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravels</td>
<td>Clean Gravels</td>
<td>Dew Gravels</td>
<td>Clean Sands</td>
<td>Dew Sands</td>
<td>Silts and Clays (Liquid limit 50% or more)</td>
<td>Inorganic</td>
<td>Organic</td>
</tr>
<tr>
<td>Gravels</td>
<td>Clean Gravels</td>
<td>Dew Gravels</td>
<td>Clean Sands</td>
<td>Dew Sands</td>
<td>Silts and Clays (Liquid limit 50% or more)</td>
<td>Inorganic</td>
<td>Organic</td>
</tr>
<tr>
<td>Gravels</td>
<td>Clean Gravels</td>
<td>Dew Gravels</td>
<td>Clean Sands</td>
<td>Dew Sands</td>
<td>Silts and Clays (Liquid limit 50% or more)</td>
<td>Inorganic</td>
<td>Organic</td>
</tr>
</tbody>
</table>

#### Laboratory Tests

<table>
<thead>
<tr>
<th>DD</th>
<th>WD</th>
<th>P200</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry Density, pcF</td>
<td>Wet Density, pcF</td>
<td>% Passing #200 sieve</td>
</tr>
</tbody>
</table>

### Soil Classification

- **Nutrient Content:**
  - Organic: Primarily organic matter, dark in color, and organic odor
  - Inorganic: Not organic

- **Moisture Content:**
  - Dry: Absence of moisture, dusty, dry to the touch
  - Moist: Damp but no visible water
  - Wet: Visible free water, usually soil is below water table

### Descriptive Terminology of Soil

- **Gravels:**
  - Clean Gravels:
  - Dew Gravels:
  - Silts and Clays:
  - Inorganic:
  - Organic:

- **Particle Size Identification:**
  - Boulders: 12" or more
  - Cobble: 3" to 12"
  - Gravel:
  - Sand:
  - Silt:
  - Clay:

- **Relative Proportions:**
  - little: 0 to 5%
  - with: 6 to 14%
  - 15%

- **Inclusion Thicknesses:**
  - lens: 0 to 1/8"
  - seam: 1/8" to 1"
  - layer: 1" or more

### Apparent Relative Density of Cohesionless Soils

- **Dry:** 0 to 4 BPF
- **Loose:** 5 to 10 BPF
- **Medium dense:** 11 to 30 BPF
- **Dense:** 31 to 50 BPF
- **Very dense:** 51 to 60 BPF

### Drilling Notes

- **BPF:** Numbers indicate blows per foot recorded in standard penetration test, also known as "N" value. The sampler was set 6 inches into undisturbed soil below the hollow-stem auger.

- **Driving Resistances:**
  - Driving resistances were then counted for second and third 6-inch increments, and added to get BPF.

### Partial Penetration

- ** WH:** WH indicates the sampler penetrated soil under weight of hammer and rods alone; driving not required.
- ** WR:** WR indicates the sampler penetrated soil under weight of rods alone; hammer weight and driving not required.
- **WL:** WL indicates the water level measured by the drillers either while driving or following drilling.
Appendix B

Grain Size Distribution Curves
Unconfined Compressive Strength Test Reports
Consolidated-U Undrained Triaxial Shear Strength Test Reports
Consolidation Test Reports
<table>
<thead>
<tr>
<th>GRAVEL</th>
<th>SAND</th>
<th>FINES</th>
</tr>
</thead>
<tbody>
<tr>
<td>COARSE</td>
<td>FINE</td>
<td>COARSE</td>
</tr>
<tr>
<td>MEDIUM</td>
<td>FINE</td>
<td>SILT</td>
</tr>
<tr>
<td>CLAY</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**GRAIN SIZE ACCUMULATION CURVE (ASTM)**

- **PERCENT PASSING**
- **PARTICLE DIAMETER, mm**
- **U.S. SIEVE SIZES**

**Braun Project B1600465**

**Geotechnical Evaluation**

64th Ave S Roadway and Bridge

64th Avenue South and I-29

Fargo, North Dakota

BORING: ST-03 DEPTH: 7.5'-8.5'

<table>
<thead>
<tr>
<th>GRAVEL</th>
<th>0.0%</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAND</td>
<td>27.8%</td>
</tr>
<tr>
<td>SILT</td>
<td>58.4%</td>
</tr>
<tr>
<td>CLAY</td>
<td>13.8%</td>
</tr>
<tr>
<td>D60=0.061</td>
<td>Cu=</td>
</tr>
<tr>
<td>D30=0.024</td>
<td>Cc=</td>
</tr>
</tbody>
</table>

**CLASSIFICATION:**

FAT CLAY with SAND (CH)
GOLD CLAY (CH)GRAVELSANDSILTCLAY 0.0% 1.2% 6.9% 91.9%

CLASSIFICATION: GRAVEL COARSE MEDIUM FINES

GRAIN SIZE ACCUMULATION CURVE (ASTM)

PERCENT PASSING

PARTICLE DIAMETER, mm

U.S. SIEVE SIZES

GRAVEL

SAND

FINES

COARSE

FINE

COARSE

MEDIUM

FINE

SILT

CLAY

U.S. SIEVE SIZES

Braun Project B1600465
Geotechnical Evaluation
64th Ave S Roadway and Bridge
64th Avenue South and I-29
Fargo, North Dakota
BORING: ST-03 DEPTH: 10.0'-11.0'

GRAVEL 0.0%
SAND 1.2%
SILT 6.9%
CLAY 91.9%
D60= Cu=
D30= Cc=

CLASSIFICATION: FAT CLAY (CH)
FAT CLAY (CH)

GRANULAR
GRAVEL
COARSE
MEDIUM
FINES
SAND
COARSE
MEDIUM
FINE
SILT
CLAY

0.0% 0.5% 6.6% 92.9%

D60 = Cw =
D30 = Ce =

GRAVEL
SAND
SILT
CLAY

Classification:

40 60 80 100

U.S. SIEVE SIZES

3" 1" 3/4" 1/2" 3/8"

90 80 70 60 50 40 30 20 10 0

PERCENT PASSING

PARTICLE DIAMETER, mm

Braun Project B1600465
Geotechnical Evaluation
64th Ave S Roadway and Bridge
64th Avenue South and I-29
Fargo, North Dakota
BORING: ST-03 DEPTH: 12.5' - 13.5'

Braun Intertec Corporation
GRAIN SIZE ACCUMULATION CURVE (ASTM)

<table>
<thead>
<tr>
<th>GRAVEL</th>
<th>SAND</th>
<th>FINES</th>
</tr>
</thead>
<tbody>
<tr>
<td>COARSE</td>
<td>FINE</td>
<td>COARSE</td>
</tr>
<tr>
<td>0.0%</td>
<td>0.3%</td>
<td>4.1%</td>
</tr>
</tbody>
</table>

CLASSIFICATION:
FAT CLAY (CH)

Braun Project B1600465
Geotechnical Evaluation
64th Ave S Roadway and Bridge
64th Avenue South and I-29
Fargo, North Dakota
BORING: ST-04 DEPTH: 29.5’-31.5’

Braun Intertec Corporation
### Grain Size Accumulation Curve (ASTM)

<table>
<thead>
<tr>
<th>GRAVEL</th>
<th>SAND</th>
<th>FINES</th>
</tr>
</thead>
<tbody>
<tr>
<td>COARSE</td>
<td>FINE</td>
<td>COARSE</td>
</tr>
<tr>
<td>0%</td>
<td>0%</td>
<td>1.6%</td>
</tr>
</tbody>
</table>

#### Particle Diameter, mm

<table>
<thead>
<tr>
<th>U.S. Sieve Sizes</th>
<th>3&quot;</th>
<th>1&quot;</th>
<th>3/4&quot;</th>
<th>1/2&quot;</th>
<th>3/8&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>PERCENT PASSING</td>
<td>100</td>
<td>90</td>
<td>80</td>
<td>70</td>
<td>60</td>
</tr>
</tbody>
</table>

#### Particle Diameter, mm

<table>
<thead>
<tr>
<th>PARTICLE DIAMETER, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.001</td>
</tr>
</tbody>
</table>

#### Classification: Fat Clay (CH)

<table>
<thead>
<tr>
<th>GRAVEL</th>
<th>SAND</th>
<th>SILT</th>
<th>CLAY</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0%</td>
<td>1.6%</td>
<td>7.2%</td>
<td>91.1%</td>
</tr>
</tbody>
</table>

**Braun Project B1600465**

Geotechnical Evaluation

64th Ave S Roadway and Bridge

64th Avenue South and I-29

Fargo, North Dakota

BORING: ST-04  DEPTH: 49.5'-51.5'

Braun Intertec Corporation
Braun Project B1600465
Geotechnical Evaluation
64th Ave S Roadway and Bridge
64th Avenue South and I-29
Fargo, North Dakota
BORING: ST-04  DEPTH: 55.0'-56.0'

GRAVITY SIEVE SIZES

<table>
<thead>
<tr>
<th>GRAVEL</th>
<th>SAND</th>
<th>FINES</th>
</tr>
</thead>
<tbody>
<tr>
<td>COARSE</td>
<td>FINE</td>
<td>COARSE</td>
</tr>
</tbody>
</table>

- GRAVEL: 0.0%
- SAND: 2.4%
- SILT: 12.1%
- CLAY: 85.4%
- D60= Cu=
- D30= Ce=

CLASSIFICATION: FAT CLAY (CH)
GRAIN SIZE ACCUMULATION CURVE (ASTM)

<table>
<thead>
<tr>
<th>GRAVEL</th>
<th>SAND</th>
<th>FINES</th>
</tr>
</thead>
<tbody>
<tr>
<td>COARSE</td>
<td>FINE</td>
<td>COARSE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MEDIUM</td>
</tr>
<tr>
<td></td>
<td></td>
<td>FINE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SILT</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CLAY</td>
</tr>
</tbody>
</table>

**Classification:**
- GRAVEL: 0.0%
- SAND: 4.2%
- SILT: 16.0%
- CLAY: 79.8%
- D60=0.002
- D30=
- D10=
- Cu=
- Cc=

**Classification:** FAT CLAY (CH)

**Boring:** ST-04  **Depth:** 60.0'-61.0'
# Grain Size Accumulation Curve (ASTM)

<table>
<thead>
<tr>
<th>GRAVEL</th>
<th>SAND</th>
<th>FINES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Classification:**

- **Gravel:** Coarse and Medium
- **Sand:** Coarse and Medium
- **Fines:** Silts and Clays

## Braun Project B1600465

**Geotechnical Evaluation**
- 64th Ave S Roadway and Bridge
- 64th Avenue South and I-29
- Fargo, North Dakota

**Boring:** ST-05  **Depth:** 59.5'-61.5'

- **GRAVEL:** 0.0%
- **SAND:** 2.8%
- **SILT:** 10.4%
- **CLAY:** 86.8%
- **D60=0.001:** Cw=
- **D30=:** Ce=

**Classification:**
- **FAT CLAY (CH)**
GRAIN SIZE ACCUMULATION CURVE (ASTM)

<table>
<thead>
<tr>
<th>GRAVEL</th>
<th>SAND</th>
<th>FINES</th>
</tr>
</thead>
<tbody>
<tr>
<td>COARSE</td>
<td>FINE</td>
<td>COARSE</td>
</tr>
</tbody>
</table>

U.S. SIEVE SIZES

PERCENT PASSING

PARTICLE DIAMETER, mm

Braun Project B1600465
Geotechnical Evaluation
64th Ave S Roadway and Bridge
64th Avenue South and I-29
Fargo, North Dakota
BORING: ST-06 DEPTH: 7.5'-8.5'

GRAVEL 0.0%
SAND 0.7%
SILT 28.6%
CLAY 70.7%
D60=0.003 Cu=
D30= Ce=
CLASSIFICATION: FAT CLAY (CH)
GRAIN SIZE ACCUMULATION CURVE (ASTM)

<table>
<thead>
<tr>
<th>GRAVEL</th>
<th>SAND</th>
<th>FINES</th>
</tr>
</thead>
<tbody>
<tr>
<td>COARSE</td>
<td>FINE</td>
<td>COARSE</td>
</tr>
</tbody>
</table>

PARTICLE DIAMETER, mm

PERCENT PASSING

GRAVITY: 34.8%
SAND: 31.2%
SILT: 34.0%
CLAY: 0.0%

Boring: ST-06  Depth: 10.0'-11.0'

CLASSIFICATION: SANDY SILT (ML)

Braun Project B1600465
Geotechnical Evaluation
64th Ave S Roadway and Bridge
64th Avenue South and I-29
Fargo, North Dakota

Braun Intertec Corporation
Braun Project B1600465
Geotechnical Evaluation
64th Ave S Roadway and Bridge
64th Avenue South and I-29
Fargo, North Dakota
BORING: ST-06 DEPTH: 12.5'-13.5'

GRAVEL
SAND
FINES

<table>
<thead>
<tr>
<th>PARTICLE DIAMETER, mm</th>
<th>PERCENT PASSING</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>100</td>
</tr>
<tr>
<td>1</td>
<td>90</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>80</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>70</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>60</td>
</tr>
<tr>
<td>4</td>
<td>50</td>
</tr>
<tr>
<td>10</td>
<td>40</td>
</tr>
<tr>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>40</td>
<td>20</td>
</tr>
<tr>
<td>60</td>
<td>10</td>
</tr>
<tr>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>200</td>
<td></td>
</tr>
</tbody>
</table>

U.S. SIEVE SIZES

Braun Intertec Corporation

CLASSIFICATION:
FAT CLAY (CH)

GRAVEL: 0.0%
SAND: 1.0%
SILT: 7.6%
CLAY: 91.4%
D60= Cw=
D30= Ce=
### UNCONFINED COMPRESSION TEST

![Graph of Compressive Stress vs. Axial Strain](image)

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined strength, tsf</td>
<td>0.871</td>
</tr>
<tr>
<td>Undrained shear strength, tsf</td>
<td>0.435</td>
</tr>
<tr>
<td>Failure strain, %</td>
<td>2.4</td>
</tr>
<tr>
<td>Strain rate, %/min.</td>
<td>1.00</td>
</tr>
<tr>
<td>Water content, %</td>
<td>44.9</td>
</tr>
<tr>
<td>Wet density, pcf</td>
<td>110.1</td>
</tr>
<tr>
<td>Dry density, pcf</td>
<td>76.0</td>
</tr>
<tr>
<td>Saturation, %</td>
<td>99.5</td>
</tr>
<tr>
<td>Void ratio</td>
<td>1.2187</td>
</tr>
<tr>
<td>Specimen diameter, in.</td>
<td>2.876</td>
</tr>
<tr>
<td>Specimen height, in.</td>
<td>5.559</td>
</tr>
<tr>
<td>Height/diameter ratio</td>
<td>1.93</td>
</tr>
</tbody>
</table>

**Description:** FAT CLAY, brown (CH)

**LL =**  | **PL =** | **PI =** | **Assumed GS =** 2.70 | **Type:** Thinwall

**Project No.:** B1600465  
**Date Sampled:** 9/2/2016  
**Remarks:**  
ASTM D 2166

**Client:** KLJ  
**Project:** 64th Ave S Roadway and Bridge  
64th Ave S and I-29, Fargo, ND  
**Sample Number:** ST-2  
**Depth:** 9.5-11.5’
UNCONFINED COMPRESSION TEST

Sample No. 1
Unconfined strength, tsf 1.294
Undrained shear strength, tsf 0.647
Failure strain, % 2.3
Strain rate, %/min. 1.00
Water content, % 55.9
Wet density, pcf 105.1
Dry density, pcf 67.4
Saturation, % 99.4
Void ratio 1.5464
Specimen diameter, in. 2.858
Specimen height, in. 5.565
Height/diameter ratio 1.95

Description: FAT CLAY, gray (CH)

LL = PL = PI = Assumed GS= 2.75 Type: Thinwall

Project No.: B1600465
Date Sampled: 9/8/2016
Remarks:
ASTM D 2166

Client: KLJ
Project: 64th Ave S Roadway and Bridge
64th Ave S and I-29, Fargo, ND
Sample Number: ST-3 Depth: 19.5-21.5'

Figure _____
**UNCONFINED COMPRESSION TEST**

![Graph showing unconfined compression test results](image)

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined strength, tsf</td>
<td>1.044</td>
</tr>
<tr>
<td>Undrained shear strength, tsf</td>
<td>0.522</td>
</tr>
<tr>
<td>Failure strain, %</td>
<td>1.9</td>
</tr>
<tr>
<td>Strain rate, %/min.</td>
<td>N/A</td>
</tr>
<tr>
<td>Water content, %</td>
<td>57.7</td>
</tr>
<tr>
<td>Wet density, pcf</td>
<td>104.1</td>
</tr>
<tr>
<td>Dry density, pcf</td>
<td>66.0</td>
</tr>
<tr>
<td>Saturation, %</td>
<td>99.1</td>
</tr>
<tr>
<td>Void ratio</td>
<td>1.6001</td>
</tr>
<tr>
<td>Specimen diameter, in.</td>
<td>2.858</td>
</tr>
<tr>
<td>Specimen height, in.</td>
<td>5.562</td>
</tr>
<tr>
<td>Height/diameter ratio</td>
<td>1.95</td>
</tr>
</tbody>
</table>

**Description:** FAT CLAY, gray (CH)

| LL | 87 |
| PL | 33 |
| PI | 54 |
| Assumed GS | 2.75 |
| Type | Thinwall |

**Project No.:** B1600465  
**Date Sampled:** 9/9/2016  
**Remarks:**  
ASTM D 2166  

**Client:** KLJ

**Project:** 64th Ave S Roadway and Bridge  
64th Ave S and I-29, Fargo, ND  
**Sample Number:** ST-4  
**Depth:** 29.5-31.5'
### UNCONFINED COMPRESSION TEST

**Sample No.** 1  
**Unconfined strength, tsf** 1.280  
**Undrained shear strength, tsf** 0.640  
**Failure strain, %** 2.5  
**Strain rate, %/min.** 1.00  
**Water content, %** 56.9  
**Wet density, pcf** 104.6  
**Dry density, pcf** 66.6  
**Saturation, %** 99.3  
**Void ratio** 1.5758  
**Specimen diameter, in.** 2.845  
**Specimen height, in.** 5.578  
**Height/diameter ratio** 1.96

**Description:** FAT CLAY, gray (CH)  
**LL =** 116  
**PL =** 25  
**PI =** 91  
**Assumed GS =** 2.75  
**Type:** Thinwall

**Project No.:** B1600465  
**Date Sampled:** 9/9/2016  
**Remarks:**  
ASTM D 2166

**Client:** KLJ  
**Project:** 64th Ave S Roadway and Bridge  
64th Ave S and I-29, Fargo, ND  
**Sample Number:** ST-4  
**Depth:** 49.5-51.5'

---

**Figure:**
UNCONFINED COMPRESSION TEST

Sample No. | 1
Unconfined strength, tsf | 0.700
Undrained shear strength, tsf | 0.350
Failure strain, % | 3.3
Strain rate, %/min. | N/A
Water content, % | 36.9
Wet density, pcf | 114.0
Dry density, pcf | 83.2
Saturation, % | 97.3
Void ratio | 1.0250
Specimen diameter, in. | 2.851
Specimen height, in. | 5.570
Height/diameter ratio | 1.95

Description: FAT CLAY, brown (CH)

LL =  PL =  PI =  Assumed GS= 2.70  Type: Thinwall

Project No.: B1600465
Date Sampled: 9/6/2016
Remarks:
ASTM D 2166

Client: KLJ
Project: 64th Ave S Roadway and Bridge
64th Ave S and I-29, Fargo, ND
Sample Number: ST-5  Depth: 9.5-11.5’

Figure ______
**UNCONFINED COMPRESSION TEST**

![Graph](image)

**Sample No.**  | 1  
---|---
**Unconfined strength, tsf**  | 1.386  
**Undrained shear strength, tsf**  | 0.693  
**Failure strain, %**  | 1.7  
**Strain rate, %/min.**  | 1.00  
**Water content, %**  | 53.1  
**Wet density, pcf**  | 106.0  
**Dry density, pcf**  | 69.2  
**Saturation, %**  | 99.9  
**Void ratio**  | 1.4360  
**Specimen diameter, in.**  | 2.862  
**Specimen height, in.**  | 5.591  
**Height/diameter ratio**  | 1.95  

**Description:** FAT CLAY, gray (CH)  
**LL = 85  **  
**PL = 25  **  
**PI = 60  **  
**Assumed GS= 2.70  **  
**Type: Thinwall**  

**Project No.:** B1600465  
**Date Sampled:** 9/6/2016  
**Remarks:**  
ASTM D 2166

---

**Client:** KLJ  
**Project:** 64th Ave S Roadway and Bridge  
64th Ave S and I-29, Fargo, ND  
**Sample Number:** ST-5  
**Depth:** 59.5-61.5'
### UNCONFINED COMPRESSION TEST

**Sample No.:** 1

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined strength, tsf</td>
<td>1.232</td>
</tr>
<tr>
<td>Undrained shear strength, tsf</td>
<td>0.616</td>
</tr>
<tr>
<td>Failure strain, %</td>
<td>1.4</td>
</tr>
<tr>
<td>Strain rate, %/min.</td>
<td>1.00</td>
</tr>
<tr>
<td>Water content, %</td>
<td>57.5</td>
</tr>
<tr>
<td>Wet density, pcf</td>
<td>105.2</td>
</tr>
<tr>
<td>Dry density, pcf</td>
<td>66.8</td>
</tr>
<tr>
<td>Saturation, %</td>
<td>100.0</td>
</tr>
<tr>
<td>Void ratio</td>
<td>1.5994</td>
</tr>
<tr>
<td>Specimen diameter, in.</td>
<td>2.854</td>
</tr>
<tr>
<td>Specimen height, in.</td>
<td>5.576</td>
</tr>
<tr>
<td>Height/diameter ratio</td>
<td>1.95</td>
</tr>
</tbody>
</table>

**Description:** FAT CLAY, brown (CH)

| LL = Assumed GS = 2.78 | PI = Type: Thinwall |

**Project No.:** B1600465
**Date Sampled:** 9/1/2016
**Remarks:**
ASTM D 2166

**Client:** KLJ

**Project:** 64th Ave S Roadway and Bridge
64th Ave S and I-29, Fargo, ND
**Sample Number:** ST-6 **Depth:** 19.5-21.5'

---

**Figure:** _______
**UNCONFINED COMPRESSION TEST**

**Sample No.** | 1
---|---
**Unconfined strength, tsf** | 1.379
**Undrained shear strength, tsf** | 0.689
**Failure strain, %** | 2.0
**Strain rate, %/min.** | 1.00
**Water content, %** | 56.9
**Wet density, pcf** | 105.1
**Dry density, pcf** | 67.0
**Saturation, %** | 99.9
**Void ratio** | 1.5721
**Specimen diameter, in.** | 2.856
**Specimen height, in.** | 5.583
**Height/diameter ratio** | 1.95

**Description:** FAT CLAY, gray (CH)

**LL =** | **PL =** | **PI =** | **Assumed GS=** 2.76 | **Type:** Thinwall

**Project No.:** B1600465
**Date Sampled:** 9/1/2016
**Remarks:**
ASTM D 2166

**Client:** KLJ

**Project:** 64th Ave S Roadway and Bridge
64th Ave S and I-29, Fargo, ND
**Sample Number:** ST-6 **Depth:** 34.5-36.5'

---

**BRAUN™ INTERTEC**
**UNCONFINED COMPRESSION TEST**

![Compressive Stress vs. Axial Strain Graph]

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Unconfined strength, tsf</th>
<th>Undrained shear strength, tsf</th>
<th>Failure strain, %</th>
<th>Strain rate, %/min.</th>
<th>Water content, %</th>
<th>Wet density, pcf</th>
<th>Dry density, pcf</th>
<th>Saturation, %</th>
<th>Void ratio</th>
<th>Specimen diameter, in.</th>
<th>Specimen height, in.</th>
<th>Height/diameter ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.709</td>
<td>0.354</td>
<td>2.5</td>
<td>1.00</td>
<td>46.1</td>
<td>110.0</td>
<td>75.3</td>
<td>99.1</td>
<td>1.2793</td>
<td>2.876</td>
<td>5.573</td>
<td>1.94</td>
</tr>
</tbody>
</table>

**Description:** FAT CLAY, brown (CH)

**LL =** | **PL =** | **PI =** | **Assumed GS** = 2.75 | **Type:** Thinwall

**Project No.:** B1600465  
**Date Sampled:** 9/1/2016  
**Remarks:**  
ASTM D 2166

**Client:** KLJ

**Project:** 64th Ave S Roadway and Bridge  
64th Ave S and I-29, Fargo, ND  
**Sample Number:** ST-7  
**Depth:** 12-14'

---

**Figure**
**Type of Test:**
CU with Pore Pressures

**Sample Type:** Thinwall

**Description:** FAT CLAY, gray (CH)

**Assumed Specific Gravity:** 2.78

**Remarks:** Rate of strain is 0.001 in/min. Failure criteria is based on the ultimate stress which occurs at 15% strain. Samples were saturated for 10 days and consolidated for 3 days.

**Figure** CU Triax ASTM D 4767

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Content, %</td>
<td>57.8</td>
<td>58.8</td>
<td>58.0</td>
</tr>
<tr>
<td>Dry Density, pcf</td>
<td>66.5</td>
<td>65.8</td>
<td>67.0</td>
</tr>
<tr>
<td>Saturation, %</td>
<td>99.8</td>
<td>99.9</td>
<td>99.8</td>
</tr>
<tr>
<td>Void Ratio</td>
<td>1.6094</td>
<td>1.6355</td>
<td>1.6547</td>
</tr>
<tr>
<td>Diameter, in.</td>
<td>1.431</td>
<td>1.431</td>
<td>1.429</td>
</tr>
<tr>
<td>Height, in.</td>
<td>2.791</td>
<td>2.792</td>
<td>2.786</td>
</tr>
</tbody>
</table>

**At Test**

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Content, %</td>
<td>57.4</td>
<td>57.8</td>
<td>56.6</td>
</tr>
<tr>
<td>Dry Density, pcf</td>
<td>66.9</td>
<td>66.6</td>
<td>68.1</td>
</tr>
<tr>
<td>Saturation, %</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>Void Ratio</td>
<td>1.5954</td>
<td>1.6073</td>
<td>1.6119</td>
</tr>
<tr>
<td>Diameter, in.</td>
<td>1.428</td>
<td>1.426</td>
<td>1.421</td>
</tr>
<tr>
<td>Height, in.</td>
<td>2.786</td>
<td>2.782</td>
<td>2.771</td>
</tr>
<tr>
<td>Pore Pressure Parameter B</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Consolidation Pressure, tsf</td>
<td>0.98</td>
<td>2.00</td>
<td>4.00</td>
</tr>
<tr>
<td>Back Pressure, tsf</td>
<td>6.15</td>
<td>5.13</td>
<td>3.13</td>
</tr>
<tr>
<td>Cell Pressure, tsf</td>
<td>7.13</td>
<td>7.13</td>
<td>7.13</td>
</tr>
<tr>
<td>Peak Deviator Stress, tsf</td>
<td>0.93</td>
<td>1.68</td>
<td>1.67</td>
</tr>
<tr>
<td>Total Pore Pr., tsf</td>
<td>6.50</td>
<td>6.11</td>
<td>5.07</td>
</tr>
<tr>
<td>Ultimate Deviator Stress, tsf</td>
<td>0.88</td>
<td>1.02</td>
<td>1.59</td>
</tr>
<tr>
<td>Total Pore Pr., tsf</td>
<td>6.48</td>
<td>6.14</td>
<td>5.54</td>
</tr>
<tr>
<td>Maj. Eff. Stress at Ultimate, tsf</td>
<td>1.53</td>
<td>2.01</td>
<td>3.19</td>
</tr>
<tr>
<td>Min. Eff. Stress at Ultimate, tsf</td>
<td>0.64</td>
<td>0.99</td>
<td>1.59</td>
</tr>
</tbody>
</table>

**Client:** KLJ

**Project:** 64th Ave S Roadway and Bridge
64th Ave S and I-29, Fargo, ND

**Sample Number:** ST-3 **Depth:** 39.5-41.5'

**Proj. No.:** B1600465 **Date Sampled:**
Type of Test: CU with Pore Pressures
Sample Type: Thinwall
Description: FAT CLAY, brown (CH)

Assumed Specific Gravity = 2.81
Remarks: Rate of strain is 0.001 in/min. Failure criteria is based on the ultimate stress which occurs at 15% strain. Samples were saturated for 10 days and consolidated for 3 days.

Figure CU Triax ASTM D 4767

Client: KLJ

Project: 64th Ave S Roadway and Bridge
64th Ave S and I-29, Fargo, ND

Sample Number: ST-4
Depth: 14.5-16.5'

Proj. No.: B1600465
Date Sampled:
Type of Test:
CU with Pore Pressures

Sample Type: Thinwall

Description: FAT CLAY, gray (CH)

Assumed Specific Gravity = 2.8

Remarks: Rate of strain is 0.001 in/min. Failure criteria is based on the ultimate stress which occurs at 15% strain. Samples were saturated for 10 days and consolidated for 3 days.

Figure CU Triax ASTM D 4767
CONSOLIDATION / SWELL TESTING

<table>
<thead>
<tr>
<th>Natural Sat.</th>
<th>Natural Moist.</th>
<th>Dry Dens. (pcf)</th>
<th>LL</th>
<th>PI</th>
<th>Sp. Gr.</th>
<th>Overburden (tsf)</th>
<th>P_c (tsf)</th>
<th>C_c</th>
<th>C_r</th>
<th>Swell Press. (tsf)</th>
<th>Swell %</th>
<th>e_o</th>
</tr>
</thead>
<tbody>
<tr>
<td>99.6 %</td>
<td>54.4 %</td>
<td>68.1</td>
<td>87</td>
<td>54</td>
<td>2.70</td>
<td>3.63</td>
<td>0.84</td>
<td>0.18</td>
<td></td>
<td></td>
<td></td>
<td>1.475</td>
</tr>
</tbody>
</table>

MATERIAL DESCRIPTION

FAT CLAY, gray (CH)

USCS | AASHTO
----|------
CH   |      

Project No.: B1600465  Client: KLJ
Project: 64th Ave S Roadway and Bridge
          64th Ave S and I-29, Fargo, ND
Remarks: ASTM D 2435

Source: Sample No.: ST-4  Elev./Depth: 29.5-31.5'

BRAUN™
INTERTEC
Dial Reading vs. Time

Project No.: B1600465
Project: 64th Ave S Roadway and Bridge
64th Ave S and I-29, Fargo, ND

Source: Sample No.: ST-4
Elev./Depth: 29.5-31.5'

**Load #3**
1.00 tsf
Cv @ 13.74 min. = 0.04 ft.²/day

**Load #4**
2.00 tsf
Cv @ 19.44 min. = 0.02 ft.²/day

**Load #5**
4.00 tsf
Cv @ 28.36 min. = 0.02 ft.²/day

**Load #6**
8.00 tsf
Cv @ 87.97 min. = 0.00 ft.²/day

Figure
Dial Reading vs. Time

Project No.: B1600465
Project: 64th Ave S Roadway and Bridge
64th Ave S and I-29, Fargo, ND
Source: Sample No.: ST-4

Elev./Depth: 29.5-31.5'

Elapsed Time (min.)

Dial Reading (in.)

Load #7
16.00 tsf

\( C_v \) @ 75.37 min. = 0.00 ft.²/day

Figure
**CONSOLIDATION / SWELL TESTING**

**MATERIAL DESCRIPTION**

<table>
<thead>
<tr>
<th>Natural</th>
<th>Dry Dens. (pcf)</th>
<th>LL</th>
<th>PI</th>
<th>Sp. Gr.</th>
<th>Overburden (tsf)</th>
<th>P_c (tsf)</th>
<th>C_c</th>
<th>C_r</th>
<th>Swell Press. (tsf)</th>
<th>Swell %</th>
<th>e_o</th>
</tr>
</thead>
<tbody>
<tr>
<td>98.9 %</td>
<td>64.3</td>
<td>116</td>
<td>91</td>
<td>2.70</td>
<td>3.64</td>
<td>1.19</td>
<td>0.18</td>
<td></td>
<td></td>
<td></td>
<td>1.623</td>
</tr>
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</table>

**Source:** Sample No.: ST-4  Elev./Depth: 49.5-51.5’

**Remarks:**

- **Project No.:** B1600465  **Client:** KLJ
- **Project:** 64th Ave S Roadway and Bridge
  64th Ave S and I-29, Fargo, ND
- **Remarks:** ASTM D 2435
Dial Reading vs. Time

Project No.: B1600465
Project: 64th Ave S Roadway and Bridge
64th Ave S and I-29, Fargo, ND

Source: Sample No.: ST-4
Elev./Depth: 49.5-51.5'

| Load #2 | 1.00 tsf | \( C_v @ 8.27 \text{ min.} = 0.06 \text{ ft.}^2/\text{day} \) |
| Load #3 | 2.00 tsf | \( C_v @ 13.35 \text{ min.} = 0.04 \text{ ft.}^2/\text{day} \) |
| Load #4 | 4.00 tsf | \( C_v @ 31.32 \text{ min.} = 0.01 \text{ ft.}^2/\text{day} \) |
| Load #5 | 8.00 tsf | \( C_v @ 64.52 \text{ min.} = 0.01 \text{ ft.}^2/\text{day} \) |
Dial Reading vs. Time

Project No.: B1600465
Project: 64th Ave S Roadway and Bridge
64th Ave S and I-29, Fargo, ND
Source: Sample No.: ST-4
Elev./Depth: 49.5-51.5'

Elapsed Time (min.)

Dial Reading (in.)

Load #6
16.00 tsf
Cv @ 55.42 min. = 0.01 ft²/day
# CONSOLIDATION / SWELL TESTING

![Graph showing consolidation/swell testing results]

<table>
<thead>
<tr>
<th>Natural Sat.</th>
<th>Moist. %</th>
<th>Dry Dens. (pcf)</th>
<th>LL</th>
<th>PI</th>
<th>Sp. Gr.</th>
<th>Overburden (tsf)</th>
<th>P_c (tsf)</th>
<th>C_c</th>
<th>C_r</th>
<th>Swell Press. (tsf)</th>
<th>Swell %</th>
<th>e_o</th>
</tr>
</thead>
<tbody>
<tr>
<td>100.0 %</td>
<td>51.1 %</td>
<td>70.8</td>
<td>85</td>
<td>60</td>
<td>2.70</td>
<td>3.09</td>
<td>0.73</td>
<td>0.15</td>
<td></td>
<td></td>
<td></td>
<td>1.379</td>
</tr>
</tbody>
</table>

**MATERIAL DESCRIPTION**

FAT CLAY, gray (CH)

### Project Details

**Project No.:** B1600465  
**Client:** KLJ  
**Project:** 64th Ave S Roadway and Bridge  
64th Ave S and I-29, Fargo, ND  
**Source:** Sample No.: ST-5  
**Elev./Depth:** 59.5-61.5'  

**Remarks:** ASTM D 2435
Project No.: B1600465
Project: 64th Ave S Roadway and Bridge
64th Ave S and I-29, Fargo, ND

Source: Sample No.: ST-5
Elev./Depth: 59.5-61.5'

Dial Reading vs. Time

Load #2
1.00 tsf
C_v @ 6.31 min. = 0.08 ft.²/day

Load #3
2.00 tsf
C_v @ 10.76 min. = 0.04 ft.²/day

Load #4
4.00 tsf
C_v @ 18.05 min. = 0.02 ft.²/day

Load #5
8.00 tsf
C_v @ 40.22 min. = 0.01 ft.²/day
Dial Reading vs. Time

Project No.: B1600465
Project: 64th Ave S Roadway and Bridge
64th Ave S and I-29, Fargo, ND
Source: ST-5
Sample No.: ST-5
Elev./Depth: 59.5-61.5'

Elapsed Time (min.)

Load #6
16.00 tsf

Cv @ 32.10 min. =
0.01 ft²/day
Appendix C

Analytical Summary
Shear Strength Evaluation
Settlement and Stability Analytical Graphics
### Shear Strength Parameters

<table>
<thead>
<tr>
<th>Formation</th>
<th>Unit Weight</th>
<th>$\theta$, Post-Peak</th>
<th>C</th>
<th>$\theta$</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surcharge</td>
<td>115</td>
<td>27.0 deg</td>
<td>0 psf</td>
<td>0 deg</td>
<td>1,000 psf</td>
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<tr>
<td>Embankment Fill</td>
<td>120</td>
<td>24.0 deg</td>
<td>100 psf</td>
<td>0 deg</td>
<td>1,500 psf</td>
</tr>
<tr>
<td>Sand</td>
<td>120</td>
<td>30.0 deg</td>
<td>0 psf</td>
<td>30.0 deg</td>
<td>0 psf</td>
</tr>
<tr>
<td>Sherack</td>
<td>111</td>
<td>17.0 deg</td>
<td>0 psf</td>
<td>0 deg</td>
<td>700 psf</td>
</tr>
<tr>
<td>Plastic Sherack</td>
<td>106</td>
<td>17.0 deg</td>
<td>0 psf</td>
<td>0 deg</td>
<td>1,200 psf</td>
</tr>
<tr>
<td>Brenna/Argusville</td>
<td>105</td>
<td>17.0 deg</td>
<td>0 psf</td>
<td>0 deg</td>
<td>1,300 psf</td>
</tr>
<tr>
<td>Glacial Till</td>
<td>130</td>
<td>34.0 deg</td>
<td>100 psf</td>
<td>0 deg</td>
<td>3,000 psf</td>
</tr>
</tbody>
</table>

### Hydraulic and Deformation Parameters

<table>
<thead>
<tr>
<th>Formation</th>
<th>$k_v$</th>
<th>$k_h$</th>
<th>$k_v/k_h$</th>
<th>$E$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surcharge</td>
<td>.01 ft/day</td>
<td>.01 ft/day</td>
<td>1.0</td>
<td>100,000 psf</td>
</tr>
<tr>
<td>Embankment Fill</td>
<td>.01 ft/day</td>
<td>.01 ft/day</td>
<td>1.0</td>
<td>100,000 psf</td>
</tr>
<tr>
<td>Sand</td>
<td>20 ft/day</td>
<td>20 ft/day</td>
<td>1.0</td>
<td>200,000 psf</td>
</tr>
<tr>
<td>Sherack</td>
<td>.0001 ft/day</td>
<td>.001 ft/day</td>
<td>0.1</td>
<td>200,000 psf</td>
</tr>
<tr>
<td>Plastic Sherack</td>
<td>.0001 ft/day</td>
<td>.001 ft/day</td>
<td>0.1</td>
<td>200,000 psf</td>
</tr>
<tr>
<td>Brenna/Argusville</td>
<td>.0001 ft/day</td>
<td>.001 ft/day</td>
<td>0.1</td>
<td>200,000 psf</td>
</tr>
<tr>
<td>Glacial Till</td>
<td>.001 ft/day</td>
<td>.001 ft/day</td>
<td>1.0</td>
<td>1,000,000 psf</td>
</tr>
</tbody>
</table>
B1600465: 64th Ave S Roadway and Bridge
Triaxial Shear Strength of Sherack and Brenna Formations

\[ \sigma = 100 \text{ psf} \]
\[ C = 105 \text{ psf} \]

\[ \phi = 16.5^\circ \]
\[ \phi = 17.2^\circ \]

- Sherack
- Brenna
West Profile Details, West End of Profile, Surcharge Stage

At completion of surcharge placement:

- Top/End of Stage 2 Embankment + Surcharge Fill
  El. 919, Approx. Sta. 118+20
- Top/End of Stage 3 Embankment + Surcharge Fill
  El. 923, Approx. Sta. 119+56
- Western Limits Drainage Sand and Wick Drains
- Western Limits of Geofoam
  El. 918, Approx. Sta. 120+80
- Design Grade

Drainage Sand
Wick Drains
Sherack
Brenna Argusville
Glacial Till

Distance: -40 0 40 80 120 160 200 240 280 320 360 400 440 480 520
B1600465: 64th Avenue South Reconstruction – Project Graphics

West Profile Details, West End of Profile, Post-Surcharge Placement of Geofoam

AT COMPLETION OF FINAL GRADING

Western Limits of Geofoam
El. 918, Approx. Sta. 120+80

Drainage Sand
Wick Drains

Sherack
Brienna/Argusville
Glacial Till
B1600465: 64th Avenue South Reconstruction – Project Graphics

West Profile Details, East End of Profile, Surcharge Stage

AT COMPLETION OF SURCHARGE PLACEMENT

Top of Stage 1 Embankment Fill
El. 915

Top of Stage 3 Embankment + Surcharge Fill
El. 923

Top/End of Stage 4 Surcharge Fill
El. 929, Approx. Sta. 126+48 to Sta. 126+96

Design Grade

Eastern Limits, Drainage Sand and Wick Drains
Approx. Sta. 127+40

520 560 600 640 680 720 760 800 840 880 920 960 1,000 1,040 1,080
B1600465: 64th Avenue South Reconstruction – Project Graphics

West Profile Details, East End of Profile, Post-Surcharge Placement of Geofoam

AT COMPLETION OF FINAL GRADING

Eastern Limits of Geofoam
El. 931, Approx. Sta. 126+72

Drainage Sand
Wick Drains
B1600465: 64th Avenue South Reconstruction – Project Graphics

Time-Rate of Settlement, West Profile Design Grade (relative to embankment performance).

Left-most highlighted node corresponds to bottom node (25678) in graph legend; right-most node corresponds to top node (28096) in legend.
B1600465: 64th Avenue South Reconstruction – Project Graphics

Time Rate of Settlement, West Profile Foundation Grade (relative to embankment overbuild).

Left-most highlighted node corresponds to top node (25105) in graph legend; right-most node corresponds to bottom node (29538) in legend.
B1600465: 64th Avenue South Reconstruction – Project Graphics

Effective Stress Factor of Safety versus Time, Surcharge Period, West Profile.
Effective Stress Factor of Safety versus Time, Post-Construction, West Profile.
Total Stress Factor of Safety, Surcharge Period, West Profile.
B1600465: 64<sup>th</sup> Avenue South Reconstruction – Project Graphics

West Section Details, East End of Profile, Surcharge Stage

AT COMPLETION OF SURCHARGE PLACEMENT

- Top of Stage 4 Embankment + Surcharge Fill El. 929
- Drainage Sand and Wick Drain Limits + 40 Feet Beyond 4:1 (h:v) Plane from Grading Pl of Road (Applies where Path is Present as Well)
- Design Grade
- Top of Stage 1 Embankment Fill El. 915

Layers:
- Sherack
- Breana/Argusville
- Glacial Till

Distance:
-280 -240 -200 -160 -120 -80 -40 0 40 80 120 160 200 240 280
West Section Details, East End of Profile, Post-Surcharge Placement of Geofoam

**AT COMPLETION OF FINAL GRADING**

3:1 (h:v) Effective Slope of Foam Prism Extending to Grading PIs while Maintaining 4 feet of Cover

- Sherack
- Brenna/Pepinville
- Glacial Till

Drainage Sand
Wick Drains
B1600465: 64th Avenue South Reconstruction – Project Graphics

Time-Rate of Settlement, West Cross Section Design Grade (relative to performance).

Left-most highlighted node corresponds to top node (25678) in graph legend; right-most node corresponds to bottom node (28096) in legend.
Time Rate of Settlement, West Cross Section Foundation Grade (relative to overbuild).

Left-most highlighted node corresponds to top node (25678) in graph legend; right-most node corresponds to bottom node (28096) in legend.
B1600465: 64th Avenue South Reconstruction – Project Graphics

Effective Stress Factor of Safety versus Time, Surcharge Period, West Cross Section.
B1600465: 64th Avenue South Reconstruction – Project Graphics

Effective Stress Factor of Safety versus Time, Post-Construction, West Cross Section.
B1600465: 64th Avenue South Reconstruction – Project Graphics

Total Stress Factor of Safety, Surcharge Period, West Cross Section.

<table>
<thead>
<tr>
<th>Color</th>
<th>Name</th>
<th>Unit Weight (pcf)</th>
<th>Cohesion' (psf)</th>
<th>Phi°</th>
<th>Cohesion (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light Green</td>
<td>Brønnøya/Arguzville (TS)</td>
<td>105</td>
<td>1,300</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yellow</td>
<td>Fill (TS)</td>
<td>120</td>
<td>1,500</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Red</td>
<td>Glacial Till (TS)</td>
<td>130</td>
<td>3,000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Light Blue</td>
<td>PL Sherack (TS)</td>
<td>105</td>
<td>1,200</td>
<td></td>
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<tr>
<td>Light Gray</td>
<td>Sand</td>
<td>120</td>
<td>0</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>Yellow</td>
<td>Sherack (TS)</td>
<td>111</td>
<td>700</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Purple</td>
<td>Surcharge (TS)</td>
<td>115</td>
<td>1,000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Distance

Elevation

2.20
Appendix D

Driven Pile Axial Resistance Graphs (3)
64th Ave S Arterial Roadway w/Bridge Construction-PN-19-A0
Embedment Analysis Results
East and West Abutment

Estimate Top of Pile Elevation ~922 feet MSL
Current Average Grade Elevation ~908 feet MSL
10x42 H-Pile
64th Ave S Arterial Roadway w/Bridge Construction-PN-19-A0
Embedment Analysis Results
West Abutment

Nominal Geotechnical Resistance (tons)

- Estimate Top of Pile Elevation ~922 feet MSL
- Current Average Grade Elevation ~908 feet MSL
- 14x73 H-Pile
Estimate Top of Pile Elevation ~906 feet MSL
Current Estimated Grade Elevation ~907 feet MSL
14x102 H-Pile
64th Ave S Arterial Roadway w/Bridge Construction-PN-19-A0
Embedment Analysis Results
East Abutment

Nominal Geotechnical Resistance (tons)

Elevation (ft)

- Estimate Top of Pile Elevation ~922 feet MSL
- Current Average Grade Elevation ~908 feet MSL
- 14x73 H-Pile