NOTE

Subject: EPA Comments on XCEL Energy – Bay Front Generating Station, Ashland, WI
Round 10 Draft Assessment Report

To: File

Date: May 15, 2012

1. Please make a global change from the term "inspection" to "assessment."

2. On second and third page of Table of Contents, the header reads “Polishing Pond.” Please verify that the name of the unit “Polishing Basin” is consistent throughout the report.

3. On p. 5, Section 1.2.8 “Hazard Potential Classification,” it may be advantageous, for the sake of full disclosure, to note the proximity of the CCW management units Surge Basin and Polishing Basin to Lake Superior, even if the units maintain the **Low** hazard rating in light of the closeness to the major water body.

4. In Appendix C, checklists, indicate both units are "less than low" for the hazard potential rating, where as in the text of the report, these units are both rated "low." Please correct, or add a statement in the text indicating that after further review of additional materials, GZA has revised the hazard potential rating from its initial view from the site visit.

5. On p. 4, Section 1.3.2 “Reservoir,” report refers to “Excel.” Is this meant to be Xcel, the utility? If so, maintain consistency of the utility’s name throughout the report.

6. On p. 9, section 2.6, please include a table that identifies each stability analysis performed, FOS achieved and a column identifying required minimum FOS sought.

7. On Appendix A, Limitations, please replace "Alliant" with "Xcel" in items 2 and 6.

8. In Appendix C, checklists, indicate that each unit is without a liner present. However, in the text (section 1.2) of the report, each unit is described as having: "Within the exterior embankments, a liner consisting of a 2 foot layer of compacted clay (“impervious blanket”) was placed over the base of the basin and extended along the upstream slopes to form an “impervious core” approximately 10 feet from upstream face." Please correct.

9. It is requested that either in Appendix C- the checklist, or in section 1.2 there be a specific statement made to address the following question: “Is any part of the impoundment built over wet ash, slag, or other unsuitable materials (like TVA)?” Please correct for the two impoundments.

Xcel Energy®
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414 Nicollet Mall
Minneapolis, Minnesota 55401-1993

July 26, 2012

Mr. Stephen Hoffman
US Environmental Protection Agency (5304P)
1200 Pennsylvania Avenue, NW
Washington, DC 20460

RE: Xcel Energy Response to Draft Report for the USEPA Assessment of Dam Safety at the Northern States Power Company/Bay Front Generating Station Surge Basin, Polishing Basin

Dear Mr. Hoffman:

Enclosed are comments from Northern States Power Company–Wisconsin, an Xcel Energy Company (NSP-W) on the draft report prepared by GZA, documenting the results of the June 14, 2011 dam safety inspection of the waste water treatment basins at NSP-W's Bay Front Generating Station in Ashland Wisconsin.

We appreciate the opportunity to review the content and technical conclusions of the draft report. Our comments are contained in Attachment 1 to this letter. In summary our comments indicate that the USEPA Coal Combustion Waste (CCW) impoundment criteria should not apply to the Bay Front waste water treatment basins, given that these structures were designed, permitted, constructed and continue to be operated as part of the facility's waste water treatment system. They were not designed nor do they serve as CCW storage or disposal impoundments, but instead only receive incidental amounts of coal combustion byproducts including slag fines. Consequently we respectfully suggest that the USEPA impoundment criteria should not apply here and these basins should be unrated. However, even if rated, the rating should be modified from a Poor rating to a Satisfactory rating, given that we have now had an opportunity to conduct further stability analysis and hydrologic/hydraulic analysis for these basins. Enclosed for your review in Attachment 2 are the results from those analyses. This report concludes that the dikes meet the required safety factors and are therefore acceptable, and that the basins are adequately sized to store a 100 year, 24 hour storm event.

As noted in GZA's inspection checklists found in Appendix C of this report, it should also be clarified that the basins have only a Less than Low hazard potential, not Low hazard potential as indicated.

Finally, we have evaluated the recommendations identified in the report for recurrent operation and maintenance. As discussed in Attachment 1, we have either completed or have scheduled to complete each of these recommendations. These recommendations are minor in nature and do not affect the performance or stability of these water treatment basins.

Therefore we respectfully submit that even if the USEPA impoundment criteria apply, the Poor rating is inappropriate and a Satisfactory rating should be provided.

If you have questions concerning our comments, please contact me by phone (612-330-5596), email (terry.e.coss@xcelenergy.com), or at the address below.

Sincerely,

Terry Coss, P.E.
Environmental Director
Xcel Energy
414 Nicollet Mall
Minneapolis, MN 55401

**Attachment 1: Comments on Draft Report Assessing Safety of Coal Combustion Surface Impoundments at the Bay Front Generating Station.**

**Attachment 2: Seismic Stability and Hydrology Analysis of Water Treatment Basins – Bay Front Generating Station.**

# Attachment 1
# Response to Draft Report for the USEPA Assessment of Dam Safety

## Northern States Power Company - Wisconsin
## Bay Front Generating Station
## Surge Basin and Polishing Basin

**Attachment 1**

# Response to Draft Report for the
# USEPA Assessment of Dam Safety of the
# Northern States Power Company
# Bay Front Generating Station
# Surge Basin and Polishing Basin

## Comments:

1. **The impoundments are not Coal Combustion Waste (CCW) storage or disposal impoundments, but instead are wastewater treatment basins which may contain incidental CCWs, and therefore, should not be rated**

   At multiple locations throughout this document the basins are characterized as "coal combustion waste (CCW)" impoundments. This is an incorrect classification; the Wisconsin Department of Natural Resources has permitted these basins as water treatment lagoons. These structures were not designed nor do they serve as CCW storage or disposal impoundments, but instead only receive incidental amounts of coal combustion byproducts including slag fines.

   The fly ash generated at this site is stored in silos and the bottom ash (slag) is collected and stored in a dewatering bin. Almost 100% of the CCW collected in these systems is beneficially re-used. Only incidental quantities CCW, along with other suspended solids from plant operations are collected in these basins.

   The basins were constructed in 1976 as an industrial waste water treatment facility. These basins are characterized by the Wisconsin Department of Natural Resources as an industrial waste water treatment facility as demonstrated in the description in the facility's Wisconsin Pollutant Discharge Elimination System (WPDES) Permit No. WI-0002887-06-0. Specifically, the basins receive process water generated from the slag dewatering bin, boiler water treatment, and various floor drains and sumps at the facility. The basins were constructed in a manner that allows settling of solids from wastewater in order to meet the conditions of the Federal Water Pollution Control Act of 1972 (FWPCA) as amended by the Clean Water Act of 1977 (CWA). Specifically, the basins are subject to total suspended solids (TSS) effluent limits of 100 mg/L daily maximum and 30 mg/L monthly average, as defined by the technology based guidelines of 40 Code of Federal Register (CFR) Part 423 for Steam Electric Power Generation point source discharges. In 1983 these limits were further reduced to 2.5 ppm. Accordingly, the basins are permitted and designed for wastewater treatment (i.e. settling of solids) prior to discharge to Lake Superior and not for the impounding of CCW.

In 1991 when the WDNR promulgated NR 213 - Lining of Industrial Lagoons and Design of Storage Structures, the basins were classified as "lagoons" which is defined as "natural or man made containment structure, constructed primarily of earthen materials and used for the treatment or storage of industrial, commercial or agricultural waste water, biological fermentation leachates or sludge." Again, the purpose of the basins is for treatment of industrial waste water and they are not coal combustion waste impoundments.

Given the above information, NSPW respectfully suggests that the following text should be revised:

- **Executive Summary** Page i, second paragraph, fifth line– starting with line 5. Delete and replace with:
  - "The basins evaluated in this assessment consist of a Surge Basin and Polishing Basin that were constructed in 1976. These basins were constructed as an industrial waste water treatment facility regulated by the Wisconsin Department of Natural Resources. These basins are not utilized for the storage of CCWs but were constructed to ensure that the facility's WPDES permitted discharges meet the applicable discharge limits. These basins only receive incidental amounts of solids from power plant operations including residual amounts of slag fines from the dewatering process. The Surge Basin was designed to allow for sufficient detention time to allow larger particles to settle prior to discharge."
- **Section 1.2.3 Purpose of the Impoundments** Page 2, first paragraph, fifth sentence- Delete and replace with:
  - "The Surge and Polishing basins at the site are embankment structures consisting of bottom ash fill that was placed and compacted with engineering oversight that were designed and constructed in 1976. The basins were built as an industrial waste water treatment facility to clarify water prior to discharge to Lake Superior. Fly ash and bottom ash (slag) produced at the BFGS are managed in silos and a dewatering bin respectfully, and trucked off-site for beneficial re-use."
- **Section 1.2.3 Purpose of the Impoundments** Page 2, second paragraph - Please replace entire paragraph with:
  - "The Surge Basin receives plant process water effluent which includes incidental quantities of solids from plant operations including residual amounts of slag fines from the dewatering bins. Solids are allowed to settle in the Surge Basin and decant water is discharged into the Polishing Basin. Discharges from the Polishing Basin are authorized by the State WPDES permit."
- **Section 1.2.4 Description of the Surge Basins and Appurtenances** Page 3, first complete paragraph-Incorrectly states that the surge basin is as a settling pond for

CCW generated by the BFGS that is not recycled for beneficial re-use. This is not correct; any residual slag fines that do settle out in the surge basin are beneficially used after they are removed from the pond during routine cleaning events. Beneficial re-use projects are authorized under NR 538. The use of the acronym CCW in this paragraph is also misleading Waste water effluent directed to the surge basin include primarily plant process water discharges and only residual (incidental) amounts of CCWs. It should also be noted that slag generated from the plant is first sent to the slag de-watering bin where the slag is separated out before the process waster is discharged into the surge basin.

- **Section 1.2.5 Description of Polishing Basin** Page 3, second paragraph, second line– Replace second and third line with the following
    - "This basin was commissioned in 1976, and receives process water, including only incidental quantities of slag fines from the Surge Basin outlet structure. Decant water and any potential unsettled solids enter the Polishing Basin from the Surge Basin flow control structure through three 12-in diameter steel discharge pipes …."

- **Section 1.3.3 Discharges at the Impoundment Sites** Page 5, only paragraph- Please amend first line to state:
    - "As discussed previously, water from the Surge Basin discharges into the Polishing Basin and then into Lake Superior as authorized in the facility's WPDES permit."

- **Section 2.1.1 Surge Basin General Findings** Since the Surge Basin was designed, permitted, constructed and operated as an industrial waste water treatment facility and not a CCW impoundment we respectfully suggest that the application of this rating to this basin is inappropriate.

- **Section 2.1.5 Surge Basin Discharge Pipes** Page 7, only paragraph- Please revise first line to:
    - "Process water and associated solids, including residual amounts of slag fines are discharged ….."

- **Section 2.1.6 Polishing Basin General Findings** Since the Polishing Basin was designed, permitted, constructed and operated as an industrial waste water treatment facility and not a CCW impoundment we respectfully suggest the application of this rating to this basin is inappropriate.

- **Section 2.3 Operation and Maintenance Procedures** Page 8, first paragraph- The document suggests that the facility has a National Pollutant Discharge Elimination System permit. This is not correct; the facility has a Wisconsin Pollutant Discharge Elimination System (WPDES) permit, which was approved by the Administrator of the US EPA. In this permit the basins are clearly identified as a wastewater treatment system/ facility. At no point have these basins ever been characterized as CCW impoundments.

2. **Hydrologic/hydraulic analysis and stability analysis for seismic loading are now available for these impoundments and demonstrate that these impoundments should not be given a POOR condition rating**

The USEPA draft dam assessment report includes recommendations to perform a stability analysis of the basins under seismic loading and update the hydrologic/hydraulic analysis for the basins to document the adequacy of the basins to accommodate the 100-year, 24-hour event. NSPW has completed these studies and have placed the results in Attachment 2 of this submittal.

The seismic analysis followed procedures developed by Olson and Stark (2003) and represent the present state-of-the-art in evaluating seismic stability of sloping ground. The seismic analysis has shown that the water treatment basins will be stable under a seismic event. The hydrology analysis was performed for a 100-year, 24-hr storm event and its impact on the water treatment basins were evaluated. The analysis showed there is adequate freeboard for the storm event. The results of these analyses meet the needs of the additional data request from the USEPA and provide the information required for a complete assessment of the water treatment basins.

Since these basins are not CCW impoundments we do not feel that it is appropriate that they are rated. However, in lieu of the results from the recent hydrological and seismic assessments if a rating is required for these basins, the rating should be revised to a rating of **SATISFACTORY**.

Given the above information, NSPW respectfully suggests that the following paragraphs should be revised:

- **Executive Summary,** Page iii, Remedial Measures- Delete section in its entirety since results of hydrologic and seismic studies indicate that no remedial measures are required.
- **Section 2.1.1 Surge Basin General Findings** Page 6, only paragraph- "The Surge Basin was found to be in **POOR** condition primarily due to inadequate information pertaining to the original 1976 hydrological/hydraulic analysis and lack of information on embankment stability under seismic loading conditions." This should be deleted and replaced with:
    - "Overall, the Surge Basin was found to be in **SATISFACTORY** condition. Recently submitted assessments demonstrate that the basins were constructed with adequate capacity for a 100 – year, 24 hour storm event. A recent seismic analysis indicates that the water treatment basins will be stable under a seismic event. An overall Site plan showing the impoundments is provided as **Figure 2**.

The location and orientation of the Surge Pond photographs provided in **Appendix F** are shown on **Figure 3**".

- **Section 2.1.6 Polishing Basin General Findings** Page 7, Only paragraph- "The Polishing Basin was found to be in **POOR** condition primarily due to inadequate information pertaining to the original 1976 hydrological/hydraulic analysis and lack of information on embankment stability under seismic loading conditions." This should be deleted and replaced with:
  - "Similarly to the Surge Basin, the overall condition of the Polishing Basin was found to be in **SATISFACTORY** condition. An overall Site plan showing the impoundments is provided as **Figure 2**. The location and orientation of photographs provided in **Appendix F** are shown on the Photo Plan in **Figure 3**".
- **Section 3.0 Assessments and Recommendations** Please re-evaluate this section in lieu of the information provided in this submittal.
- **Section 4.0 Engineer's Certification** Please re-evaluate this section in lieu of the information provided in this submittal.

### 3. NSPW response to recommendations for additional recurrent operation and maintenance (Executive Summary page ii and iii)

The draft report identified the following recommendations to address potential recurrent operation and maintenance activities:
1. Repair erosion on the downstream slope of the Surge Basin;
2. Fill currently observed animal burrows by injecting grout under low to moderate pressures to ensure the entire limits of the respective burrow is adequately filled;
3. Repair observed erosion on the upstream slopes of the Surge and Polishing Basins;
4. Monitor decant outflow structures and clear silt or debris which may block or impede outflow; and,
5. Take measures as necessary so as to maintain operability and function of the various impoundment water level control mechanisms.

NSPW has assessed these recommendations and Bay Front Station has taken actions to either complete or schedule the completion of these recommendations. Specifically we have taken the following actions:

Item 1 - the repair of the minor erosion on the down stream slope of the Surge Basin is scheduled for completion in August of 2012.

Item 2 - the filling of observed animal burrows by injecting grout under low to moderate pressures has been completed.

Item 3 - the repair of minor erosion on the upstream slope of the Surge and Polishing Basins has been scheduled for completion in August of 2012.

Item 4 - our routine pond inspection process has been modified to incorporate the monitoring of decant outflow structures and clearing silt or debris that may block or impede outflow.

Item 5 - our Preventive Maintenance Program includes a recurrent work order to take measures as necessary to maintain operability and function of the various impoundment water level control mechanisms.

4. **In addition, these impoundments have a "Less than Low" not "Low" hazard potential as defined in Appendix C**

   - **Executive Summary, Page i**, fourth paragraph, first line– "it is GZA's opinion that the Surge and Polishing Basins would be considered as having a **Low** hazard potential" In the inspection checklist completed by GZA and found in Appendix C the basins were identified as a "**Less than low hazard potential**," not a low hazard potential as indicated in the report.

   - **Section 1.2.8 Hazard Potential Classification** According to the inspection checklist completed by GZA and found in Appendix C the basins should have a "**Less than low hazard potential**" rating.

5. **NSPW also identified a few cleanup edits that are needed before a final draft is issued**

   - **Section 1.3.2 Reservoir.** Page 5, only paragraph- Note "Xcel" not "Excel".

# Attachment 2 of Seismic Stability and Hydrology Analysis of Water Treatment Basins

## Bay Front Generating Station

# Seismic Stability and Hydrology Analysis of Water Treatment Basins
## Bay Front Generating Station

*Prepared for*
*Xcel Energy*

*July 2012*

BARR

# Seismic Stability and Hydrology Analysis of Water Treatment Basins

## Bay Front Generating Station

*Prepared for*
*Xcel Energy*

*July 2012*

BARR

4700 West 77th Street
Minneapolis, MN 55435-4803
Phone: (952) 832-2600
Fax: (952) 832-2601

# Seismic Stability and Hydrology Analysis of Water Treatment Basins
# Bay Front Generating Station
## July 2012

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## Certification

I hereby certify that this report was prepared by me or under my direct supervision and that I am a duly licensed Professional Engineer under the laws of the State of Wisconsin.

Aaron T. Grosser, PE (Reg. #40823-6)

7/24/2012
Date

AARON T.
GROSSER
40823-6
EDINA,
MN

Reviewed by:

Ivan A. Contreras, PhD

7/24/2012
Date

# Executive Summary

On June 14, 2011 the United States Environmental Protection Agency's (USEPA) contractor GZA GeoEinvironmental Inc. conducted an assessment of dike safety for the water treatment basins located at Xcel Energy's Bay Front Generating Station. The USEPA issued a draft report for NSPW (d/b/a Xcel Energy) review and comment on May 29th, 2012. The draft report recommended that Xcel Energy perform a stability analysis of the basins under seismic loading and update the hydrologic/hydraulic analysis for the basins to document their adequacy to accommodate the 100-year, 24 – hour event.

Barr Engineering was hired by Xcel Energy to conduct these analyses. The results of these analyses confirm that the dikes and ponds associated with the Water Treatment Basins, although designed and constructed in 1976, meet current criteria for seismic stability and water storage. The specific conclusions are:

- A geotechnical investigation was performed to obtain up to date information on the dikes and foundations of the ponds. The investigation confirmed the stratigraphy that was assumed based on the previous design and the historical geotechnical information. The cone penetrometer testing also identified a layer of fill material below the water treatment basins that was identified as exhibiting potentially contractive behavior. The seismic stability analysis was performed based on procedures developed by Olson and Stark (2003). The analysis shows that the dikes meet the minimum required factor of safety of 1.2 and are therefore acceptable.
- A hydrology analysis was also performed on the water treatment basins. The ponds are large enough to store and discharge the 100–year, 24-hour storm event with 2.5 feet of freeboard in the surge basin and 2.8 Feet of freeboard in the polishing basin.

# 1.0 Introduction

Xcel Energy has requested Barr Engineering Company (Barr) perform a seismic slope stability analyses and hydrology analysis for the water treatment basins that exist on the Bay Front Generating Station site in Ashland, Wisconsin. These analyses were not available at the time of an inspection of the water treatment basins at the Bay Front Generating Station in June of 2011 by GZA GeoEnvironmental, Inc., a USEPA contractor. The inspection was part of the USEPA's program to conduct an assessment of dam safety on coal combustion surface impoundments. Due to recognized lack of seismic stability information and current hydrology analyses for the water treatment basins for review by the USEPA contractor, the facility was preliminarily graded as POOR condition. Therefore the analyses presented herein are for the use of updating the design, which was completed in 1976 by Barr Engineering Company, and to satisfy the data needs for the settling pond inspection. The results of this study demonstrate that the rating should be revised from POOR because the results show that the water treatment basins meet current design criteria.

As part of the seismic analysis, a geotechnical investigation was performed to collect current data regarding the construction materials, foundation conditions, and groundwater levels. A review of the previous design and construction documentation was also performed. Previously a geotechnical investigation had been performed on the site prior to construction in 1976 which included eight soil borings. That investigation characterized the ground conditions before any modification due to construction. Therefore, it was necessary to evaluate the changes caused by the construction and better define the subsurface conditions for this analysis. Furthermore the current investigation used updated techniques through the use of cone penetration testing (CPT) to collect soil behavior information and evaluate the strengths of the materials. This report will present the evaluation of the following considerations in regard to seismic analysis which must occur for instability to develop:

- shaking that is strong enough to trigger undrained strength loss,
- strength loss must be significant to result in post-liquefaction strengths less than the driving stresses; and
- there must be sufficient material that experiences loss in strength.

# 2.0 Geotechnical Investigation

## 2.1 Site Exploration

A total of seven CPT soundings were performed to facilitate this analysis and compliment the geotechnical information from the investigation completed in 1976 as part of the original design and before construction of the facility. The previous borings and new CPT locations are shown on Figure 1. The CPT data are presented in Appendix B. The CPT locations shown on the map were placed to approximately coincide with the previous boring locations. The adjacent CPT locations were an attempt to evaluate any changes in the stratigraphy shown on the boring logs that may have occurred due to site grading. Laboratory tests were not performed as part of this investigation because CPT was used however the data collected in 1976 was reviewed and used wherever possible. The historical data are presented in Appendix A.

Based upon a review of the historical data and the new CPT probes, the general site stratigraphy at the settling pond location consists of reworked fills soils compacted and used to construct the dikes. Descriptions of the fill can be seen on the boring logs in Appendix A which is generally characterized as ash with a soil classification of fine to medium sand, clayey silt, to silty fine sand based on the historical boring logs. The fill ranges in thickness, depending on the boring or CPT probe location, from about 5 to 15 feet thick. At the base of the fill there is a layer of the fill classified as silty sand to silt about 1 to 4 feet thick with low CPT tip resistances (<20 tons per square foot) which is indicative of materials that could be considered contractive or susceptible to liquefaction during a seismic event. It is known that throughout the fill deposit buried logs exist from old logging operations. The settling pond design shows a clay blanket exists within the ponds and acts as a relatively impermeable barrier along the pond sides and bottom. This clay blanket was encountered at CPT-2. Below the fill soils and clay blanket is a thin natural silty sand layer about 0.5 feet thick which overlies the silty clay to clay natural foundation lacustrine soils common in the Lake Superior basin.

## 2.2 Shear Strength

Along with using the CPT to evaluate the current stratigraphy near the settling basins, the CPT data were used to evaluate the behavior of the materials under undrained shear conditions. All CPT soundings were conducted by Minnesota Geoservices (MNGEO) of St. Paul, Minnesota. The CPT testing was performed with a 20-ton track-mounted rig with an enclosed work space. Testing was performed in general accordance with ASTM D5778.

The cones used in the investigation have a 15-centimeters-squared (2.3-inches-squared) base area and a 60-degree apex angle. The sleeve area of the cones is 225 square centimeters (34.9 square inches). The fluid used to saturate the filter was glycerin. MNGEO provided Barr with complete records of tip resistance, sleeve friction, pore water pressure, and friction ratio for each CPT sounding, along with results of any dissipation tests (Appendix B).

The CPT data interpretation was performed using an in-house program designed by Barr. The in-house program has been cross-checked with CPTINT version 5.2, commercially available software, for quality assurance and has been deemed comparable. The program uses the soil behavior type classification system from CPT data. The classification system is based on the corrected tip resistance ($q_t$), the friction ratio ($R_f$), and pore-water pressure parameter ($B_q$), and includes a total of 12 soil behavior types. The relevant cone parameters are defined as follows:

$$q_t = q_c + (1-a)u_2$$

$$R_f = \left( {}^{f_s}\!/_{q_t} \right) 100$$

$$B_q = \frac{u_2 - u_o}{q_t - \sigma_{vo}}$$

Where:

$q_c$ = tip resistance measured by the cone, load per area

$a$ = the area ratio of the cone (0.75)

$u_2$ = measured pore water pressure during cone penetration, load per area at the shoulder location

$f_s$ = unit sleeve friction resistance, load per area

$\sigma_{vo}$ = total overburden stress, load per area

$u_o$ = in-situ pore water pressure, load per area

Published relationships exist relating these cone parameters to soil behavior type, unit weight, undrained shear strength (for fine-grained soils) or relative density (for coarse-grained soils), overconsolidation ratio, strength, deformation moduli, friction angle and contractive/dilative behavior. The CPT data were used in this evaluation for determining stratigraphy, strength parameters, and behavior of the soils. The raw CPT logs are in Appendix B. The data were divided by stratigraphy, as determined through CPT soil behavior relationships and SPT boring logs, and used to determine soil shear strength.

Figure 2 presents the friction angle of the materials encountered on the site. The data are plotted by elevation and show the higher friction angle soils (approximately 37 to 45 degree friction angle) are located within the upper 10 to 15 feet or elevation range 613 to 600 feet. The remainder of the soils that behave in a drained manner have reported friction angle of 32 to about 40 degrees below about elevation 600 feet. Figure 3 presents the undrained shear strength of the materials that appear to behave in an undrained manner.

Figure 3 shows that the clay liner was encountered around elevation 607 feet and the strength of the clay is about 500 pounds per square foot (psf). Below about elevation 600 feet, the natural clay foundation is encountered with shear strengths ranging from about 500 to 5,000 psf.

Figure 3 through Figure 6 presents some data points approximately between 580 and 595 feet in elevation which have uncharacteristically little scatter. This is due to poor readings from the CPT pore water pressure sensor. Close inspection of the pore water pressure sensor filter ring showed that the fly ash fill had blocked the pores of the filter ring whereby not allowing pore water to pass freely.

## 2.3 Susceptibility to Liquefaction

Liquefaction refers to post-yield undrained behavior of saturated contractive silts and sands. The potential for the soils exhibiting low CPT tip stresses that exist above the beach sand layer to liquefy was evaluated using the CPT data. Fear and Robertson (1995) presented a relationship to assess the tendency for relatively clean sands to contract or dilate, based on corrected SPT blow counts and effective vertical stress, which was later amended to relate to corrected tip resistance from CPT. CPT analysis was performed to evaluate the contractive/dilative behavior. With CPT data, the corrected tip resistance ($q_{c1}$) is plotted against overburden pressure with a line initially proposed by Fear and Robertson (1995) dividing contractive and dilative behavior. Values plotting to the left or below the line are contractive and those values plotted to the right or above the line are dilative. Olson and Stark (2003) further filters out test points for soil that should not be characterized with a $USSR_{liq}$, beyond the Fear and Robertson contractive/dilative analysis by limiting definition to tests with a tip resistance less than about 67 tons per square foot or 6.5 megapascals.

Figure 4 presents the results of the analysis and shows that a portion of the silty sand to silt fill layer identified above the natural silty sand is contractive and therefore susceptible to liquefaction. Because this layer is susceptible to liquefaction, the CPT data were used to determine the yield

($USSR_{yield}$) and liquefied ($USSR_{liq}$) undrained shear strength ratios for the contractive data points shown on Figure 4. The CPT data were analyzed to estimate a yield undrained shear strength ratio based on corrected cone tip resistance ($q_{t1}$ in megapascals), are shown on Figure 5, based on Olson and Stark (2003):

$$\mathrm{USSR}_{\mathrm{yield}} = \frac{s_{u(yield)}}{\sigma'_{vo}} = 0.205 + 0.0143(q_{t1})$$

Material characterization was also performed using methods presented in Olson and Stark (2003) to evaluate the post-liquefaction or liquefied strength ratio as shown on Figure 6 as:

$$\mathrm{USSR}_{\mathrm{liq}} = \frac{s_{u(liq)}}{\sigma'_{vo}} = 0.03 + 0.0143(q_{t1})$$

These relationships presented were developed based on back analysis of data from case histories of failed slopes comprised of sands, silty sands, and tailings and are also suitable for use in the materials found on this site.

# 3.0 Stability Analysis

A seismic stability analysis was performed for through a section of the water treatment basins. The analysis performed is described in detail below.

## 3.1 Engineering Analysis Methodology

### 3.1.1 General

The dike analyzed for both seepage and seismic slope stability under the current configuration using the traditional limit-equilibrium approach. In this approach, the soil is assumed to be at the state of limiting equilibrium and a factor of safety is computed. Soil seepage and strength parameters were determined from geotechnical investigations at the site and laboratory tests performed on soil from the site. Geometry of the cross section was based on estimated crest and pond elevations and adjusted, where necessary to develop a stable model cross section. The end results of the analyses are presented in this section of the report.

For the analysis discussed here, we used available information and made performance predictions for reasonable conditions due to the variability of foundation conditions at the site. The analysis presented conforms to prudent engineering practices.

This evaluation integrates seepage and slope-stability modeling software. This incorporates the permeability of the individual layers within the cross section to calculate seepage and then incorporates the seepage forces into the stability analysis. The modeling techniques, assumptions, and limitations of these approaches are described in the following sections.

### 3.1.2 Seepage

The seepage analysis provides a good understanding of groundwater flow and how it is related to dike and pond stability. Seepage parameters were based on previous assessment performed in 1976 for the hydrogeologic materials. The seepage simulation for the cross section presented in this report model groundwater flow for steady-state conditions. It does not consider the impact from transient conditions such as fill placement, pore-pressure increases or decreases, or other conditions.

The seepage analysis is an important aspect of the modeling process. For complicated cross sections, the use of estimated phreatic surfaces may lead to models that are not conservative. Therefore, the computer model used to create a flow net is also used to evaluate seepage flow through dams. The model uses the flow net to calculate the cross-section seepage forces, which are then incorporated into the slope stability model. This method is used in lieu of relying on an estimated phreatic surface developed from piezometer readings (which are not available at this time) or visual observations, ignoring seepage forces within the model. The seepage forces should be representative of those in the dike cross section provided the model is calibrated using the range of permeability recommended based on geotechnical test results.

#### 3.1.2.1 SEEP/W 2007 Software

The seepage was modeled using SEEP/W, a computer modeling program developed by Geo-Slope International. SEEP/W uses finite-element analysis to model the movement of water and pore-pressure distribution within porous materials, such as soils. It was chosen because comprehensive formulation makes it possible to analyze both simple and highly complex seepage problems. SEEP/W can formulate saturated and unsaturated flow, steady-state and transient conditions, and a variety of boundary conditions. Model integration (SEEP/W and SLOPE/W) allows the use of seepage files in limit-equilibrium slope-stability analysis. SEEP/W generates an output file containing the heads at the nodes of the finite-element mesh. The integration of Geo-Slope products allows the use of the SEEP/W head file in the slope stability program to compute the effective stress, allowing evaluation of the seepage impact on stability. This information was used to evaluate dike stability under steady-state conditions.

#### 3.1.2.2 Seepage Mesh and Boundary Conditions for Proposed Conditions

The finite-element mesh was created to conform as closely as possible to the existing conditions for the cross section. Quadrilateral and triangular iso-parametric elements were used to build the mesh in accordance with the geometry lines. The boundary conditions for the model were defined by setting a constant total head at the nodes representing maximum pond level and Lake Superior water elevation. Potential seepage-face review nodes were placed on the downstream face of the dike. These nodes allow the model to check for possible boundary seepage.

### 3.1.3 Slope Stability

These analyses assess dike stability in terms of factor of safety. The limit-equilibrium methodology incorporated the seepage and slope stability analyses for evaluating stability. This two-phased approach first determines the steady-state flow conditions and seepage pressures and then calculates the factor of safety of the slope using the seepage pressures.

#### 3.1.3.1 SLOPE/W 2007 Software

The slope stability analyses were conducted using SLOPE/W, a computer-modeling program developed by GEO-Slope International that uses the limit-equilibrium theory to compute the factor of safety of earth and rock slopes. It is capable of modeling using a variety of methods to compute the factor of safety of a slope while analyzing complex geometry, stratigraphy, and loading conditions. As previously discussed, to compute effective stress, SLOPE/W allows importation of the head file from the seepage analysis. As a result, this approach incorporates the calculation of seepage forces when computing the factor of safety.

##### *3.1.3.1.1 Factor of Safety Calculation*

Spencer's method was used to calculate the factor of safety of the dike cross section. It is considered adequate because it satisfies all conditions of static equilibrium and provides a factor of safety based on both force and moment equilibrium.

##### *3.1.3.1.2 Searching Technique for Critical Failure Surface*

In SLOPE/W the critical failure surface can be circular, block, or user-specified. In the circular and block searching technique, the grid of circle-centers (or center of block) and radius (or ends blocks) is established by the user and then the program searches for the circle or block yielding the minimum factor of safety. With the user-specified technique, the user completely defines the shape of the failure surface and the factor of safety is computed for that surface. In the limit-equilibrium approach, the shape of the critical failure surface (circular, block, log spiral, piecewise linear, etc.) must be specified in advance.

#### 3.1.3.2 Drained and Undrained Analyses

Drained and undrained stress conditions will both occur during the life span of the proposed dikes. The modeling procedure included evaluating the dike section for undrained loading in the undrained shear strength analysis (USSA) and the effective stress or drained loading in the effective shear strength analysis (ESSA).

The yield shear strength ($Su_{(yield)}$) of a saturated, contractive, and sandy soil is defined as the peak shear strength available during undrained loading. The shear strength mobilized at large deformation is the liquefied shear strength ($Su_{liq}$), sometimes also called the post-liquefaction shear strength. The yield and liquefied shear-strength ratios are, respectively, the yield and liquefied shear strengths normalized with respect to the vertical effective stress within the zone of liquefaction prior to failure. As discussed in Section 2.0, they are identified as $USSR_{yield}$ and $USSR_{liq}$, respectively.

Liquefaction can occur if a rapid change in stress is applied to the dike in the form of an earthquake. Initially, the change from normally drained to undrained shearing may be localized, but the decrease in resistance may lead to a rapid transfer of shear stresses to adjacent soil zones. These adjacent zones then behave as if under undrained conditions, eventually leading to overall undrained behavior of the fine tailings/slimes. It is important to evaluate this rapid change in stress developed through seismic activity. The analysis methodology is described in subsequent sections.

## 3.2 Geometry

The typical dike cross section chosen for analysis is located on the central portion of the water treatment basins and includes the ponded water behind the dikes. The crest elevation is about 613 feet and is about 13.5 feet wide. The downstream slope of the dike is about 3:1 (H:V) and extends down to natural grade at about elevation 604 feet. Beyond the toe of the slope about 40 feet laterally is Lake Superior at about elevation 601 feet. The slope below the water level was estimated by reviewing local fishing maps and discussions with plant staff. The upstream slope is 3:1 (H:V). The water level in the basin was assumed at 2.5 feet below the crest of the dike. The configuration of the cross section can be seen on the modeling outputs presented in Appendix C.

## 3.3 Modeling Parameters

In-situ CPT testing data along with engineering judgment were used to select representative parameters for the analysis of the slope. Shear strength and permeability parameters were derived for the various material types as described in the following sections.

### 3.3.1 Permeability

The main parameter related to seepage analysis is hydraulic conductivity, otherwise known as permeability. The proposed parameters are presented in Table 1. These values are similar to those used in 1976 analyses.

**Table 1 Permeability Parameters**

| Material | Permeability (cm/sec) |
|---|---|
| Dike Fill | $1.2 \times 10^{-4}$ |
| Upper Fill | $2.5 \times 10^{-2}$ |
| Lower Fill | $2.5 \times 10^{-2}$ |
| Silty Sand | $1.0 \times 10^{-4}$ |
| Clay Blanket | $1.0 \times 10^{-7}$ |
| Clayey Silt | $1.0 \times 10^{-7}$ |
| Deep Clay | $1.0 \times 10^{-8}$ |

### 3.3.2 Shear Strength

Table 2 provides a summary of the proposed shear-strength model parameters. The material strengths are presented as a friction angle and cohesion value (Mohr-Coulomb material) where appropriate. For other materials, strength is represented as an undrained shear-strength ratio.

**Table 2 Estimated Strength Parameters**

| Material | Saturated Unit Weight (pcf) | ESSA | | USSA | |
|---|---|---|---|---|---|
| | | Phi' (degrees) | C' (psf) | $Phi_{cu}$ (degrees) | $C_u$ (psf) |
| Dike Fill | 110 | 42 | - | 42 | - |
| Upper Fill - Dilative | 107 | 38 | - | 38 | - |
| Lower Fill - Contractive | 107 | 32 | - | $S_{u(yield)} = 0.22$<br>$S_{u(liq)} = 0.05 - 0.12$ | |
| Silty Sand | 105 | 40 | - | 40 | - |
| Clay Blanket | 133 | 26 | - | - | 500 |
| Clayey Silt | 125 | 26 | - | - | 800 |
| Deep Clay | 125 | 24 | - | - | 800 |

### 3.3.3 Required Minimum Factors of Safety

Typical acceptable factors of safety for dike stability are 1.3 for the USSA analysis considering the peak or yield strength of the soil and 1.5 for the ESSA analysis—were used for this study. For the seismic stability analysis, a minimum factor of safety of 1.2 was targeted.

## 3.4 Seismic Slope Stability Analysis

The seismic slope stability analysis was based on procedures developed and presented in Olson and Stark (2003) and was used extensively by Olson to evaluate case histories of dam failures. The analysis assumed right to left failure where an exterior failure would occur through the dike initiating inside the pond and extending past the toe of the dike. The procedure is discussed in subsequent sections of the report and is used throughout industry in evaluating the stability of sloping ground.

### 3.4.1 Evaluating Liquefaction Triggering

Liquefaction can be triggered through seismic events or statically. A seismic triggering event (earthquake) occurs globally and instantly impacts all soils. The potential for the fine-grained materials encountered to liquefy in response to seismic triggering events is due to the fact that these materials may have been placed in a loose condition. This loose condition generally results in contractive behavior of these materials during undrained shearing such as a seismic event. Therefore undrained shear strength analyses (USSA) are performed to evaluate the seismic stability of dikes and embankments.

This study evaluated seismic liquefaction triggering. The basic steps of the liquefaction triggering analyses for seismic liquefaction, consistent with Olson and Stark's methodology, are described below:

1. Back-analyze the critical failure surface using limit equilibrium theory by incrementally reducing yield undrained shear strength values for the contractive, undrained materials until the factor of safety equals 1.0.
2. Analyze a model with the identified critical failure surface input as a fully- specified failure surface. This model specifically uses undrained shear strengths for soils that behave in an undrained manner and undrained shear strength ratios for those materials that are susceptible to liquefaction.
3. Utilize resulting stresses from the USSA model with the fully-specified failure surface to assess liquefaction triggering in each slice of the failure surface. These stresses are evaluated against an increase of driving forces due to seismic triggering.

Methods used to determine whether and where liquefaction would be triggered along the critical failure surface are discussed in the following sections.

### 3.4.2 Method of Analysis

The triggering of liquefaction was assessed for seismic conditions. All liquefaction triggering analyses used the results of the SEEP/W models. The seismic stresses were estimated using published relationships and added to the static stresses from the static SLOPE/W model.

The method used was based on procedures outlined by Olson and Stark (2003) and all references can be found in this document. With this procedures, the steady-state or liquefied, strength may be presented as a ratio by normalizing the strength to the effective overburden pressure ($USSR_{liq} = S_{u(liq)} / \sigma'_{vo}$) as discussed previously.

### 3.4.2.1 General Procedure

The Olson and Stark (2003) procedures can generally be summarized in the following steps:

Step 1 – Perform a limit equilibrium analysis (SLOPE/W) to determine the driving shear stress ($\tau_{driving}$) and effective overburden stress ($\sigma'_{vo}$) for each slice along the critical failure surface.

Step 2 – Calculate the average static shear stress ratio ($\tau_{driving} / \sigma'_{vo,ave}$) for each slice using the limit equilibrium results.

Step 3 – Estimate the average seismic shear stress ($\tau_{seismic,ave}$) using the published relationships in Olson and Stark (2003).

Step 4 – Compute $USSR_{yield}$ and $USSR_{liq}$ and using corrected mean CPT penetration resistance.

Step 5 – Determine the values of $S_{u(yield)}$, $S_{u(liq)}$ and $\tau_{driving}$ along the base of each slice.

Step 6 – Calculate the factor of safety against liquefaction triggering for each slice as:

$$FS_{triggering} = \frac{Su_{peak}}{\tau_{driving} + \tau_{seismic,ave} + \tau_{other}}$$

*Note: $\tau_{other}$ relates to external driving stresses, such as surcharges, that would not be included within the static driving shear stress. Values for this parameter were not used in this analysis.*

Step 7 – Revise the slope stability model based on the results of the triggering analysis. .

### 3.4.2.2 Seismic Shear Stress Estimation

As noted in Step 1, a yield USSA model is first run to determine where the most critical slip surfaces exist. The base stresses along each slice, including $\tau_{driving}$ and $\sigma'_{vo}$, as well as slice base width and average slice height, are exported from the model and put into a spreadsheet. The shear resistance (or shear strength, $S_u$) along the base of each slice can also be exported and used to assess what material type exists at the base of the slices.

With the data, the weighted average overburden stress value ($\sigma'_{vo,\ ave}$) along the failure surface for the potentially liquefiable soils and the average static shear stress ratio ($\tau_{driving} / \sigma'_{vo,ave}$) are computed.

In 1982, Seed and Idriss analyzed multiple sites that experienced an earthquake with a magnitude around 7.5, evaluating when and where liquefaction did or did not occur. From these analyses, relationships were proposed to identify when materials would or would not liquefy (similar to the contractive-dilative behavior relationships). To adjust this liquefaction potential curve for sites with magnitudes higher or lower than 7.5, correction factors called Magnitude Scaling Factors were introduced. Since then, multiple scaling factors have been proposed. Based on the results of the NCEER/NSF workshops, the following MSF relationship was recommended and presented in Olson and Stark (2003):

$$MSF = \frac{10^{2.24}}{M^{2.56}} = 3.5$$

The average seismic shear stress ($\tau_{seismic,ave}$) can then be estimated using published relationships. This is the maximum sustained seismic shear stress averaged normalizing to 15 cycles of uniform shaking with a Magnitude Scaling Factor. The seismic shear stress computed over 15 cycles is sustained for a sufficient number of cycles to generate substantial excess pore water. Olson and Stark (2003) proposed that the average seismic shear stress can be calculated by:

$$\tau_{seismic,ave} = \frac{0.65 * \frac{a_{max}}{g} * \sigma_{vo} * r_d}{C_M}$$

Where: $a_{max}$ = peak free-field surface acceleration, ft/s$^2$ (0.02, conservative assessment for the region)

$g$ = acceleration of gravity, 32.2 ft/s$^2$

$\sigma_{vo}$ = total overburden stress, psf

$r_d$ = depth reduction factor

$C_M$ = lower bound of the range of Magnitude Scaling Factors

The depth reduction factor ($r_d$) is a stress reduction coefficient, as deeper soils are less likely to liquefy due to confining pressures, computed as a function of depth ($z$) in meters by:

$$r_d = \frac{1.000 - 0.4113z^{0.5} + 0.04052\,z + 0.001753\,z^{1.5}}{1.000 - 0.4177z^{0.5} + 0.05792z - 0.006205z^{1.5} + 0.001210z^{2}}$$

The average seismic shear stress ($\tau_{seismic,ave}$) that is computed using the equation above is then added into the denominator to calculate the factor of safety against triggering ($FS_{triggering}$) for each slice. For any slices where the $FS_{triggering}$ is below 1.1, the material strength at the base of the slice is changed to the post-liquefaction strength for calculation of the post-liquefied slope stability factor of safety against flow ($FS_{flow}$).

#### 3.4.2.3 Determining Factor of Safety

As described in the previous section if any of the factors of safety for individual slices in the liquefaction triggering analysis were inadequate, post-liquefaction strengths would need to be applied to those areas in the model. The $FS_{triggering}$ is again computed as the yield shear strength divided by the driving shear forces (static and, when appropriate, seismic).

According to Olson and Stark (2003), any segments where the computed $FS_{triggering} > 1$ are unlikely to liquefy, and if all segments have a $FS_{triggering} > 1$, a post-liquefaction stability analysis is not necessary. Segments with a $FS_{triggering} < 1$ should have their strength values reduced to the liquefied shear strength ratio ($USSR_{liq}$) during a post-liquefaction analysis for the same failure surfaces. It is also prudent to model segments with marginal stability against triggering ($FS_{triggering} < 1.1$) with post-liquefaction strengths in a post-liquefaction analysis as some deformation can be expected to occur when the safety factor against flow is marginal ($FS_{flow} < 1.1$). This helps protect against the potential for deformation-induced liquefaction and progressive failure in marginally non-liquefiable zones.

## 3.5 Results of Slope Stability Modeling

The results of the seismic slope stability analysis are provided in Appendix C. The triggering analysis results show that although there are silty sand to silt contractive soils found at the site the cross section analyzed is stable. In fact none of the stability model segments in the triggering analysis resulted in a factor of safety less than 1.4 when the seismic shear stress was applied. Therefore none of the sections were assumed to liquefy. The resulting stability analysis where $USSR_{yield}$ strength values were used results in a factor of safety of about 1.66 which exceeds the minimum required factor of safety of 1.2. This resulting factor of safety is significantly high and therefore deformation is not expected. The factors of safety for each of the USSA and ESSA models is reported in Table 3 with the corresponding required minimum factors of safety.

**Table 3 Summary of Stability Analyses**

| Material | Factor of Safety | Minimum Required |
|---|---|---|
| USSA | 2.20 | 1.3 |
| USSA Seismic | 1.66 | 1.2 |
| ESSA | 2.68 | 1.5 |

The seismic factor of safety is a result of the low seismicity of the region which limits the seismic driving stresses in the settling pond, at the locations evaluated, to generally less than five pounds per square foot. These computed results indicate that seismic events are unlikely to occur that could impact the stability of the water treatment basins.

# 4.0 Hydrology Analysis

## 4.1 Pond Description

The two ponds modeled in this analysis consist of the Surge Basin and the Polishing Basin. Process water enters the surge basin from the plant. The process water has three peak flows of about 1,382 gallons per minute (gpm) at 8 hour intervals in addition to a base flow of about 120 gpm. The Surge basin settles the coarse particles and attenuates the peak flows. The water flows through 5.5 inch square orifice to the Polishing basin. The polishing basin is designed with a long flow path and minimal velocity to settle fine particles before discharge to Lake Superior. Flow from the polishing basin is controlled by a weir that is 15.7 feet wide that minimizes the bounce in the basin.

## 4.2 Analysis

The Surge Basin and Polishing Basin were modeled using HydroCAD to verify that the basins have adequate capacity to handle the 24-hour, 100-year storm event in addition to the normal plant discharges. The 24-hour, 100-year storm event is a rainfall of 6.01 inches as per the Rainfall Frequency Atlas of the Midwest for Zone 2 in Wisconsin.

The model input flows were timed to model the critical condition. The design rainstorm was lagged such that the peak of the storm coincided with the 3rd discharge from the plant. This allowed the assessment of the worst case scenario of a heavy rainfall combined with normal discharge from the plant.

## 4.3 Results

Results of the modeling are summarized in Table 4. The freeboard during the peak flow event is presented and within design standards.

**Table 4 Results of Hydrology Analysis**

| Pond Location | Normal operations Maximum flow CFS | Normal operations Maximum elevation MSL | 24–Hour, 100-year Flood peak Maximum Flow CFS | 24–Hour, 100-year Flood peak Maximum elevation MSL | 24- hour, 100-year flood peak Freeboard Feet |
|---|---|---|---|---|---|
| Surge Basin | 0.95 | 610.6 | 1.14 | 611.0 | 2.5 |
| Polishing Basin | 1.0 | 608.6 | 4.6 | 608.7 | 2.8 |

# 5.0 Summary

Analyses have been performed to satisfy the recommendations outlined in the dike assessment report for the USEPA. The specific analyses recommended were the seismic and hydrology analysis of the water treatment basins. The seismic analysis followed procedures developed by Olson and Stark (2003) and represent the present state-of-the-art in evaluating seismic stability of sloping ground. The seismic analysis has shown that the water treatment basins will be stable under a seismic event with an acceleration similar to those shown on USGS maps. The hydrology analysis was performed for a 100-year, 24-hr storm event and its impact on the water treatment basins was evaluated. The analysis showed there is adequate freeboard for the storm event. The results of these analyses meet the needs of the additional data request from the USEPA and will allow for a complete assessment of the water treatment basins.

# 6.0 Report Qualifications

## 6.1 Variations in Subsurface Conditions

### 6.1.1 Material Variability

The evaluation, analyses, and recommendations were developed from a limited amount of site and subsurface information. Strata boundaries and thicknesses are, inferred to some extent based on behavior reported in the CPT test and the previous soil boring logs. Strata boundaries may also be gradual transitions, and they can be expected to vary in depth, elevation, and thickness away from the boring locations. Although strata boundaries can be determined with continuous sampling, the boundaries apparent at some locations likely vary away from each investigation location. Variations in subsurface conditions present between borings/CPT may not be revealed and could be present.

### 6.1.2 Groundwater Variability

Groundwater measurements were made under the conditions reported within the report, shown on the CPT logs, and interpreted in the text of this report. It should be noted that the observation periods were generally relatively short, and groundwater can be expected to fluctuate in response to rainfall, snowmelt, flooding, irrigation, seasonal freezing and thawing, surface drainage modifications, and other seasonal and annual factors.

### 6.1.3 Precautions Regarding Changed Information

Barr's understanding of the site conditions has been presented to the extent it was reported to Barr by others through conversations or data review. If Barr has not correctly presented or interpreted the project details, Barr should be notified. New or changed information could render the evaluation, analysis, and recommendations invalid.

## 6.2 Limitations of Analysis

This report is for the exclusive use of Xcel Energy without written approval by Barr, no responsibility to other parties regarding this report is assumed. Barr's evaluation, analysis and recommendations may not be appropriate for other parties or projects.

No established national standards exist for data retrieval and geotechnical evaluations. Barr has used the methods and procedures described in this report. In performing its services, Barr used the degree of care, skill, and generally accepted engineering methods and practices ordinarily exercised under similar circumstances and under similar budget and time restraints by reputable members of its profession currently practicing in the same locality. Reasonable effort was made to characterize the project site based on the site-specific field work, however, the analyses represent a large area, and variations in stratigraphy, strength, and groundwater conditions from any of the locations at which testing was performed may occur. No warranty of the investigation, analysis, or design presented herein, expressed or implied, is made.

# 7.0 References

Fear, C.E. and P.K. Robertson, 1995. Estimating the Undrained Strength of Sand: A Theoretical Framework. Canadian Geotechnical Journal, Vol. 32, p859-870.

Olson, S.M. and T.D. Stark, 2003b. Yield Strength Ratio and Liquefaction Analysis of Slopes and Embankments. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, 727-737.

# Figures

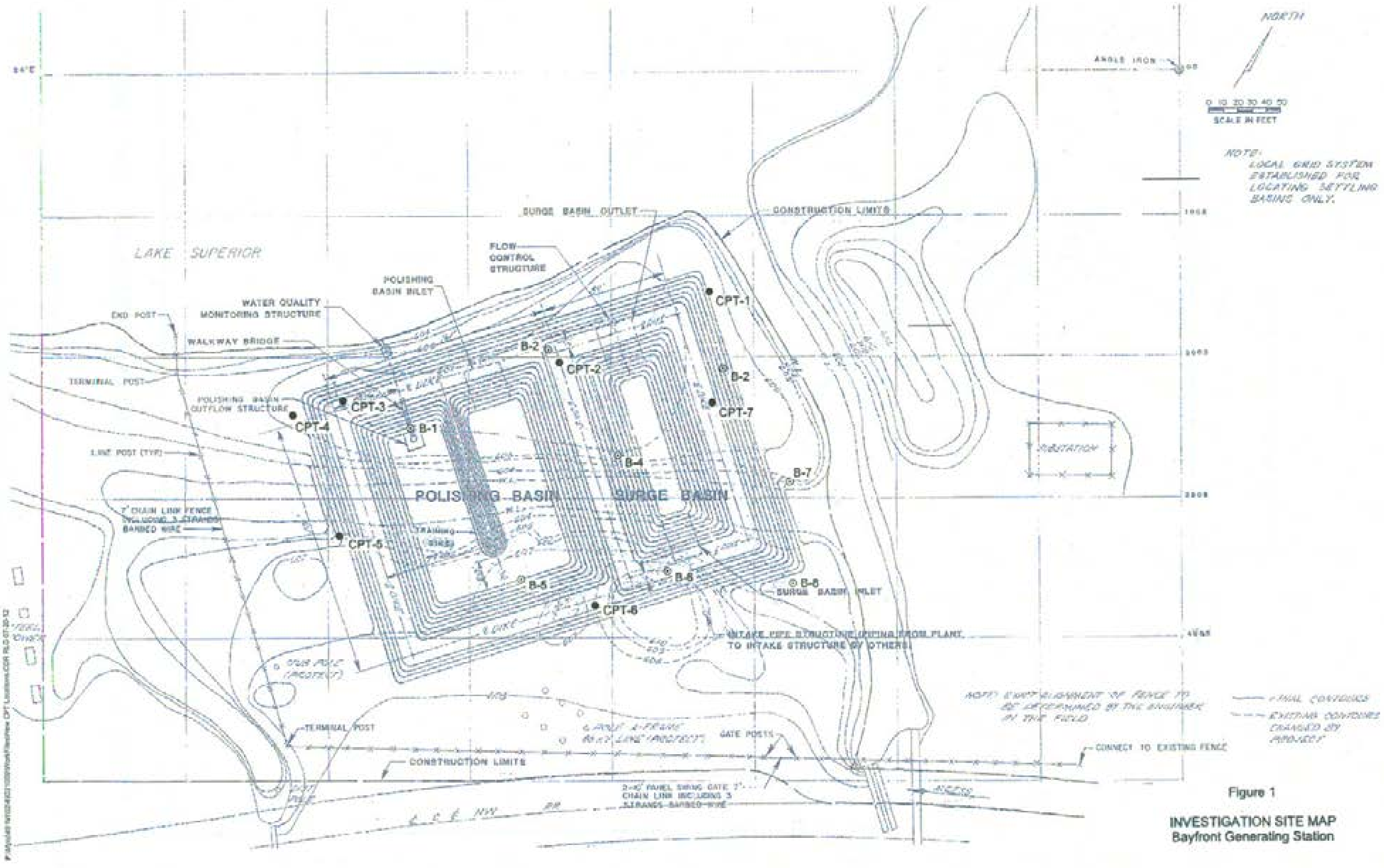

Figure 1

INVESTIGATION SITE MAP
Bayfront Generating Station

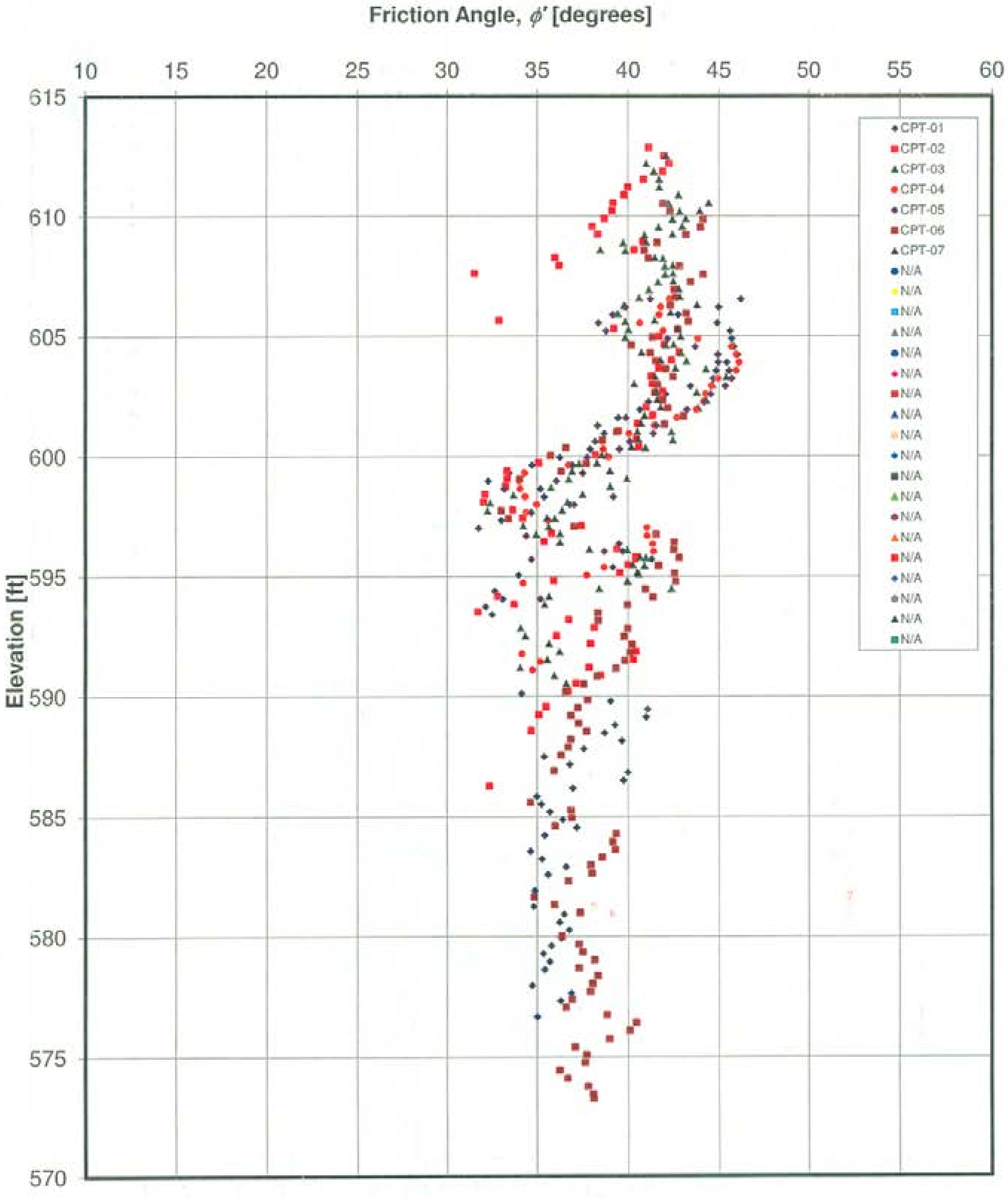

Figure 2. Friction Angle vs. Elevation (Kulhawy & Mayne, 1990)

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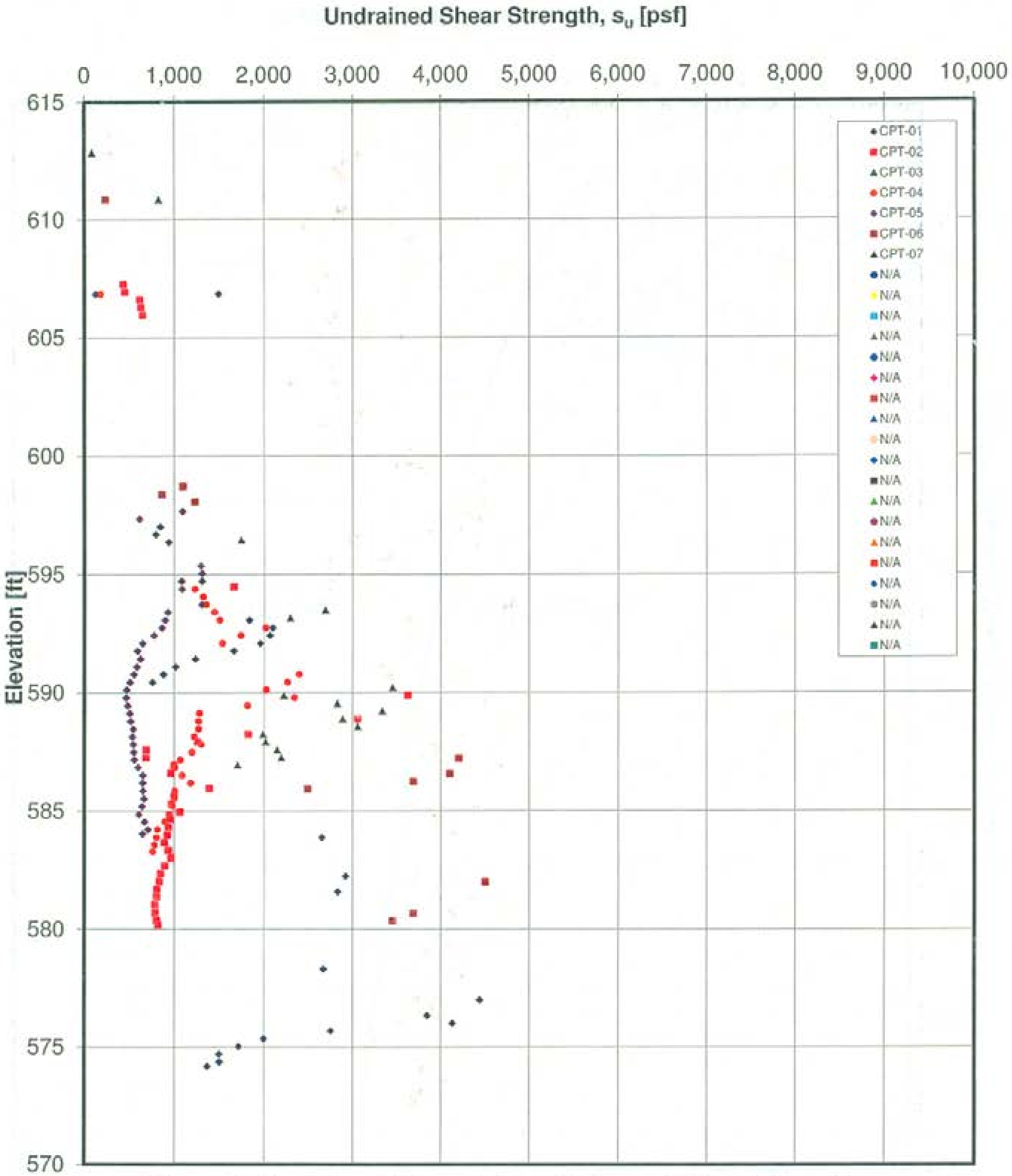

Figure 3. Undrained Shear Strength from CPT vs. Elevation ($N_{kt}$=16)

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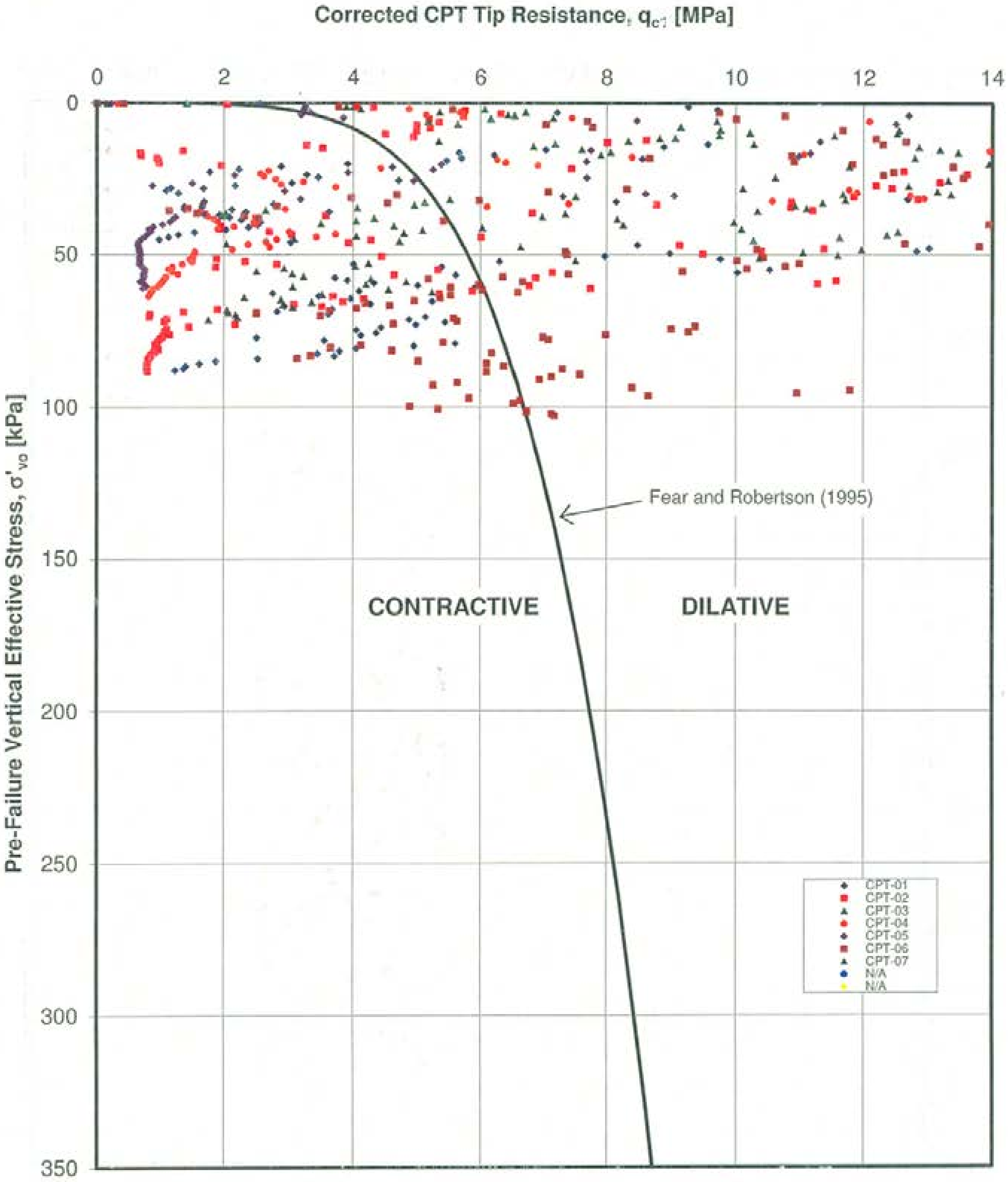

Figure 4. Contractive/Dilative Behavior (after Fear and Robertson, 1995)

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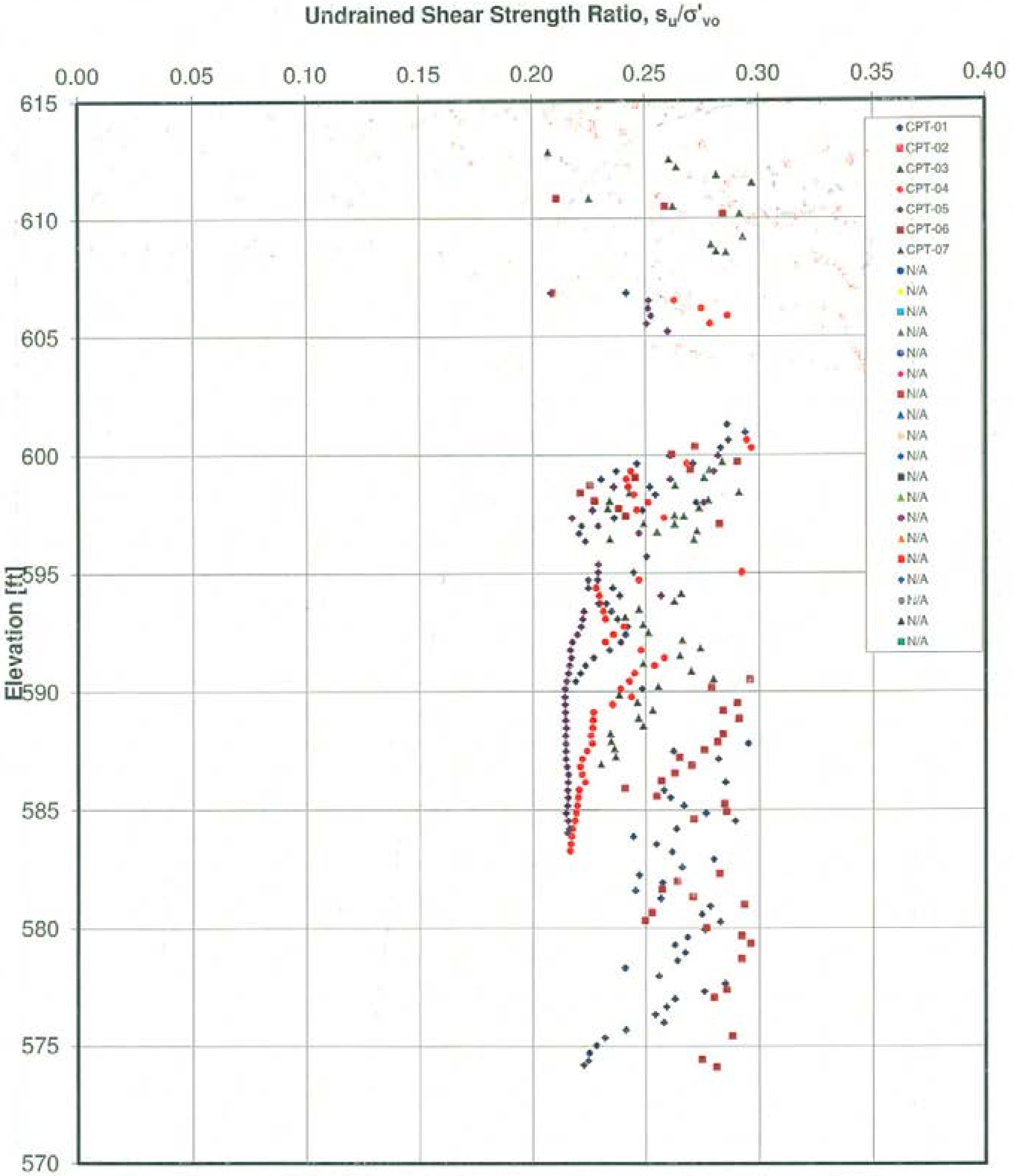

Figure 5. Yield Undrained Shear Strength Ratio from CPT vs. Elevation (Olson and Stark, 2003)

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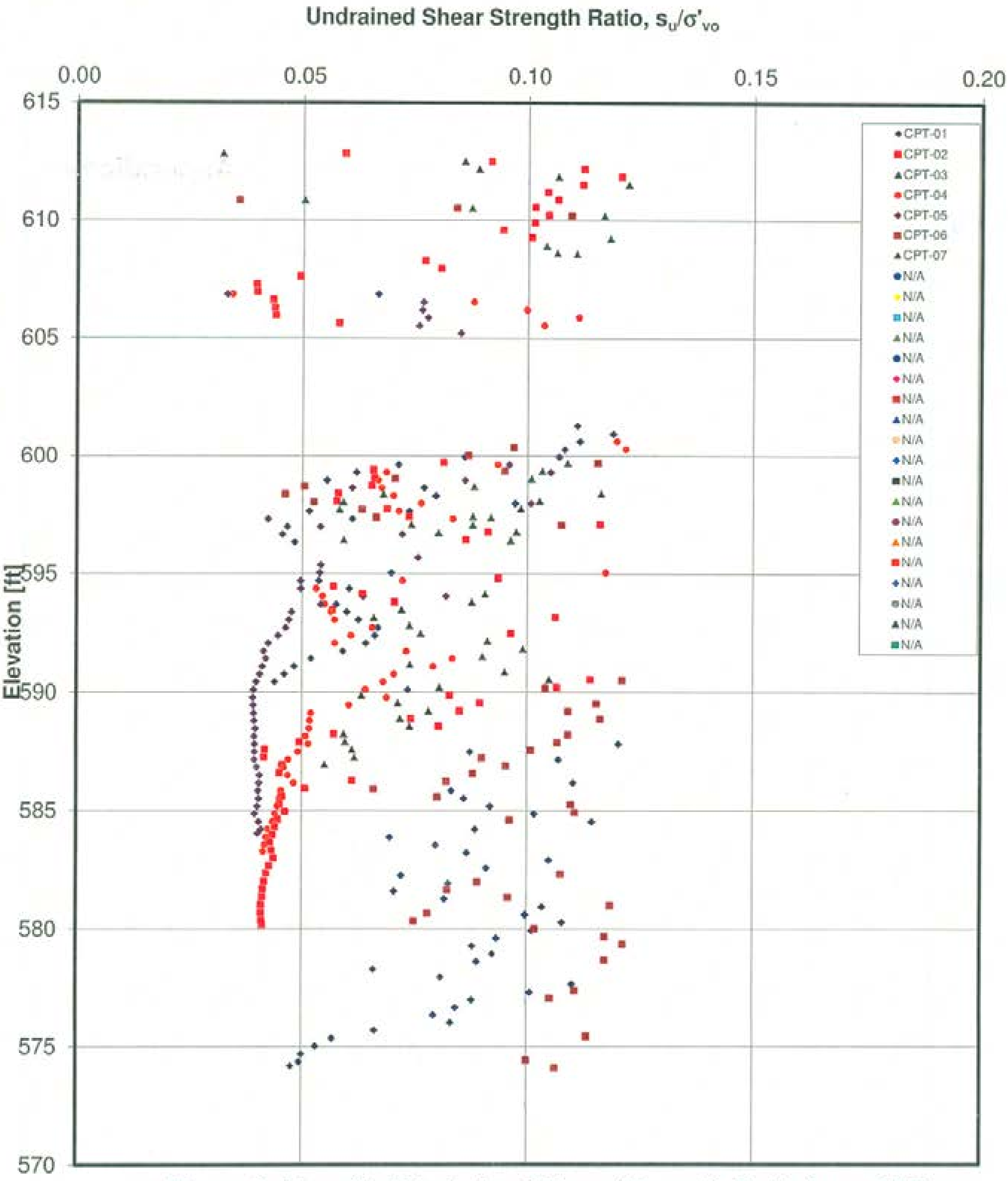


Figure 6. Liquefied Undrained Shear Strength Ratio from CPT vs. Elevation (Olson and Stark, 2003)

# Appendices

# Appendix A

## Historical Geotechnical Data

# LOG OF BORING NO. 1

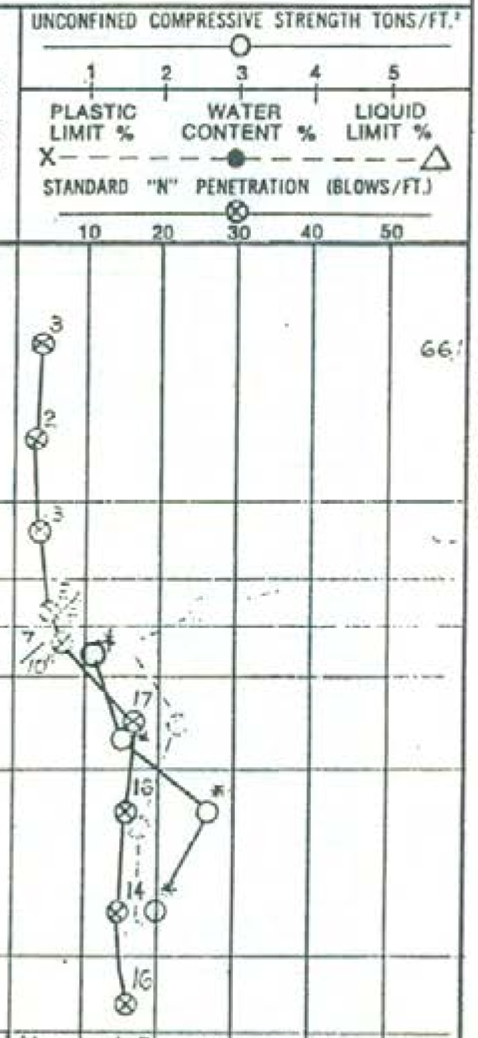
OWNER
Lake Superior Power District

ARCHITECT-ENGINEER
Barr Engineering Company

SITE
Ashland,Wisconsin

PROJECT NAME
Bayfront Settling Basins

| DEPTH ELEVATION | SAMPLE NO. | TYPE SAMPLE | SAMPLE DIST. | RECOVERY | DESCRIPTION OF MATERIAL | UNIT DRY WT. LBS./FT. 3 |
|---|---|---|---|---|---|---|
| ☒ | | | | | SURFACE ELEVATION ↓ 100.4 6.. | |
| | 1 | PA | | | Ash; soil classification-fine to medium sand, trace to some silt(SP-SM)-black-wet to saturated-loose-(Fill) | |
| | 2 | SS | | | | |
| 5 | 3 | SS | | | | |
| | 4 | SS | | | Wood, trace ash-dark brown-loose-(Fill) | |
| 10 | 5 | SS | | | Silty fine sand, trace to some pockets of red clayey silt(ML)-reddish brown-loose-moist to wet-(SM) | |
| | 5A | SS | | | Silty clay, trace sand-red brown-tough-(CL) | |
| | 6 | SS | | | Clayey silt, trace sand-reddish brown-medium dense-very tough-(ML-CL) | |
| 15 | 7 | SS | | | Silt-trace to some clay-red brown-medium dense-very tough-(ML) | |
| | 8 | SS | | | | |
| 20<br>21.0 | 9 | SS | | | Fine to medium sand, trace to some silt, trace gravel-red brown-medium dense-saturated-(SP-SM) | |
| | | | | | End of Boring | |



*Calibrated Penetrometer

Boring advanced by hollow stem auger to a depth of 19.5 feet
No wash water used

WATER LEVEL OBSERVATIONS

| | | | | |
|---|---|---|---|---|
| W.L. | 1.5' W.S. | | | |
| W.L. | 19' | B.C.R. | 2.5' | A.C.R. |
| W.L. | 3.8' 24 hours AB | | | |

**SOIL TESTING SERVICES**
OF WIS., INC.
540 LAMBEAU STREET
GREEN BAY, WIS. 54303

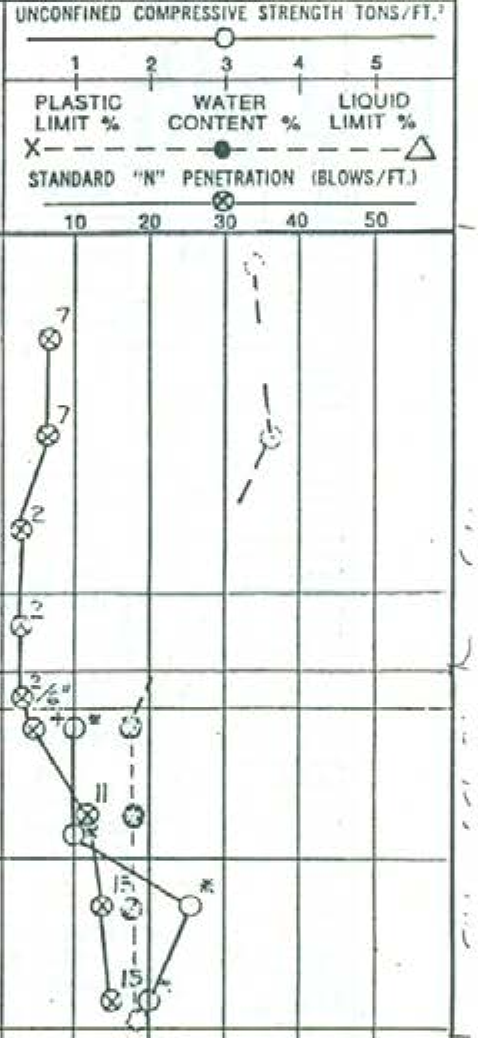
| | | | |
|---|---|---|---|
| BORING STARTED | 3-11-76 | | |
| BORING COMPLETED | 3-11-76 | | |
| RIG | Bomb. | FOREMAN | HH |
| DRAWN | KO | APPROVED | DBE |
| JOB # | 7149 | SHEET | |

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

# LOG OF BORING NO. 2

| | |
|---|---|
| OWNER<br>Lake Superior Power District | ARCHITECT-ENGINEER<br>Barr Engineering Company |
| SITE<br>Ashland, Wisconsin | PROJECT NAME<br>Bayfront Settling Basins |

| DEPTH ELEVATION | SAMPLE NO. | TYPE SAMPLE | SAMPLE DIST. | RECOVERY | DESCRIPTION OF MATERIAL | UNIT DRY WT. LBS./FT. 3 |
|---|---|---|---|---|---|---|
| ☒ | | | | | SURFACE ELEVATION 102.5 | |
| | 1 | PA | | | Ash;soil classification-clayey silt-(ML-CL) to fine to medium sand-(SP) black-loose-wet to saturated-(Fill) | |
| | 2 | SS | | | | |
| 5 | 3 | SS | | | | |
| | 4 | SS | | | | |
| 10 | 5 | SS | | | Wood chips-dark brown-loose-(Fill) | |
| | 6 | SS | | | Silty fine sand-red brown-loose-wet-(SM) | |
| | 6A | SS | | | Silty clay-trace fine sand-trace roots-brown-tough-(CL-CH) | |
| 15 | 7 | SS | | | | |
| | 8 | SS | | | Silt-trace to some clay-reddish brown-medium dense-very tough-(ML) | |
| 20 | 9 | SS | | | | |
| 21.0 | | | | | | |



End of Boring

*Calibrated Penetrometer

Boring advanced by hollow stem auger to 19.5 feet
No wash water used

| WATER LEVEL OBSERVATIONS | |
|---|---|
| W.L. | 4.5' W. S. |
| W.L. | 19.5' B.C.R. \| 6.5' A.C.R. |
| W.L. | 5.3' 24 hours AB |
| | |

**SOIL TESTING SERVICES**
OF WIS., INC.
540 LAMBEAU STREET
GREEN BAY, WIS. 54303

| | | | |
|---|---|---|---|
| BORING STARTED | 3-11-76 | | |
| BORING COMPLETED | 3-11-76 | | |
| RIG | Bomb. | FOREMAN | HH |
| DRAWN | PH | APPROVED | DBE |
| JOB # | 7149 | SHEET | |

The stratification lines represent the approximate boundary

# LOG OF BORING NO. B-3

**OWNER**
Lake Superior Power District

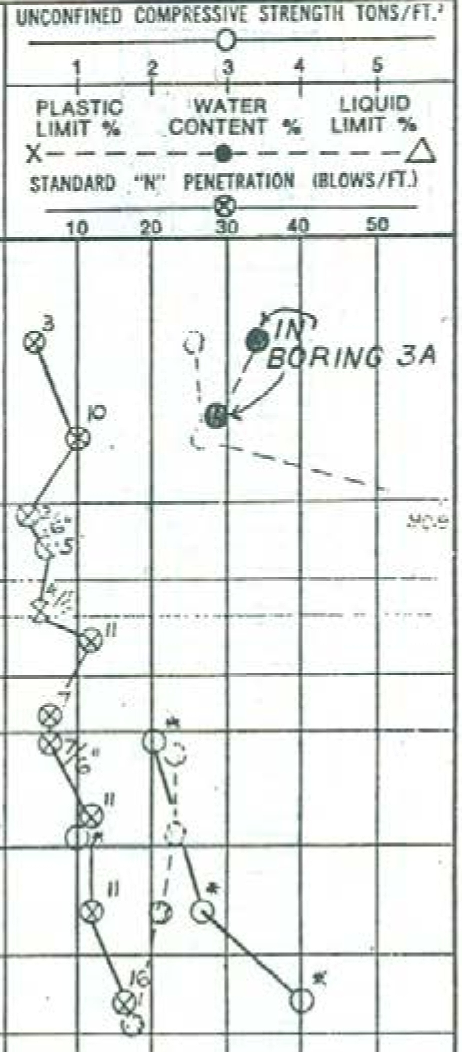
**ARCHITECT-ENGINEER**
Barr Engineering Company

**SITE**
Ashland, Wisconsin

**PROJECT NAME**
Bayfront Settling Basins

SURFACE ELEVATION 101.8

| DEPTH ELEVATION | SAMPLE NO. | TYPE SAMPLE | DESCRIPTION OF MATERIAL | UNIT DRY WT. LBS./FT. 3 |
|---|---|---|---|---|
| | 1 | PA | Ash-soil classification-clayey silt (ML-CL)(0-2') to silty fine medium sand(SM)-black-loose to medium dense saturated to moist-(Fill) | |
| | 2 | SS | | 99 |
| 5 | 3 | SS | | 104 |
| | 4 | SS | Fine medium sand, trace ash-brown and black-loose-(SP-fill) | |
| | 4A | SS | | |
| 10 | 5 | SS | Silty clayey fine sand with wood-brown and black-loose-(SM-SC-Fill) | |
| | 5A | SS | Silt, trace to some fine sand-red brown-medium dense-(ML) | |
| | 6 | SS | Fine medium sand, trace to some silt-red brown-medium dense-moist to wet-(SP-SM) | |
| | 6A | SS | | |
| 15 | 7 | SS | Silty clay, trace fine sand-reddish brown-tough-(CH) | |
| | 8 | SS | Clayey silt, trace fine sand-reddish brown-medium dense-very tough-(CL-ML) | |
| 20 | | | Silt, trace clay-red brown-medium dense-(ML) | |
| 21.0 | 9 | SS | | |
| | | | End of Boring<br>Boring advanced by hollow stem auger<br>No wash water used | |



*Calibrated Penetrometer

WATER LEVEL OBSERVATIONS

| | |
|---|---|
| W.L. | 7.0' WS |
| W.L. | 19.5' B.C.R. \| 6.5' A.C.R. |
| W.L. | 5.0' 24 hours AB |

**SOIL TESTING SERVICES**
OF WIS., INC.
540 LAMBEAU STREET
GREEN BAY, WIS. 54303

| | | | |
|---|---|---|---|
| BORING STARTED | 3-11-76 | | |
| BORING COMPLETED | 3-11-76 | | |
| RIG | Bomb. | FOREMAN | HH |
| DRAWN | PH | APPROVED | DBE |
| JOB # | 7149 | SHEET | |

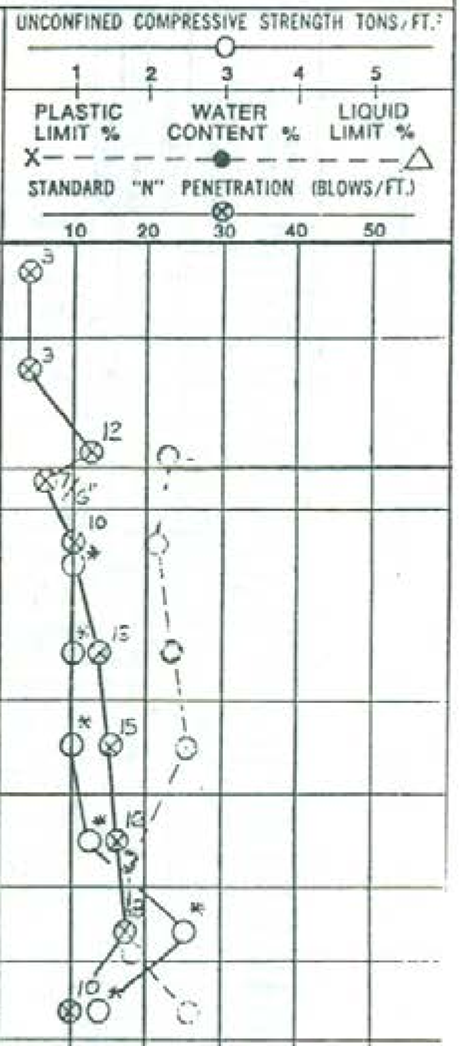
The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

# LOG OF BORING NO. B-4

| | |
|---|---|
| OWNER<br>Lake Superior Power District | ARCHITECT-ENGINEER<br>Barr Engineering Company |
| SITE Ashland, Wisconsin | PROJECT NAME<br>Bayfront Settling Basins |

SURFACE ELEVATION 98.4

| DEPTH ELEVATION | SAMPLE NO. | TYPE SAMPLE | DESCRIPTION OF MATERIAL | UNIT DRY WT. LBS./FT. 3 |
|---|---|---|---|---|
| | 1 | SS | Ash;soil classification-fine to medium sand-trace silt(SP)-black-loose-saturated(0-2') to moist-(Fill) | |
| | 2 | SS | Fine to medium sand, trace wood-brown-loose-saturated-(SP-fill) | |
| 5 | 3 | SS | | |
| | 3A | SS | Silty fine sand-red brown medium dense-moist-(SM) | |
| | 4 | SS | Clayey silt,trace to some pockets of sand-red brown-medium dense-tough-(ML-CL) | |
| 10 | 5 | SS | | |
| | 6 | SS | Silt, trace to some clay-red brown-medium dense-(ML) | |
| 15 | 7 | SS | Clayey silt-red brown-medium dense-(ML-CL) | |
| | 8 | SS | Silty clay-red brown-very tough (CL-ML) | |
| 20<br>21.0 | 9 | SS | Silty clay-reddish brown-tough-(CH) | |



End of Boring

*Calibrated Penetrometer

Boring advanced by hollow stem auger
No wash water used

| WATER LEVEL OBSERVATIONS | | | |
|---|---|---|---|
| W.L. | Dry to 19.5' WD | | |
| W.L. | Dry B.C.R. | 0.5' A.C.R. | |
| W.L. | 2.0' 24 hours AB | | |

**SOIL TESTING SERVICES**
OF WIS., INC.
540 LAMBEAU STREET
GREEN BAY, WIS. 54303

| | |
|---|---|
| BORING STARTED 3-11-76 | |
| BORING COMPLETED 3-11-76 | |
| RIG Bomb. | FOREMAN HH |
| DRAWN PH | APPROVED DBE |
| JOB # 7149 | SHEET |

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

# LOG OF BORING NO. 5

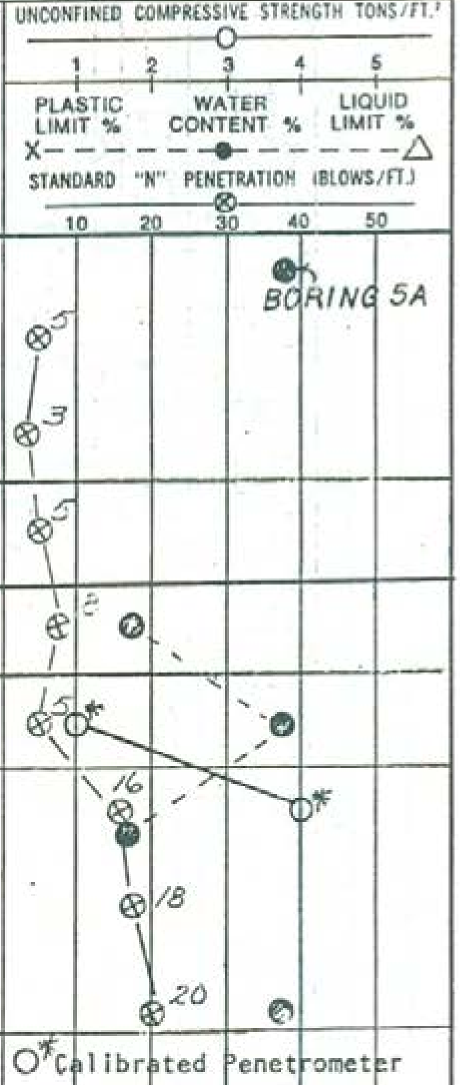
| | |
|---|---|
| OWNER<br>Lake Superior Power District | ARCHITECT-ENGINEER<br>Barr Engineering Company |
| SITE<br>Ashland, Wisconsin | PROJECT NAME<br>Bayfront Settling Basins |

SURFACE ELEVATION 100.9

| DEPTH ELEVATION | SAMPLE NO. | TYPE SAMPLE | DESCRIPTION OF MATERIAL | UNIT DRY WT. LBS./FT. 3 |
|---|---|---|---|---|
| | 1 | HA | Ash;soil classification-silty clay(CL) (0-2 ft.) silty fine sand(SM)(2-4.5 ft) fine to coarse sand(SW) (4.5-6.5 ft.) black-loose-saturated-(Fill) | 78 |
| | 2 | SS | | |
| 5 | 3 | SS | | |
| | 4 | SS | Wood, trace ash-dark brown-loose-Fill | |
| 10 | 5 | SS | Fine sand, trace to some silt-brown-loose-moist to wet-(SP-SM) | |
| | 6 | SS | Silty clay-reddish brown-tough-(CL-CH) | |
| 15 | 7 | SS | Clayey silt-reddish brown-medium dense-very tough-(ML-CL) | |
| | 8 | SS | | |
| 20<br>21.0 | 9 | SS | | |

End of Boring

Boring advanced by hollow stem auger
No wash water used



| WATER LEVEL OBSERVATIONS | | |
|---|---|---|
| W.L. | | |
| W.L. | Dry to 21.0' B.C.R. | 4.0' A.C.R. |
| W.L. | 4.0' 24 hours AB | |

**SOIL TESTING SERVICES**
OF WIS., INC.
540 LAMBEAU STREET
GREEN BAY, WIS. 54303

| | |
|---|---|
| BORING STARTED 3-12-76 | |
| BORING COMPLETED 3-12-76 | |
| RIG Bomb. | FOREMAN HH |
| DRAWN KO | APPROVED DBE |
| JOB # 7149 | SHEET |

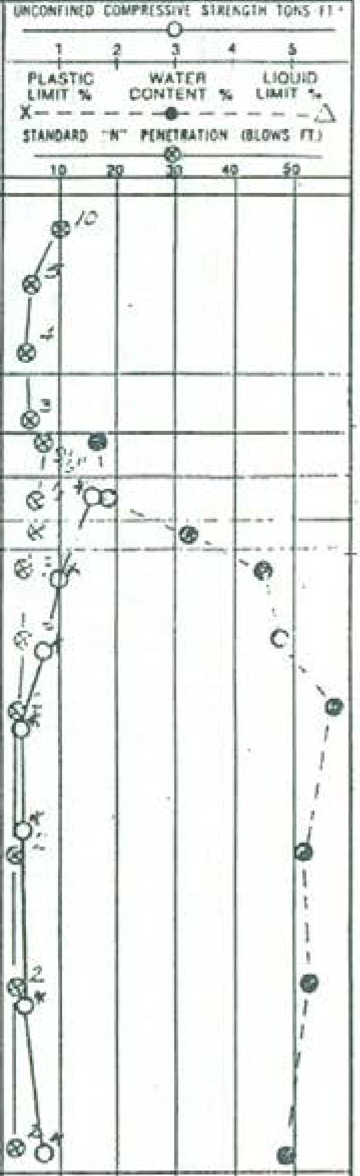
The stratification lines represent the approximate boundary

# LOG OF BORING NO. 6

| | |
|---|---|
| OWNER<br>Lake Superior Power District | ARCHITECT-ENGINEER<br>Barr Engineering Company |
| SITE<br>Ashland, Wisconsin | PROJECT NAME<br>Bayfront Settling Basins |

SURFACE ELEVATION → 100.4

| DEPTH ELEVATION | SAMPLE NO. | TYPE SAMPLE | DESCRIPTION OF MATERIAL |
|---|---|---|---|
| ☒ | | HA | Concrete rubble fill |
| | 1 | SS | Ash;soil classification-silty fine sand(SM) to fine to coarse sand-(SW) black-medium dense to loose-moist to saturated-(Fill) |
| | 2 | SS | |
| 5 | | | |
| | 3 | SS | |
| | 4 | SS | Wood with clayey silt-reddish brown-loose-(Fill) |
| | 4A | SS | Sandy silt-trace clay-reddish brown-loose-(SM-ML) |
| 10 | | | |
| | 5 | SS | Silty clay, trace sand and roots-reddish brown-tough-(CL-ML) |
| | 5A | SS | Silty clay-reddish brown-tough-(CL-CH) |
| | 6 | SS | Silty clay-reddish brown-tough to soft-(CH) |
| 15 | | | |
| | 7 | SS | |
| | 8 | SS | |
| 20 | | | |
| | 9 | SS | |
| 25 | | | |
| | 10 | SS | |
| 30 | | | |
| | 11 | SS | |
| 35 | | | |



O* Calibrated Penetrometer

End of Boring

Boring advanced by hollow stem auger
No wash water used

| WATER LEVEL OBSERVATIONS | | | |
|---|---|---|---|
| W.L. | | | |
| W.L. | Dry to 35.0' | B.C.R. 2.0' | A.C.R. |
| W.L. | | | |

**SOIL TESTING SERVICES**
OF WIS., INC.
540 LAMBEAU STREET
GREEN BAY, WIS. 54303

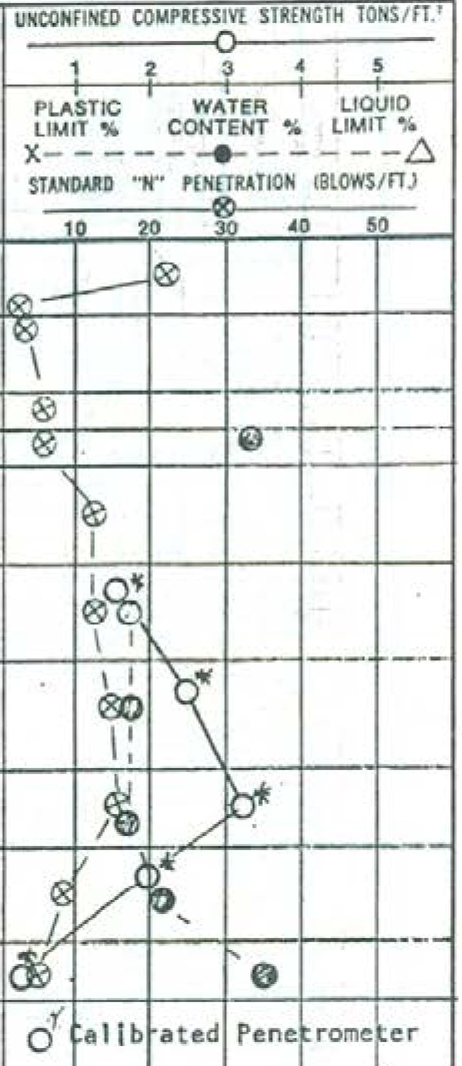
| | | | |
|---|---|---|---|
| BORING STARTED | | 3-10-76 | |
| BORING COMPLETED | | 3-11-76 | |
| RIG | Bomb. | FOREMAN | HH |
| DRAWN | KO | APPROVED | DBE |
| JOB # | 7149 | SHEET | |

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

# LOG OF BORING NO. 7

| | |
|---|---|
| OWNER<br>Lake Superior Power District | ARCHITECT-ENGINEER<br>Barr Engineering Company |
| SITE<br>Ashland, Wisconsin | PROJECT NAME<br>Bayfront Settling Basins |

| DEPTH ELEVATION | SAMPLE NO. | TYPE SAMPLE | SAMPLE DIST. | RECOVERY | DESCRIPTION OF MATERIAL | UNIT DRY WT. LBS./FT. 3 |
|---|---|---|---|---|---|---|
| ☒ | | | | | SURFACE ELEVATION ↴ 97.2 | |
| | 1 | SS | | | Silty fine sand with sandstone fragments-dark red-medium dense-wet-(SM-fill) | |
| | 2 | SS | | | | |
| | 2A | SS | | | Fine medium sand, trace silt and gravel-brown-loose-wet-(SP-fill) | |
| 5 | 3 | SS | | | Wood-loose-(Fill) | |
| | 3A | SS | | | Clayey silt with 1/8 inch thick silty sand seams, trace wood, brown-medium dense-(ML) | |
| | 4 | SS | | | Very fine to fine sand, trace silt-reddish brown-medium dense-moist-(SP) | |
| 10 | 5 | SS | | | Silt-trace to some clay-reddish brown-medium dense-(ML) | |
| | 6 | SS | | | Clayey silt-reddish brown-medium dense-(ML-CL) | |
| 15 | 7 | SS | | | Silt-trace clay-reddish brown medium dense-(ML) | |
| | 8 | SS | | | Clayey silt-reddish brown-loose-(ML-CL) | |
| 20 | 9 | SS | | | Silty clay-reddish brown-soft-(CH) | |
| | | | | | End of Boring<br>Boring advanced by hollow stem augers<br>No wash water used | |



| WATER LEVEL OBSERVATIONS | | |
|---|---|---|
| W.L. | | |
| W.L. | 5.0' B.C.R. | 0.0' A.C.R. |
| W.L. | | |

**SOIL TESTING SERVICES**
OF WIS., INC.
540 LAMBEAU STREET
GREEN BAY, WIS. 54303

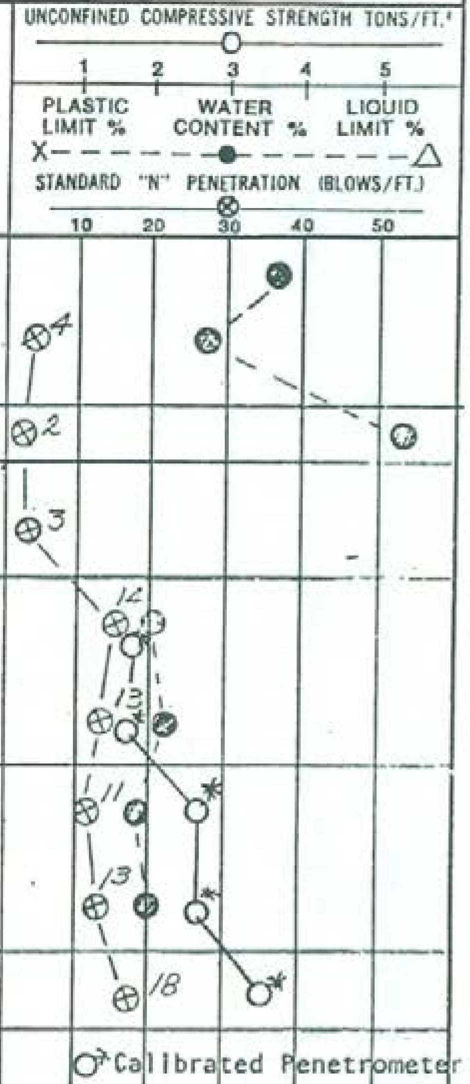
| | | | |
|---|---|---|---|
| BORING STARTED | 3-13-76 | | |
| BORING COMPLETED | 3-13-76 | | |
| RIG | Bomb. | FOREMAN | HH |
| DRAWN | KO | APPROVED | DBE |
| JOB # | 7149 | SHEET | |

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

# LOG OF BORING NO. 8

| | |
|---|---|
| OWNER<br>Lake Superior Power District | ARCHITECT-ENGINEER<br>Barr Engineering Company |
| SITE<br>Ashland, Wisconsin | PROJECT NAME<br>Bayfront Settling Basins |

| DEPTH ELEVATION | SAMPLE NO. | TYPE SAMPLE | SAMPLE DIST. | RECOVERY | DESCRIPTION OF MATERIAL | UNIT DRY WT. LBS./FT. 3 |
|---|---|---|---|---|---|---|
| | | | | | SURFACE ELEVATION 101.3 | |
| | 1 | HA | | | Ash soil classification-clayey silt(ML-CL) (0-2 ft.) silty fine to medium sand(SM) (2-4.5 ft.) | |
| | 2 | SS | | | | |
| 5 | 3 | SS | | | Organic silt,trace to some silty fine sand-brown and black-loose-wet-(CL-SM-fill) | |
| | | | | | Wood-brown to black-loose-(Fill) | |
| | 4 | SS | | | | |
| 10 | 5 | SS | | | Clayey silt-reddish brown-medium dense-tough-(ML-CL) | |
| | 6 | SS | | | | |
| 15 | 7 | SS | | | Silty clay-reddish brown-very tough medium dense-(CL-ML) | |
| | 8 | SS | | | | |
| 20 | | | | | Clayey silt-reddish brown-medium dense-very tough-(ML-CL) | |
| 21.0 | | | | | End of Boring<br>Boring advanced by hollow stem auger<br>No wash water used | |



| WATER LEVEL OBSERVATIONS | |
|---|---|
| W.L. | |
| W.L. | Dry to 21.0' B.C.R. 15.0' A.C.R. |
| W.L. | 4.5' 1 hour AB |

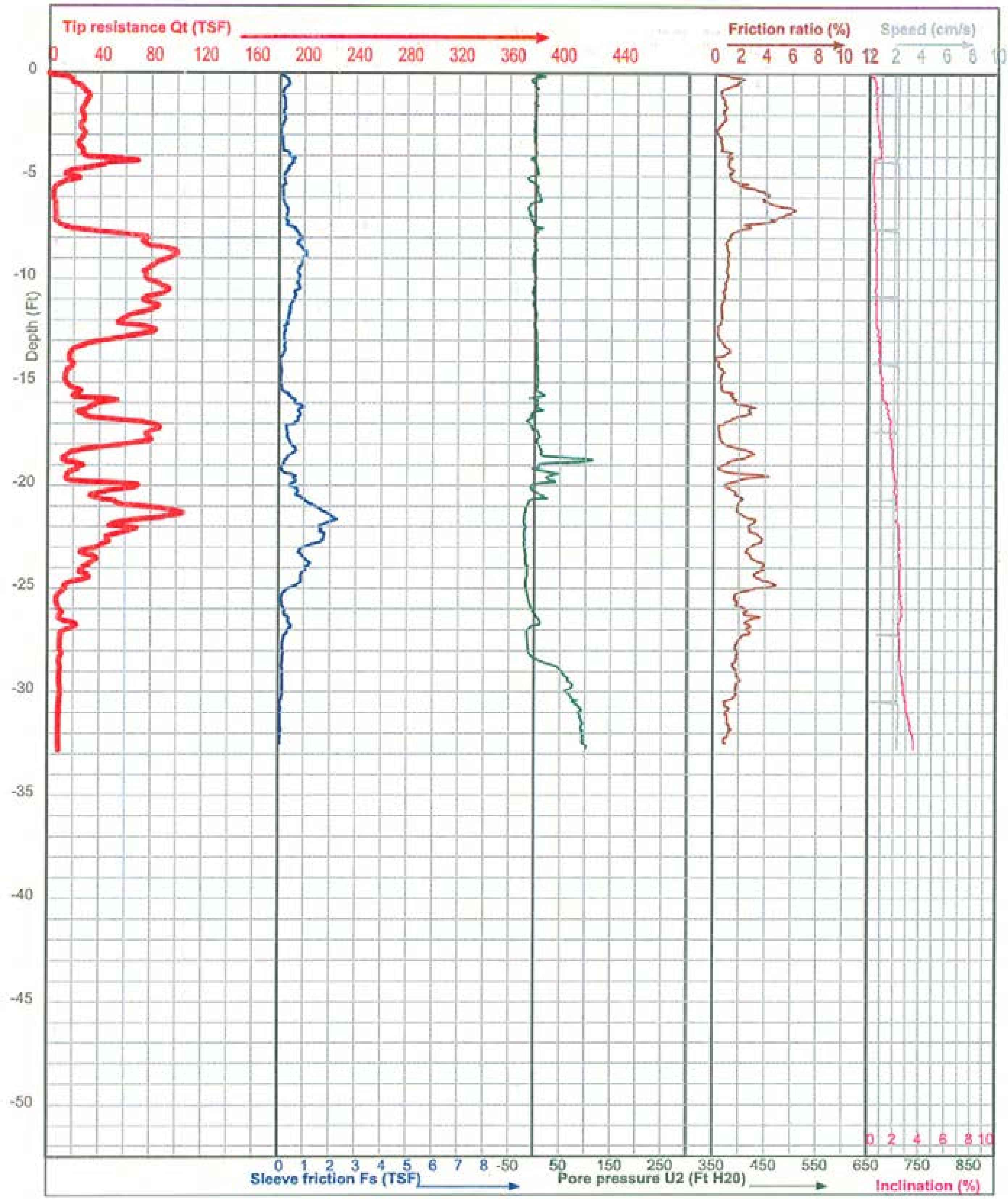
**SOIL TESTING SERVICES**
OF WIS., INC.
540 LAMBEAU STREET
GREEN BAY, WIS. 54303

| | | | |
|---|---|---|---|
| BORING STARTED | 3-13-76 | | |
| BORING COMPLETED | 3-13-76 | | |
| RIG | Bomb. | FOREMAN | HH |
| DRAWN | KO | APPROVED | DBE |
| JOB # | 7149 | SHEET | |

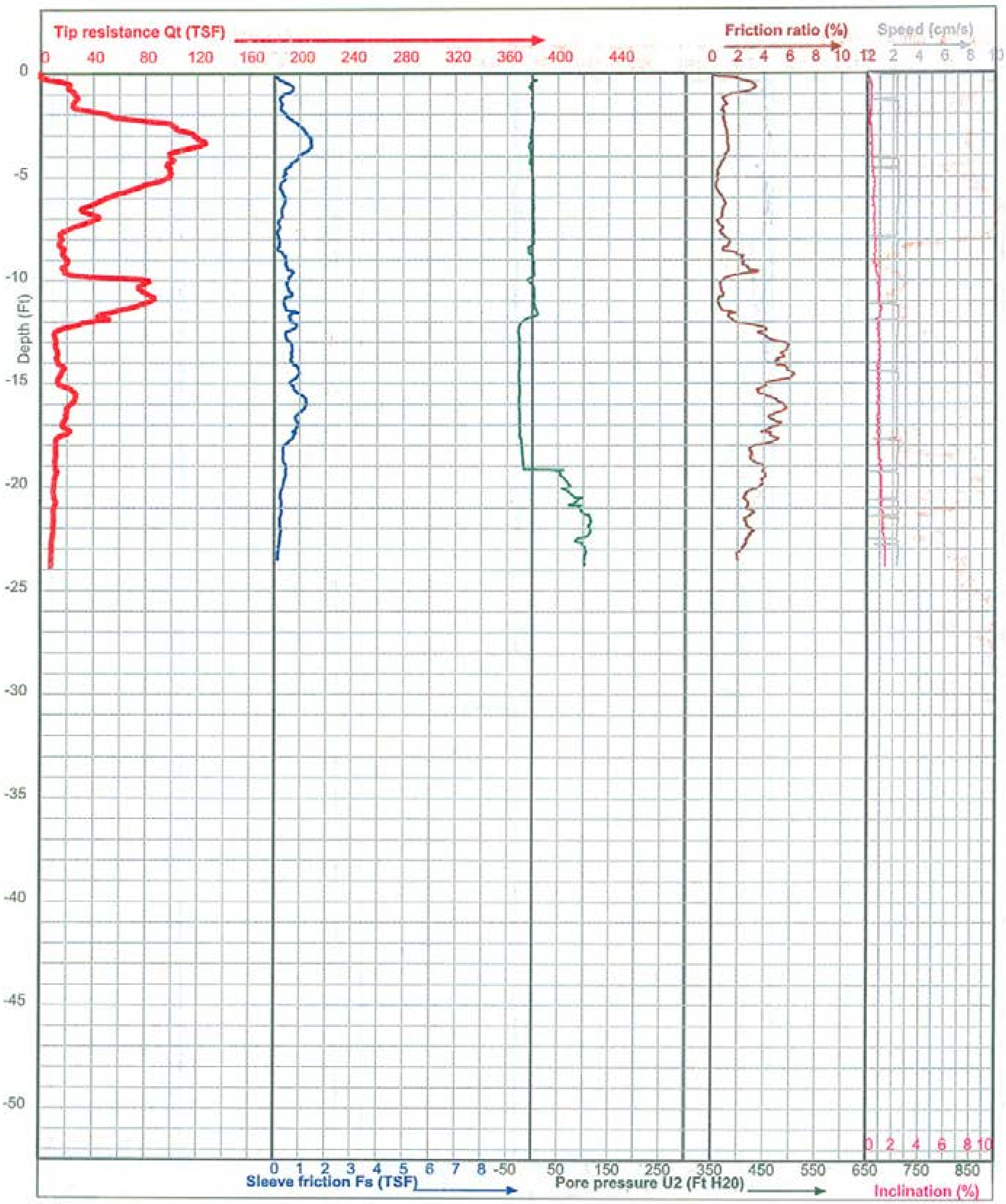
The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

# Appendix B

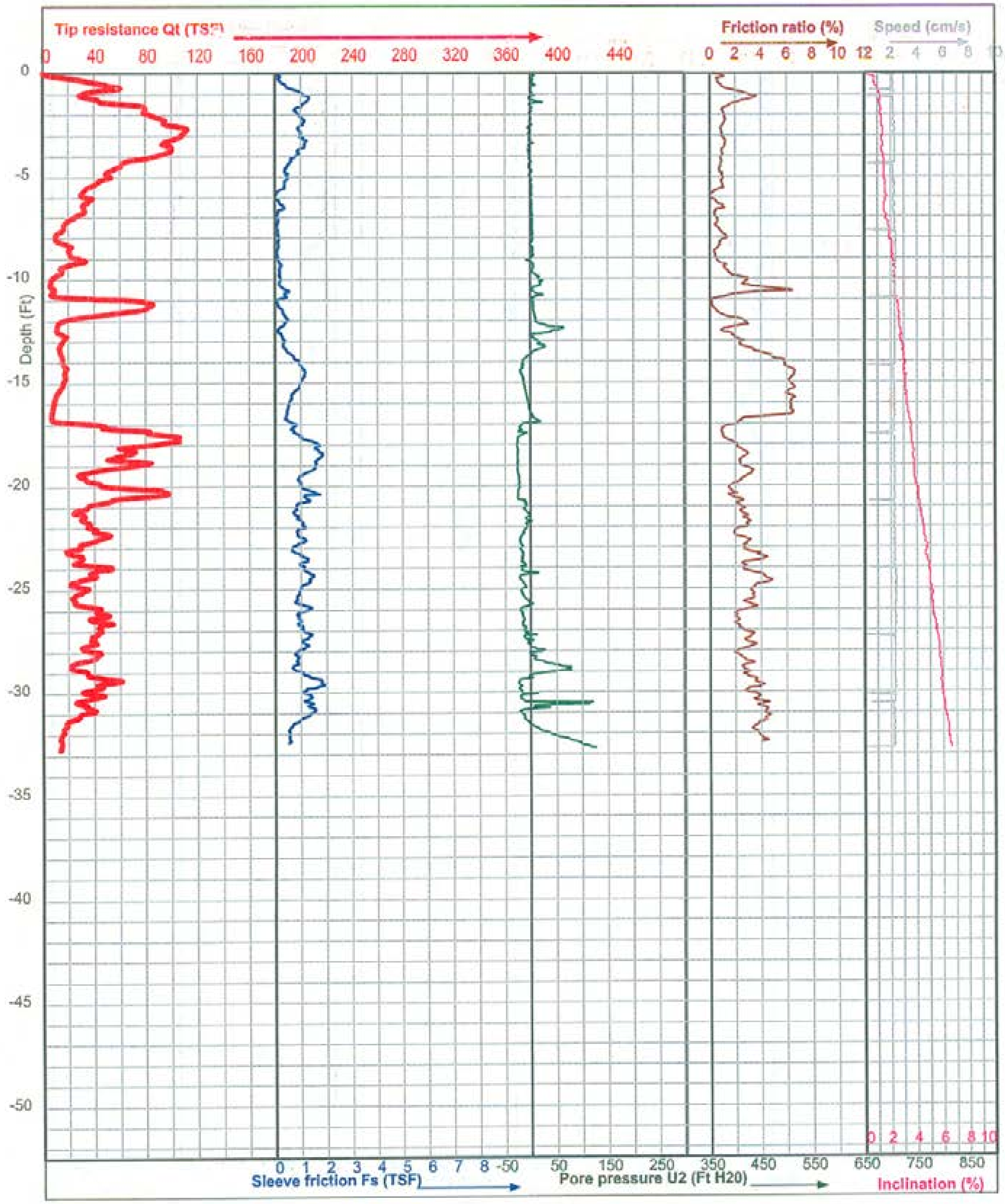
## 2012 CPT Logs



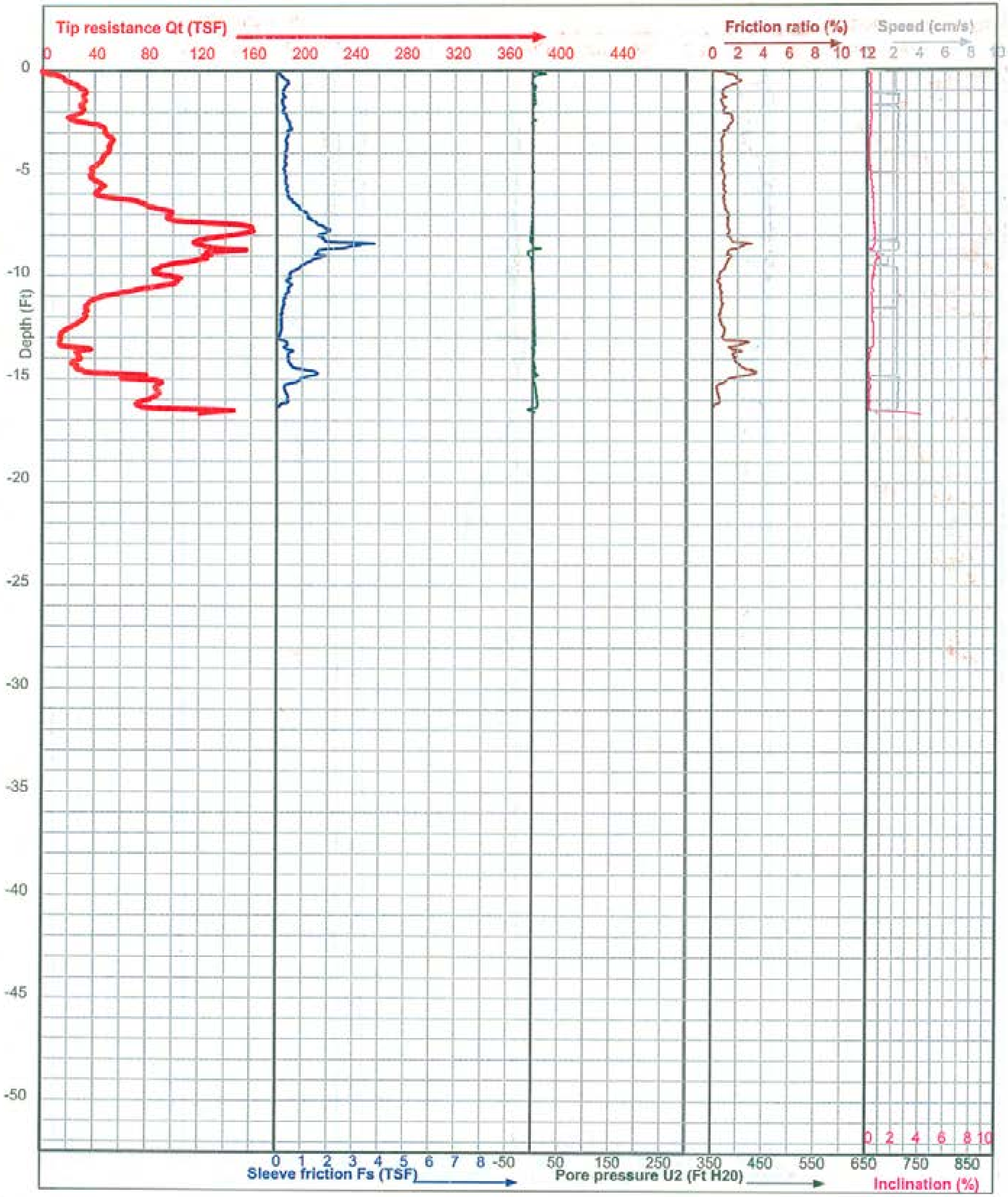
| | | | | | |
|---|---|---|---|---|---|
| Project | Xcel/Bayfront Settling Ponds | CPT Operator | M. Brassington | Elevation (Ft MSL) | |
| Location | CPT-2 | Cone Type | I-CFXYP20-15 | | |
| Test Number | 2 | Cone ID | 111038 | X-Coord | -90.90307 |
| Client | Barr Engineering Co., Inc. | Start Time | 10:04 | Y-Coord | 46.58557 |
| Boring Type | CPTU Sounding | Date | 27-6-2012 | Elev. Datum: WGS 84 | |



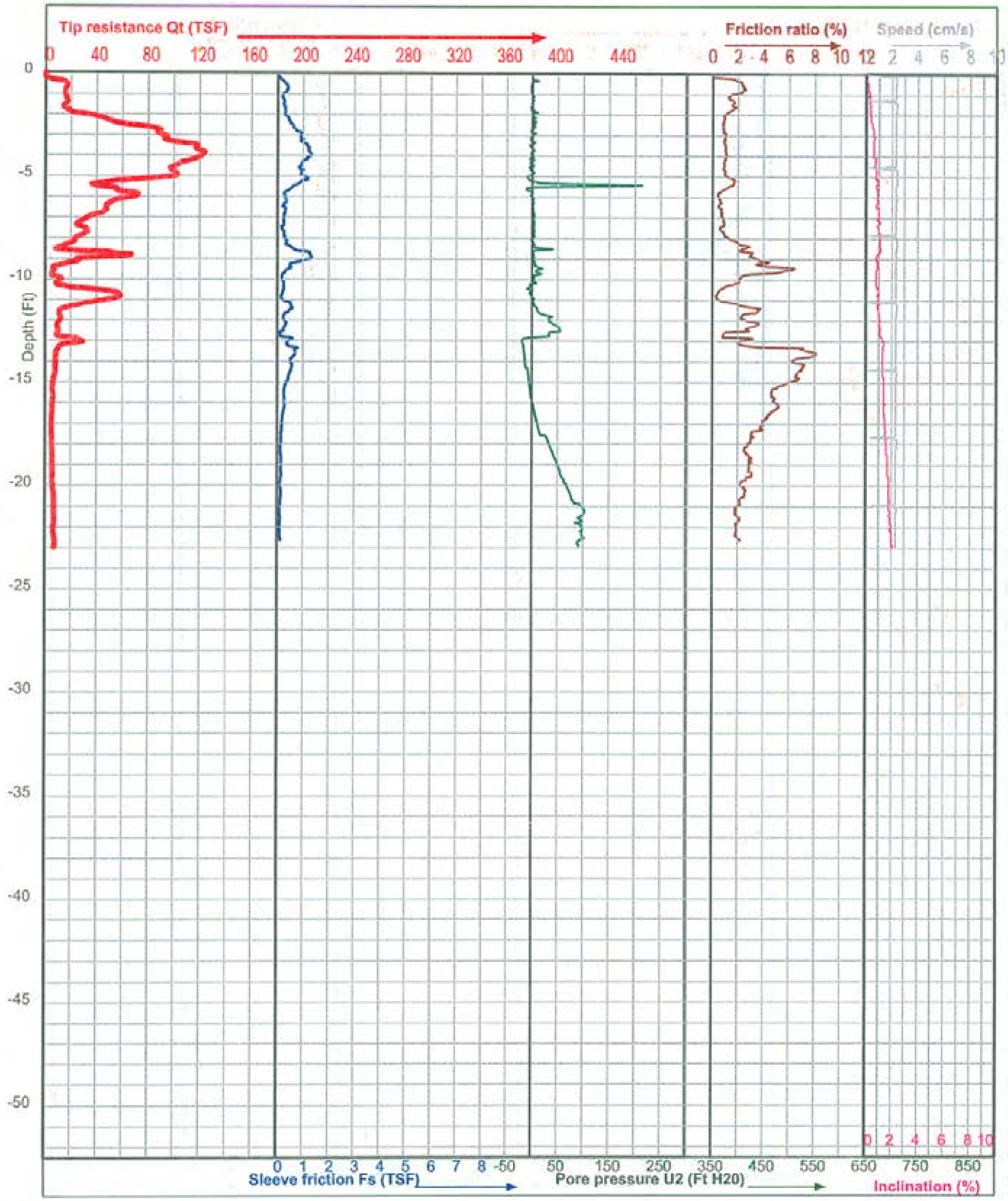
| | | | | | |
|---|---|---|---|---|---|
| Project | Xcel/Bayfront Settling Ponds | CPT Operator | M. Brassington | Elevation (Ft MSL) | |
| Location | CPT- 4 | Cone Type | I-CFXYP20-15 | | |
| Test Number | 4 | Cone ID | 111037 | X-Coord | -90.90355 |
| Client | Barr Engineering Co., Inc. | Start Time | 11:08 | Y-Coord | 46.58524 |
| Boring Type | CPTU Sounding | Date | 27-6-2012 | Elev. Datum: WGS 84 | |



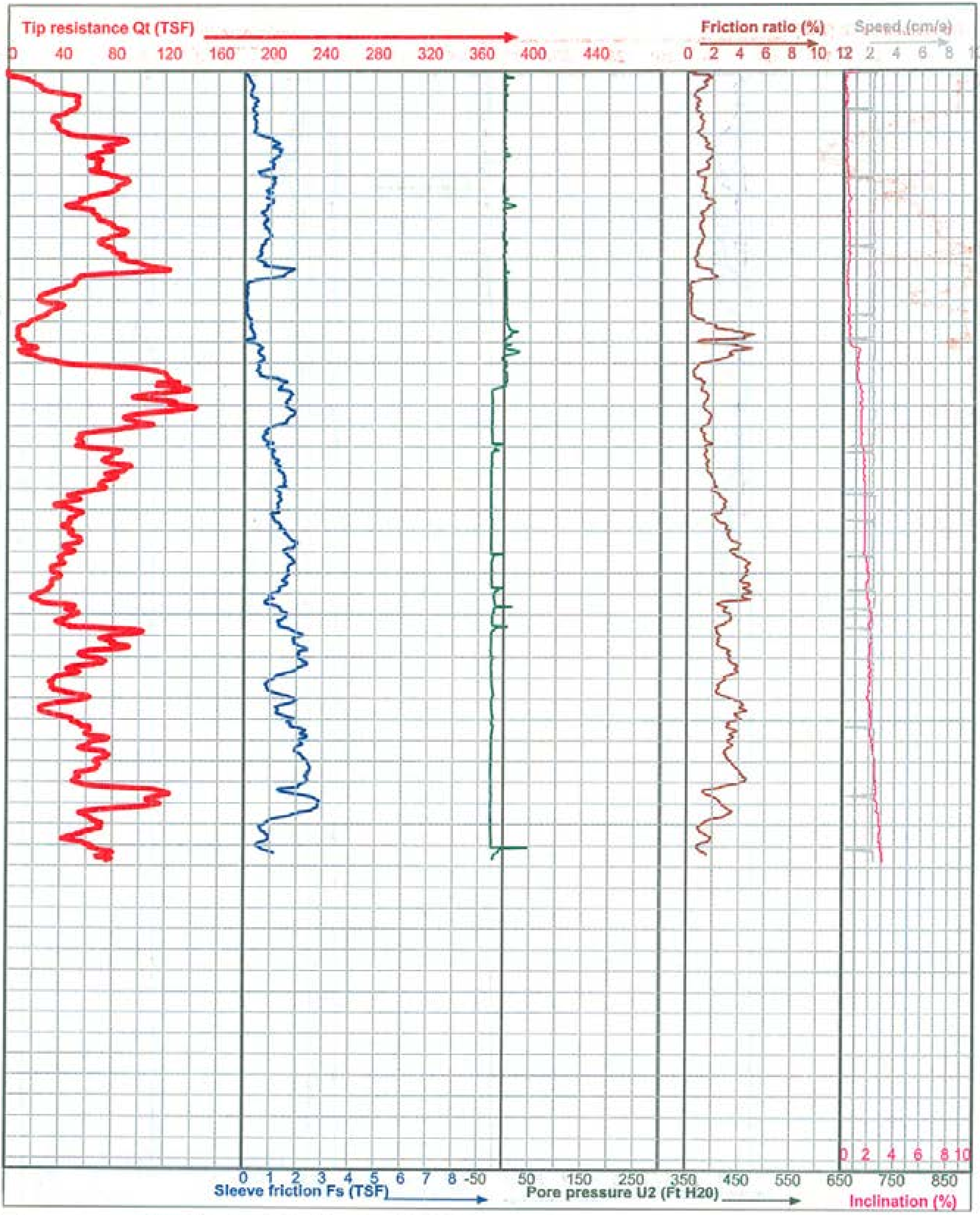
| | | | | | |
|---|---|---|---|---|---|
| Project | Xcel/Bayfront Settling Ponds | CPT Operator | M. Brassington | Elevation (Ft MSL) | |
| Location | CPT-1 | Cone Type | I-CFXYP20-15 | | |
| Test Number | 1 | Cone ID | 090709 | X-Coord | -90.90273 |
| Client | Barr Engineering Co., Inc. | Start Time | 9:19 | Y-Coord | 46.58580 |
| Boring Type | CPTU Sounding | Date | 27-6-2012 | Elev. Datum: WGS 84 | |



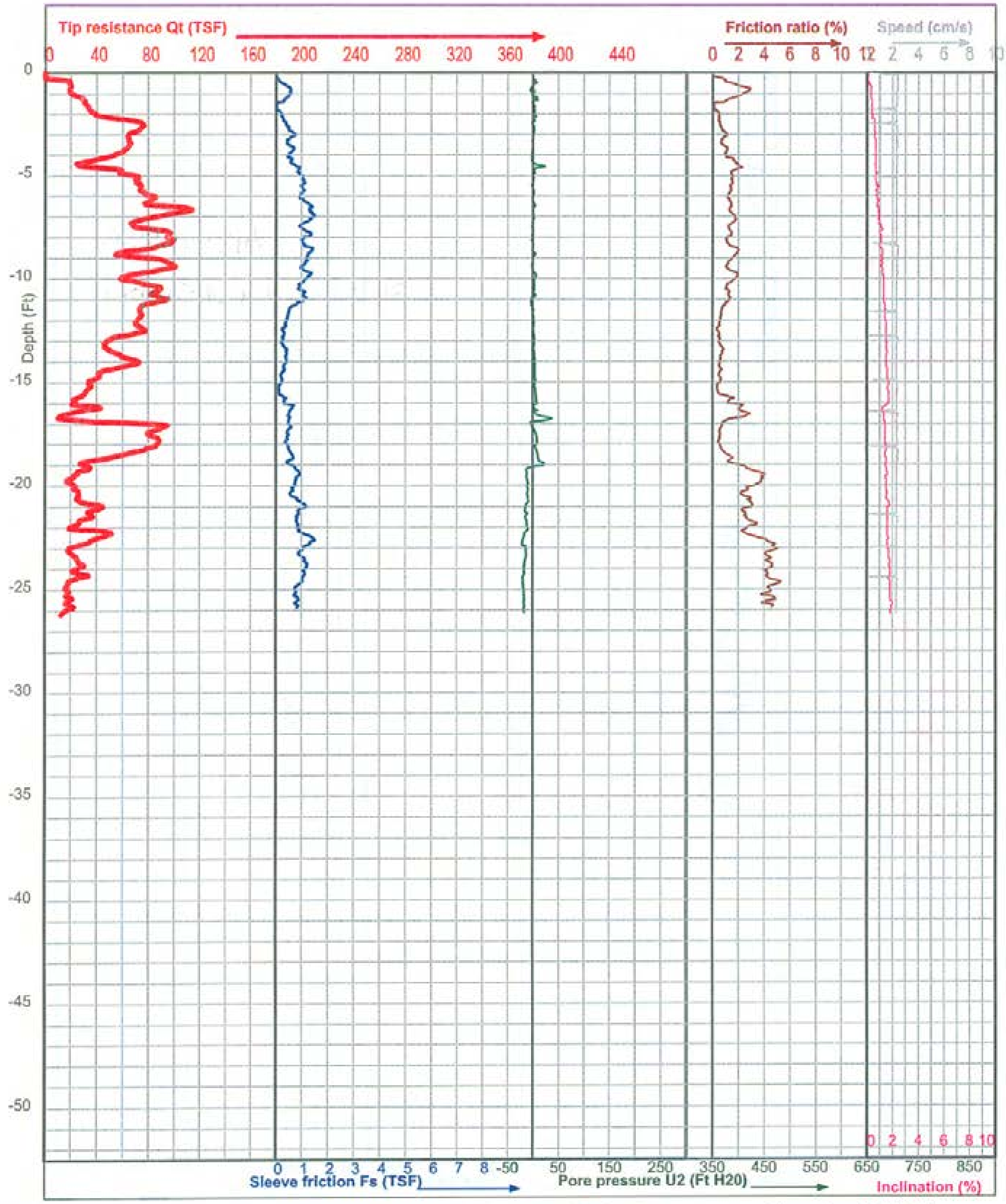
| | | | | | |
|---|---|---|---|---|---|
| Project | Xcel/Bayfront Settling Ponds | CPT Operator | M. Brassington | Elevation (Ft MSL) | |
| Location | CPT- 3 | Cone Type | I-CFXYP20-15 | | |
| Test Number | 3 | Cone ID | 090710 | X-Coord | -90.90352 |
| Client | Barr Engineering Co., Inc. | Start Time | 10:45 | Y-Coord | 46.58531 |
| Boring Type | CPTU Sounding | Date | 27-6-2012 | Elev. Datum: WGS 84 | |



| | | | | | |
|---|---|---|---|---|---|
| Project | Xcel/Bayfront Settling Ponds | CPT Operator | M. Brassington | Elevation (Ft MSL) | |
| Location | CPT-5 | Cone Type | I-CFXYP20-15 | | |
| Test Number | 5 | Cone ID | 111038 | X-Coord | -90.90328 |
| Client | Barr Engineering Co., Inc. | Start Time | 11:49 | Y-Coord | 46.58508 |
| Boring Type | CPTU Sounding | Date | 27-6-2012 | Elev. Datum: WGS 84 | |



| | | | | | |
|---|---|---|---|---|---|
| Project | Xcel/Bayfront Settling Ponds | CPT Operator | M. Brassington | Elevation (Ft MSL) | |
| Location | CPT-6 | Cone Type | I-CFXYP20-15 | | |
| Test Number | 6 | Cone ID | 090710 | X-Coord | -90.90252 |
| Client | Barr Engineering Co., Inc. | Start Time | 12:19 | Y-Coord | 46.58523 |
| Boring Type | CPTU Sounding | Date | 27-6-2012 | Elev. Datum: WGS 84 | |



| | | | | | |
|---|---|---|---|---|---|
| Project | Xcel/Bayfront Settling Ponds | CPT Operator | H. Garcia | Elevation (Ft MSL) | |
| Location | CPT-7 | Cone Type | I-CFXYP20-15 | | |
| Test Number | 7 | Cone ID | 111038 | X-Coord | -90.90261 |
| Client | Barr Engineering Co., Inc. | Start Time | 13:28 | Y-Coord | 46.58566 |
| Boring Type | CPTU Sounding | Date | 27-6-2012 | Elev. Datum: WGS 84 | |

# Appendix C

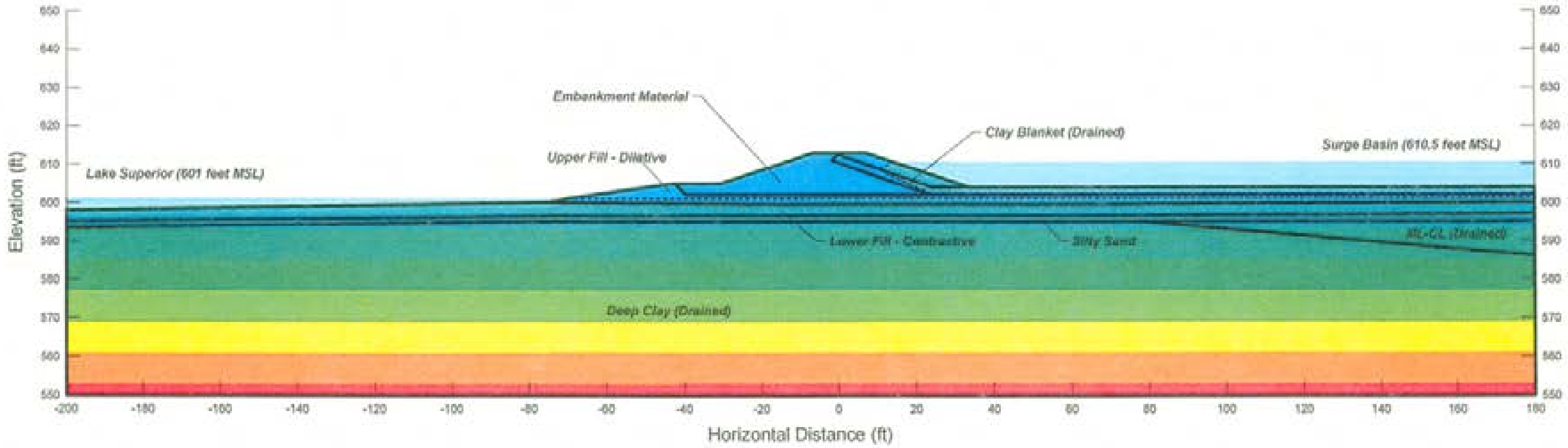
## Stability Model Output

# Seepage Analysis

## Settling Ponds Analyses
## Xcel Energy
## Bayfront Generating Facility
## Ashland, Wisconsin

## Steady-State Seepage

Name: Embankment Material Model: Saturated Only K-Sat: 3.9e-006 ft/sec Volumetric Water Content: 0 $ft^3/ft^3$ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °
Name: Upper Fill - Dilative Model: Saturated Only K-Sat: 0.00082 ft/sec Volumetric Water Content: 0 $ft^3/ft^3$ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °
Name: Clay Blanket (Drained) Model: Saturated Only K-Sat: 3.3e-009 ft/sec Volumetric Water Content: 0 $ft^3/ft^3$ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °
Name: Deep Clay (Drained) Model: Saturated Only K-Sat: 3.3e-010 ft/sec Volumetric Water Content: 0 $ft^3/ft^3$ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °
Name: Silty Sand Model: Saturated Only K-Sat: 3.3e-006 ft/sec Volumetric Water Content: 0 $ft^3/ft^3$ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °
Name: ML-CL (Drained) Model: Saturated Only K-Sat: 3.3e-009 ft/sec Volumetric Water Content: 0 $ft^3/ft^3$ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °
Name: Lower Fill - Contractive Model: Saturated Only K-Sat: 0.00082 ft/sec Volumetric Water Content: 0 $ft^3/ft^3$ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °



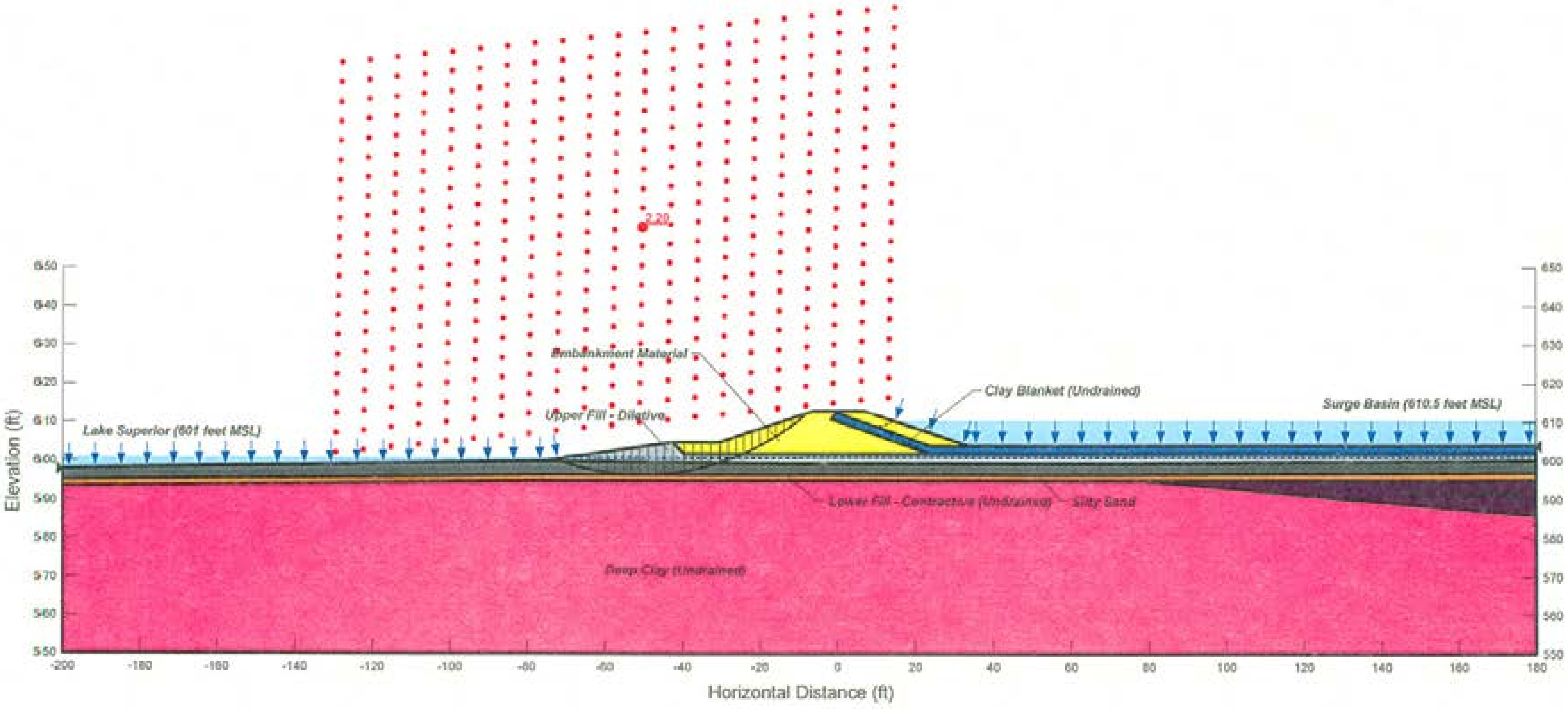
Directory: P:\Mpls\49 WI\02\49021000 Bayfront Settling Ponds Analys\WorkFiles\Bayfront\

# USSA Analysis

## Settling Ponds Analyses
## Xcel Energy
## Bayfront Generating Facility
## Ashland, Wisconsin

### Static - Undrained

Name: Embankment Material Model: Mohr-Coulomb Unit Weight: 110 pcf Unit Wt. Above Water Table: 110 pcf Cohesion: 0 psf Phi: 42 ° Phi-B: 0 °
Name: Upper Fill - Dilative Model: Mohr-Coulomb Unit Weight: 107 pcf Unit Wt. Above Water Table: 90 pcf Cohesion: 0 psf Phi: 38 ° Phi-B: 0 °
Name: Silty Sand Model: Mohr-Coulomb Unit Weight: 105 pcf Cohesion: 0 psf Phi: 40 ° Phi-B: 0 °
Name: Clay Blanket (Undrained) Model: Undrained (Phi=0) Unit Weight: 133 pcf Unit Wt. Above Water Table: 120 pcf Cohesion: 500 psf
Name: Deep Clay (Undrained) Model: Undrained (Phi=0) Unit Weight: 125 pcf Unit Wt. Above Water Table: 110 pcf Cohesion: 800 psf
Name: ML-CL (Undrained) Model: Undrained (Phi=0) Unit Weight: 125 pcf Cohesion: 800 psf
Name: Lower Fill - Contractive (Undrained) Model: S=f(overburden) Unit Weight: 107 pcf Tau/Sigma Ratio: 0.22 Minimum Strength: 0



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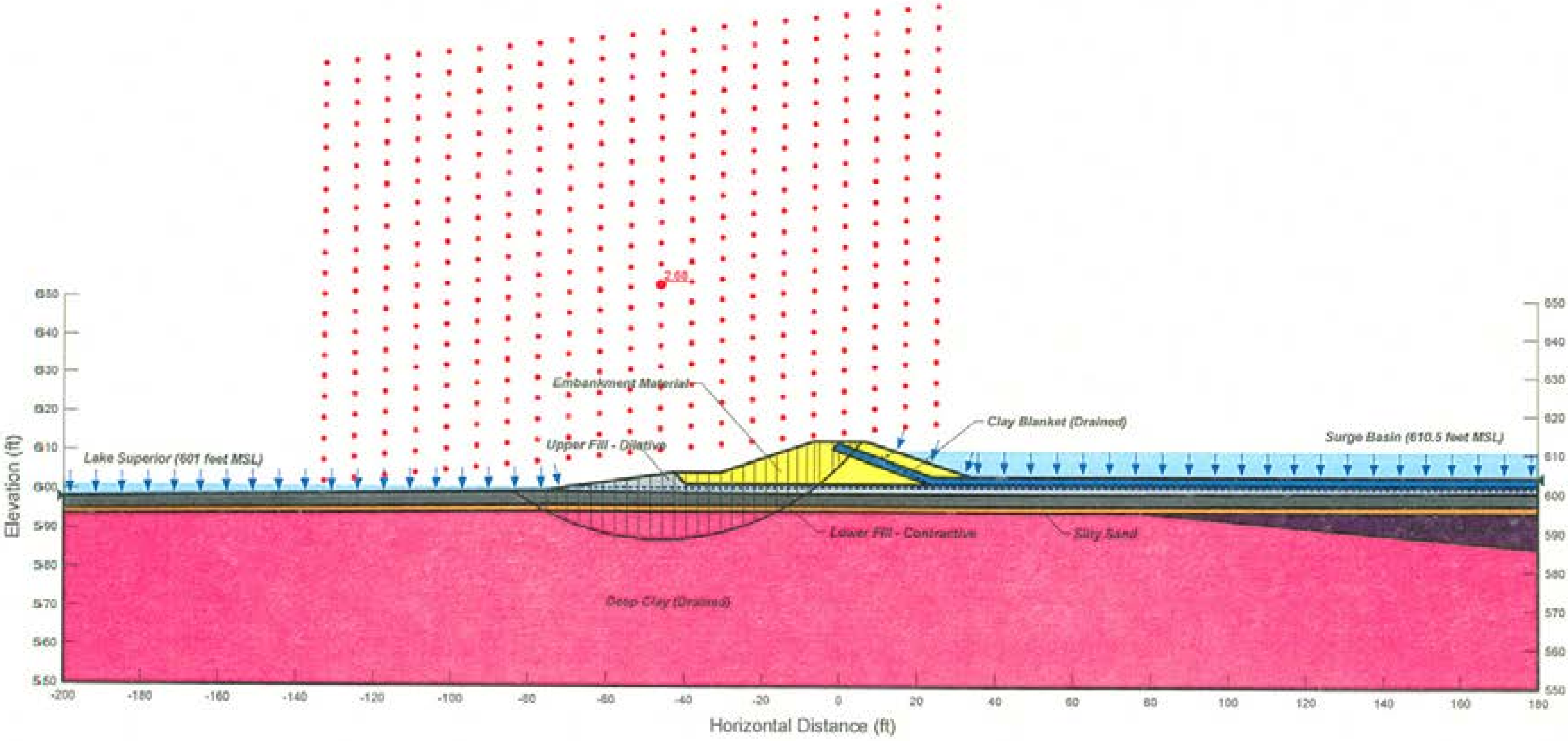
# ESSA Analysis

**Settling Ponds Analyses**
**Xcel Energy**
**Bayfront Generating Facility**
**Ashland, Wisconsin**

**Static - Drained**

Name: Embankment Material Model: Mohr-Coulomb Unit Weight: 110 pcf Unit Wt. Above Water Table: 110 pcf Cohesion: 0 psf Phi: 42 ° Phi-B: 0 °
Name: Upper Fill - Dilative Model: Mohr-Coulomb Unit Weight: 107 pcf Unit Wt. Above Water Table: 90 pcf Cohesion: 0 psf Phi: 38 ° Phi-B: 0 °
Name: Clay Blanket (Drained) Model: Mohr-Coulomb Unit Weight: 133 pcf Unit Wt. Above Water Table: 120 pcf Cohesion: 0 psf Phi: 26 ° Phi-B: 0 °
Name: Deep Clay (Drained) Model: Mohr-Coulomb Unit Weight: 125 pcf Unit Wt. Above Water Table: 110 pcf Cohesion: 0 psf Phi: 24 ° Phi-B: 0 °
Name: Silty Sand Model: Mohr-Coulomb Unit Weight: 105 pcf Cohesion: 0 psf Phi: 40 ° Phi-B: 0 °
Name: ML-CL (Drained) Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 0 psf Phi: 28 ° Phi-B: 0 °
Name: Lower Fill - Contractive Model: Mohr-Coulomb Unit Weight: 107 pcf Cohesion: 0 psf Phi: 32 ° Phi-B: 0 °



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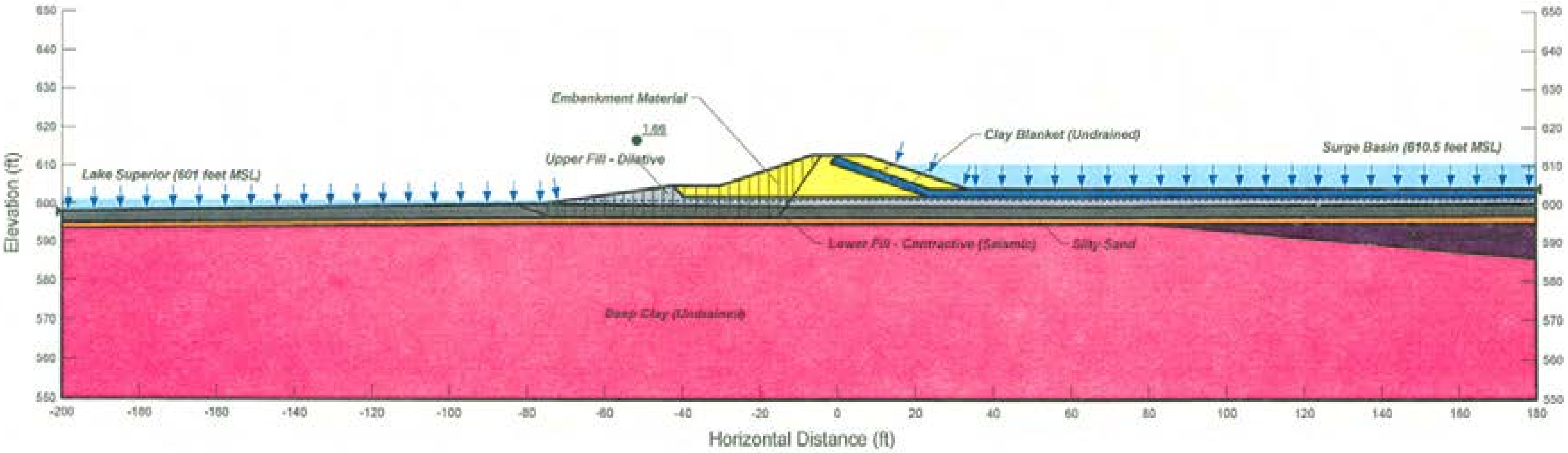
# Seismic Analysis

## Settling Ponds Analyses
## Xcel Energy
## Bayfront Generating Facility
## Ashland, Wisconsin

## Seismic Triggering

Name: Embankment Material Model: Mohr-Coulomb Unit Weight: 110 pcf Unit Wt. Above Water Table: 110 pcf Cohesion: 0 psf Phi: 42 ° Phi-B: 0 °
Name: Upper Fill - Dilative Model: Mohr-Coulomb Unit Weight: 107 pcf Unit Wt. Above Water Table: 90 pcf Cohesion: 0 psf Phi: 38 ° Phi-B: 0 °
Name: Silty Sand Model: Mohr-Coulomb Unit Weight: 105 pcf Cohesion: 0 psf Phi: 40 ° Phi-B: 0 °
Name: Clay Blanket (Undrained) Model: Undrained (Phi=0) Unit Weight: 133 pcf Unit Wt. Above Water Table: 120 pcf Cohesion: 500 psf
Name: Deep Clay (Undrained) Model: Undrained (Phi=0) Unit Weight: 125 pcf Unit Wt. Above Water Table: 110 pcf Cohesion: 800 psf
Name: ML-CL (Undrained) Model: Undrained (Phi=0) Unit Weight: 125 pcf Cohesion: 800 psf
Name: Lower Fill - Contractive (Seismic) Model: S=f(overburden) Unit Weight: 107 pcf Tau/Sigma Ratio: 0.22 Minimum Strength: 0



Directory: P:\Mpls\49 WI\02\49021009 Bayfront Settling Ponds Analysis\WorkFiles\Bayfront

**Bay Front Settling Ponds**
**Olson Methodology for Liquefaction Triggering Analysis**
**Steady State Conditions - Static Triggering**
***File: Bayfront Settling Ponds_BL_Deep_09112012.gsz***
*Analysis: Triggering Analysis Stability USSA - Right to Left Deep Slip Surface*

**REF: Olson and Stark 2003 Yield Strength Ratio and Liquefaction Analysis of Slopes and Embankments**
**REF: Olson 2009 U of Ida Geotech Conference**

| Material | Slice # | Distance (ft) | Step 1<br>FROM FOS=1.0 model Shear Mobilized, $\tau_{mobg}$ (psf) | Step 2<br>Effective Vertical Stress, $\sigma'_{vo}$ (psf) | $\tau_{mobg}/\sigma'_{vo}$ |
|---|---|---|---|---|---|
| Fill - Contractive | 1 | 0 | 3.0 | 29.5 | 0.1013 |
| Fill - Contractive | 2 | 2.67686133 | 8.9 | 79.6 | 0.1113 |
| Fill - Contractive | 3 | 5.357763059 | 14.8 | 130.4 | 0.1137 |
| Fill - Contractive | 4 | 6.065093977 | 18.7 | 164.4 | 0.1135 |
| Fill - Contractive | 5 | 8.24345661 | 20.9 | 161.4 | 0.1292 |
| Fill - Contractive | 6 | 10.35345661 | 23.5 | 180.5 | 0.1299 |
| Fill - Contractive | 7 | 12.56845661 | 27.4 | 210.5 | 0.1302 |
| Fill - Contractive | 8 | 15.46845661 | 32.6 | 250.6 | 0.1302 |
| Fill - Contractive | 9 | 17.88845661 | 37.7 | 289.5 | 0.1302 |
| Fill - Contractive | 10 | 20.48845661 | 42.8 | 328.9 | 0.1302 |
| Fill - Contractive | 11 | 22.98845661 | 48.0 | 368.3 | 0.1302 |
| Fill - Contractive | 12 | 25.48845661 | 53.1 | 407.8 | 0.1302 |
| Fill - Contractive | 13 | 27.98845661 | 58.2 | 447.2 | 0.1302 |
| Fill - Contractive | 14 | 30.48845661 | 63.4 | 486.6 | 0.1302 |
| Fill - Contractive | 15 | 32.98845661 | 68.5 | 526.1 | 0.1302 |
| Fill - Contractive | 16 | 35.48845661 | 73.6 | 565.5 | 0.1302 |
| Fill - Contractive | 17 | 37.98845661 | 78.8 | 605.0 | 0.1302 |
| Fill - Contractive | 18 | 40.71845661 | 81.8 | 628.1 | 0.1302 |
| Fill - Contractive | 19 | 43.71845661 | 82.5 | 633.4 | 0.1302 |
| Fill - Contractive | 20 | 46.71845661 | 82.4 | 633.1 | 0.1302 |
| Fill - Contractive | 21 | 49.71845661 | 82.4 | 632.8 | 0.1302 |
| Fill - Contractive | 22 | 52.20179161 | 88.6 | 680.8 | 0.1302 |
| Fill - Contractive | 23 | 55.06845661 | 101.0 | 775.7 | 0.1302 |
| Fill - Contractive | 24 | 57.63512161 | 113.4 | 871.1 | 0.1302 |
| Fill - Contractive | 25 | 60.20179161 | 125.8 | 966.5 | 0.1302 |
| Fill - Contractive | 26 | 62.76845661 | 138.1 | 1061.9 | 0.1302 |
| Fill - Contractive | 27 | 65.33512161 | 150.7 | 1157.3 | 0.1302 |
| Fill - Contractive | 28 | 67.89058771 | 154.3 | 417.5 | 0.0388 |

| Slice # | Step 3<br>Total Stress (psf) | $a_{max}$ | Mid-Slice Height (ft) | Mid-Slice Height (m) | $r_d$ | MSF | $Sh_{avg,seismic}$ (psf) |
|---|---|---|---|---|---|---|---|
| 1 | 112.5 | 0.02 | 0.50125 | 0.15 | 1.000 | 3.5 | 0.5 |
| 2 | 229.1 | 0.02 | 1.50375 | 0.46 | 0.998 | 3.5 | 0.8 |
| 3 | 335.7 | 0.02 | 2.50625 | 0.76 | 0.996 | 3.5 | 1.2 |
| 4 | 403.4 | 0.02 | 3.25375 | 0.99 | 0.995 | 3.5 | 1.5 |
| 5 | 423.5 | 0.02 | 3.525 | 1.07 | 0.994 | 3.5 | 1.6 |
| 6 | 443.2 | 0.02 | 3.875 | 1.18 | 0.993 | 3.5 | 1.6 |
| 7 | 472.8 | 0.02 | 4.381818 | 1.34 | 0.992 | 3.5 | 1.7 |
| 8 | 512.3 | 0.02 | 4.745455 | 1.45 | 0.991 | 3.5 | 1.9 |
| 9 | 552.1 | 0.02 | 5.109091 | 1.56 | 0.990 | 3.5 | 2.0 |
| 10 | 591.8 | 0.02 | 5.472727 | 1.67 | 0.989 | 3.5 | 2.2 |
| 11 | 631.4 | 0.02 | 5.836364 | 1.78 | 0.988 | 3.5 | 2.3 |
| 12 | 671.1 | 0.02 | 6.2 | 1.89 | 0.987 | 3.5 | 2.5 |
| 13 | 710.7 | 0.02 | 6.563636 | 2.00 | 0.987 | 3.5 | 2.6 |
| 14 | 750.4 | 0.02 | 6.927273 | 2.11 | 0.986 | 3.5 | 2.7 |
| 15 | 790.0 | 0.02 | 7.290909 | 2.22 | 0.985 | 3.5 | 2.9 |
| 16 | 829.7 | 0.02 | 7.654545 | 2.33 | 0.984 | 3.5 | 3.0 |
| 17 | 869.3 | 0.02 | 8.018182 | 2.44 | 0.983 | 3.5 | 3.2 |
| 18 | 893.7 | 0.02 | 8.2 | 2.50 | 0.983 | 3.5 | 3.3 |
| 19 | 898.3 | 0.02 | 8.2 | 2.50 | 0.983 | 3.5 | 3.3 |
| 20 | 898.3 | 0.02 | 8.2 | 2.50 | 0.983 | 3.5 | 3.3 |
| 21 | 898.3 | 0.02 | 8.2 | 2.50 | 0.983 | 3.5 | 3.3 |
| 22 | 998.1 | 0.02 | 8.626713 | 2.63 | 0.982 | 3.5 | 3.5 |
| 23 | 1067.8 | 0.02 | 9.480133 | 2.89 | 0.980 | 3.5 | 3.8 |
| 24 | 1137.3 | 0.02 | 10.33356 | 3.15 | 0.978 | 3.5 | 4.1 |
| 25 | 1233.2 | 0.02 | 11.18698 | 3.41 | 0.977 | 3.5 | 4.5 |
| 26 | 1328.8 | 0.02 | 12.0404 | 3.67 | 0.975 | 3.5 | 4.8 |
| 27 | 1424.3 | 0.02 | 12.89382 | 3.93 | 0.973 | 3.5 | 5.1 |
| 28 | 529.9 | 0.02 | 12.25917 | 3.74 | 0.974 | 3.5 | 1.9 |

| Slice # | STEP 4<br>Material type | Input $USR_{yield}$ eq. 7 | Step 5<br>$s_u$ (yield) (psf) | tau driving (psf) | Step 6<br>FOS | Flow Liq? | Material |
|---|---|---|---|---|---|---|---|
| 1 | Fill - Contractive | 0.17 | 4.5 | 3.0 | 1.48 | non liq | Fill - Contractive |
| 2 | Fill - Contractive | 0.18 | 14.8 | 8.9 | 1.66 | non liq | Fill - Contractive |
| 3 | Fill - Contractive | 0.19 | 24.6 | 14.8 | 1.65 | non liq | Fill - Contractive |
| 4 | Fill - Contractive | 0.19 | 30.9 | 18.7 | 1.64 | non liq | Fill - Contractive |
| 5 | Fill - Contractive | 0.21 | 34.8 | 20.9 | 1.66 | non liq | Fill - Contractive |
| 6 | Fill - Contractive | 0.21 | 39.0 | 23.5 | 1.66 | non liq | Fill - Contractive |
| 7 | Fill - Contractive | 0.22 | 45.4 | 27.4 | 1.66 | non liq | Fill - Contractive |
| 8 | Fill - Contractive | 0.22 | 54.0 | 32.6 | 1.67 | non liq | Fill - Contractive |
| 9 | Fill - Contractive | 0.22 | 62.5 | 37.7 | 1.67 | non liq | Fill - Contractive |
| 10 | Fill - Contractive | 0.22 | 71.0 | 42.8 | 1.66 | non liq | Fill - Contractive |
| 11 | Fill - Contractive | 0.22 | 79.5 | 48.0 | 1.66 | non liq | Fill - Contractive |
| 12 | Fill - Contractive | 0.22 | 88.0 | 53.1 | 1.66 | non liq | Fill - Contractive |
| 13 | Fill - Contractive | 0.22 | 96.9 | 58.2 | 1.66 | non liq | Fill - Contractive |
| 14 | Fill - Contractive | 0.22 | 105.1 | 63.4 | 1.66 | non liq | Fill - Contractive |
| 15 | Fill - Contractive | 0.22 | 113.6 | 68.5 | 1.66 | non liq | Fill - Contractive |
| 16 | Fill - Contractive | 0.22 | 122.1 | 73.6 | 1.66 | non liq | Fill - Contractive |
| 17 | Fill - Contractive | 0.22 | 130.6 | 78.8 | 1.66 | non liq | Fill - Contractive |
| 18 | Fill - Contractive | 0.22 | 135.8 | 81.8 | 1.66 | non liq | Fill - Contractive |
| 19 | Fill - Contractive | 0.22 | 136.7 | 82.5 | 1.66 | non liq | Fill - Contractive |
| 20 | Fill - Contractive | 0.22 | 136.7 | 82.4 | 1.66 | non liq | Fill - Contractive |
| 21 | Fill - Contractive | 0.22 | 136.6 | 82.4 | 1.66 | non liq | Fill - Contractive |
| 22 | Fill - Contractive | 0.22 | 146.9 | 88.6 | 1.66 | non liq | Fill - Contractive |
| 23 | Fill - Contractive | 0.22 | 167.3 | 101.0 | 1.66 | non liq | Fill - Contractive |
| 24 | Fill - Contractive | 0.22 | 188.0 | 113.4 | 1.66 | non liq | Fill - Contractive |
| 25 | Fill - Contractive | 0.22 | 208.6 | 125.8 | 1.66 | non liq | Fill - Contractive |
| 26 | Fill - Contractive | 0.22 | 229.2 | 138.1 | 1.66 | non liq | Fill - Contractive |
| 27 | Fill - Contractive | 0.22 | 249.8 | 150.7 | 1.66 | non liq | Fill - Contractive |
| 28 | Fill - Contractive | 0.19 | 234.2 | 154.3 | 1.52 | non liq | Fill - Contractive |