Foth | Envirocon

Memorandum

Joint Venture

101 International Drive, P.O. Box 16655
Missoula, MT 59808

March 22, 2017

TO: Denis Roznowski and Steve Garbaciak

FR: Mitch Vanderydt and Jim Hutchison

RE: Ashland Phase 2 Wet Dredge Gap Closure Construction Details
Ashland/NSP Lakefront Site

Introduction

This memo presents the stability analysis and proposed subgrade preparation for temporary geotextile tube barrier system (gap closures) that will be placed in the east and west gaps on each side of the Breakwater at the Ashland/NSP Lakefront Site (Site). The gap closures will be constructed in 2017 for the dredging operation and remain in place until restorative layer placement is complete in 2018. The purpose of the gap closures is to perform as a component of a multi-layered barrier system for the Site Phase 2 Wet Dredge remedial action (RA) construction work, to isolate potential impacts of the RA activities from Lake Superior. The geotextile tubes will be filled with restorative layer material and extend from the lake bottom to 2-4 feet above the anticipated water elevation (tube crest elevation 605 feet NAVD88). The location of the east and west gap closures is shown on Figure 1 in Attachment 1.

Background

The potential wave forces and general hydraulic stability of the geotextile tubes were analyzed by Baird & Associates and documented in a letter from Baird dated January 30, 2017 (Baird, 2017) (see Attachment 2). The wave transmission forces on the proposed geotextile tubes were determined using published empirical methods presented in DELOS (2003). Wave loads were estimated using Goda method for wave loads on a vertical wall (Goda, 1994), as extended by Tanimoto et al. (1976) and Takahashi et al. (1994). The theoretical wave pressures on the side of the geotextile tubes was estimated. The general geotextile tube hydraulic stability was also estimated using a range of porosities of the restorative layer material inside the tubes. The safety factor of hydraulic stability was 2.5 or greater when the porosity of the restorative layer material within the tube was estimated to be 50% or less. Actual restorative layer porosity is expected to be 30 to 40%.

Global Stability

A Global Stability Analysis was performed by Foth Infrastructure & Environment/ Envirocon Joint Venture (FE JV) for the geotextile tube barrier at the west gap closure. (Note: The west gap closure consists of three stacked layers of geotextile tubes and the east gap closure consists of two stacked layers of geotextile tubes.) The site geotechnical conditions in the west gap closure were estimated using Borings AQ-BW-05 (FE JV, 2015), RD-B-09 (FE JV, 2017), and 31N 15E by SEH dated March 12, 1996. The boring logs for these borings are provided in Attachment 3. The material parameters used in the analysis, along with the results, are provided on Figure 2 in Attachment 1. Wave loads determined in Baird's January 30, 2017 letter were applied to the geotextile tubes in the analysis. Geostudios® SLOPE/W software (2012 Edition) was used to evaluate the stability of the west gap closure. The minimum factor of safety against instability was calculated to be 1.5, which meets the minimum industry accepted design criteria of 1.5 for long-term stability. The minimum accepted design criteria for short-term stability is 1.3.

Subgrade Preparation

During anchor installation for turbidity barriers constructed for the Pilot Wet Dredge project in 2016, relatively soft subgrade conditions were encountered near the west gap area. The softer conditions appear to be located in isolated areas. In order to prepare the softer subgrade for receipt of geotextile tubes, the subgrade area below the geotextile tubes will be enhanced with a geogrid and angular stone fill. The subgrade that will support the geotextile tubes will be prepared with two construction materials:

- Miragrid© 7XT
- Angular bedding stone (2-inch nominal diameter). P_{200} content $\leq$ 0.5%.

The Miragrid will be installed prior to the stone fill. Specifications for Miragrid© 7XT are in Attachment 4. The Miragrid will be placed upon the existing subgrade where geotextile tubes will be installed and extended approximately 6.2 feet beyond the proposed outer edges of the filled tubes to facilitate spreading the load of the geotextile tubes out to a larger subgrade area.

The stone fill will consist of 2-inch nominal diameter angular stone with P_{200} content $\leq$ 0.5%. The stone will be placed on top of the Miragrid and will extend another 3.8 feet beyond the edge of the Miragrid or 10.0 feet beyond the outside edge of the proposed geotextile tube footprint. The extra aerial stone extent will provide lateral support for the stone above the geogrid and will also protect the edge of the geogrid from erosive forces. The top elevation of the stone fill will be a maximum of +/- 588.5 feet (NAVD88). The existing grade in the area of the geotextile tube footprint undulates between approximately 587 and 589 feet; therefore, the thickness of stone will vary, up to a maximum of approximately 18 inches. The geotextile tubes will be placed on the angular stone and be hydraulically filled with a restorative layer slurry. Figures 3 and 4, in Attachment 1, show the plan view and cross section of the filled geotextile tubes, respectively.

The transition of the West Peninsula temporary rock berm (north/south gap closure) to the geotextile tube (east/west gap closures) will be constructed in stages. The temporary rock berm will be placed to the northern extent of the point where the rock toe contacts the bottom geotextile tube. Next, the geotextile tubes will be placed and filled to create the gap closure. Then, the wedge-shaped void between the temporary rock berm face and the geotextile tubes western terminus face will be filled with stone and rock. Angular bedding stone will first be placed on the geotextile tubes to fill and smooth the contact area (and protect the geotextile tubes from the larger core stone rock), followed by core stone placement (see *Final Design for Ashland Breakwater* [FE JV, 2015]) for core stone specifications) to fill the void and complete the gap closure.

During removal of the geotextile tubes during west gap closure decommissioning, the geogrid underlying the bedding stone will also be removed with the barge mounted excavator, prior to final leveling of the bedding stone and clean restorative layer material contained in the geotextile tubes. The final elevation of the spread materials in the channel area of the west gap will not exceed 590 feet (NAVD88).

The east gap closure will not require the installation of the Miragrid due to the fact that the lakebed in this area already consists of a surface bedding stone layer placed during Breakwater construction. As in the west gap, prior to placement of the geotextile tubes, the basal area will be covered with 2-inch nominal diameter angular stone (approximate 6-inch layer) to prepare the subgrade. The two base geotextile tubes will be filled with restorative layer to achieve a top of tube elevation of approximately 597.5 feet (NAVD88). The final tube will then be placed above the two base tubes, with its crest elevation at approximately 605 feet (NAVD88). The wedge-shaped void between the existing rock fill (the Breakwater and the existing East Peninsula) and the geotextile tubes will be filled in the same manner as the west gap using 2-inch nominal diameter angular stone in contact with the geotextile tubes, followed by core stone rock.

The final elevation of the spread materials in the channel area of the east gap, upon gap decommissioning, will not exceed 593 feet (NAVD88).

References

Baird & Associates, 2017. *Baird Response to EPA Design Comment 3 for the 95% Design for Phase 2 Wet Dredge* letter to Steve Garbaciak, Foth Infrastructure & Environment/Envirocon Joint Venture. January 30, 2017.

Foth Infrastructure & Environment/Envirocon Joint Venture, 2015. *Final Design for Ashland Breakwater, Ashland/NSP Lakefront Site*. July 2015.

Foth Infrastructure & Environment/Envirocon Joint Venture, 2017. *Final (100%) Design for Phase 2 Wet Dredge – Ashland/NSP Lakefront Site*. March 2017.

Attachments

Attachment 1

Figures

NOTES:

1. HORIZONTAL DATUM: NAD83 WISCONSIN PLANE, NORTH ZONE, U.S. FEET.
2. VERTICAL DATUM: NAVD88 USING GEOID 12A.
3. UPLAND SURVEY FROM FE JV DATED SEPTEMBER 2015. BATHYMETRIC SURVEY PERFORMED BY J.F. BRENNAN DATED MAY 2, JULY 19, 23, AUGUST 10 AND SEPTEMBER, 2016.
4. APPROXIMATE WEST PENINSULA BULKHEAD LINE LOCATION DETERMINED FROM WILHELM ENGINEERING COMPANY, INC. 3-6-1986 DRAWING TITLED, ASHLAND MARINA BULKHEAD LINE. UNABLE TO FIELD LOCATE LEGAL DESCRIPTION POINT OF BEGINNING OR POINT OF ENDING. METES AND BOUNDS DESCRIPTION USED TO CREATE BULKHEAD LINE SHAPE AND A "BEST FIT" LOCATION FOR THE BULKHEAD LINE WAS USED BASED ON EXISTING SITE FEATURES.

LEGEND

- PHASE 2 PROJECT AREA
- PHASE 2 WET DREDGE AREA
- APPROXIMATE SHORELINE (602 FT)
- 88-2 — GEOTECHNICAL BORING NUMBER AND LOCATION (NET, 1989)
- AQ-BW-01 — GEOTECHNICAL BORINGS NUMBER AND LOCATION (GEOTECHNICAL INVESTIGATION\ REPORT ASHLAND BREAKWATER)
- SB-179 / SB-173 — GEOTECHNICAL BORING NUMBER AND LOCATION (GEOTECHNICAL INVESTIGATION REPORT, FE JV 2013)
- 2900N–1500E / AQ-SB-07 — GEOTECHNICAL BORING NUMBER AND LOCATION (OFFSHORE SAMPLING REPORT, ANCHOR QEA 2013)
- SD-125A — GEOTECHNICAL BORING NUMBER AND LOCATION (DATA GAPS INVESTIGATION REPORT, BURNS AND MCDONNELL, 2011)
- EP-B-2 — GEOTECHNICAL EAST BORING PENINSULA NUMBER AND LOCATION (TWIN PORTS TESTING, 2015)
- 2500N–2200E — CHEMISTRY SAMPLE NUMBER AND LOCATION REMEDIAL INVESTIGATION REPORT APPENDIX J – HISTORICAL DATA (URS, 2007)
- NSP-SE-C5 — CHEMISTRY SAMPLE NUMBER AND LOCATION REMEDIAL INVESTIGATION REPORT APPENDIX J – SUPPLEMENTAL DATA (URS, 2007)
- SD-126A — CHEMISTRY SAMPLE NUMBER AND LOCATION DATA GAPS INVESTIGATION (BURNS & MCDONNEL, 2011)
- AQ-SD-01 — CHEMISTRY SAMPLE NUMBER AND LOCATION DESIGN PACKAGE FOR WET DREDGE PILOT STUDY (ANCHOR QEA, 2014)
- B1B — CHEMISTRY SAMPLE NUMBER AND LOCATION EXTENDED PILOT AREA B1B SEDIMENT CORE (FOTH INFRASTRUCTURE & ENVIRONMENT/ ENVIROCON JOINT VENTURE [FE JV, 2015])
- RD-B-06 — GEOTECH BORING NUMBER AND LOCATION (FE JV, 2016)
- PDI-30 — SUPPLEMENTAL GEOTECHNICAL BORING LOCATION (FE JV, 2016)
- PDI-29 — CHEMICAL BORING NUMBER AND LOCATION (FE JV, 2016)
- PDI-39 — CHEMISTRY SAMPLE NUMBER AND LOCATION (FE JV, 2016)
- INDICATOR KRIGING BOUNDARY
- 600 — EXISTING LAKE BED CONTOUR
- 590 — NULL AREA TARGET DREDGE ELEVATION (EXISTING BATHYMETRY MINUS 1 FOOT)
- 590 — TARGET DREDGE ELEVATION
- DREDGE PRISM DAYLIGHT LIMIT
- PROPOSED GEOTUBE GAP CLOSURE
- BREAKWATER/GAP BARRIER CURTAIN SYSTEM
- ISOLATION CURTAIN
- APPROXIMATE LOCATION TEMPORARY PARTIAL HEIGHT CURTAIN
- EXISTING PILES
- EXTENT OF PILOT STUDY DREDGING
- APPROXIMATE DMU BOUNDARY/ DAYLIGHT LIMIT
- BULKHEAD WALL LOCATION
- APPROXIMATE WEST PENINSULA BULKHEAD LINE LOCATION
- WATER TREATMENT DISCHARGE PIPE (LTWTS)
- APPROXIMATE EXTENT OF EXISTING PENINSULA FILL
- NULL AREA
- SEDIMENT INVENTORY DREDGE AREA
- SEDIMENT RESIDUAL DREDGE AREA

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Foth Infrastructure & Environment, LLC
2121 Innovation Court, Suite 300
P.O. Box 5126
De Pere, WI 54115-5126
Phone: 920-497-2500 Fax: 920-497-8516

ASHLAND/NSP LAKEFRONT SITE FINAL (100%) REMEDIAL DESIGN FOR PHASE 2 WET DREDGE WEST GAP CLOSURE

NORTHERN STATES POWER COMPANY

ASHLAND COUNTY WISCONSIN

REVISIONS			
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RECORD DRAWINGS OF COMPLETED CONSTRUCTION CONFORMING TO CONTRACTOR AND/OR OWNERS RECORDS.
BY DATE

DATE OF PREPARATION	BY	DATE
SURVEYED		
DRAWN	JOW	MARCH 2017
DESIGNED		
CHECKED	DMR	MARCH 2017

OVERALL DREDGE PLAN

HORIZONTAL SCALE: 0 60' 120'

PROJECT ID: 16X002

1

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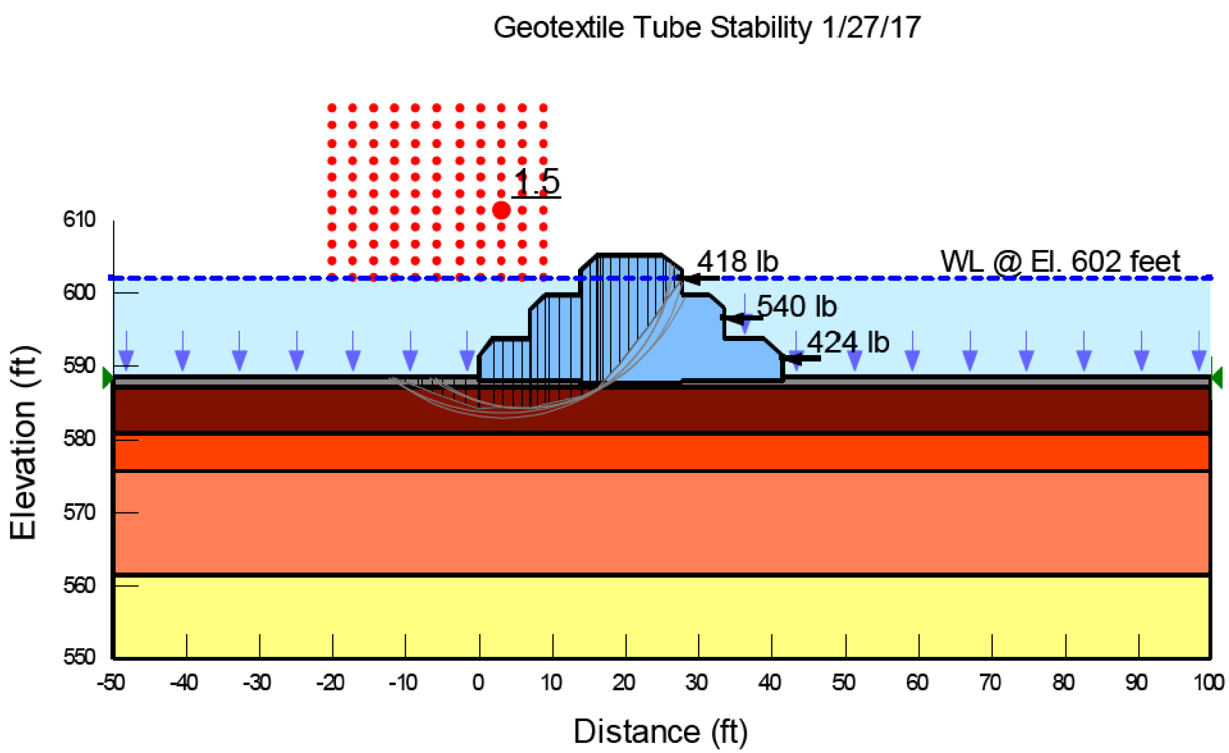

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Name: Gravel Model: Mohr-Coulomb Unit Weight: 117 pcf Cohesion': 0 psf Phi': 35 ° Piezometric Line: 1
Name: SP-SM Model: Mohr-Coulomb Unit Weight: 102 pcf Cohesion': 0 psf Phi': 30 ° Piezometric Line: 1
Name: Upper CL Model: Mohr-Coulomb Unit Weight: 102 pcf Cohesion': 1,200 psf Phi': 0 ° Piezometric Line: 1
Name: Lower CL Model: Mohr-Coulomb Unit Weight: 107 pcf Cohesion': 500 psf Phi': 0 ° Piezometric Line: 1
Name: ML Model: Mohr-Coulomb Unit Weight: 105 pcf Cohesion': 35 psf Phi': 35 ° Piezometric Line: 1

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NOT TO SCALE

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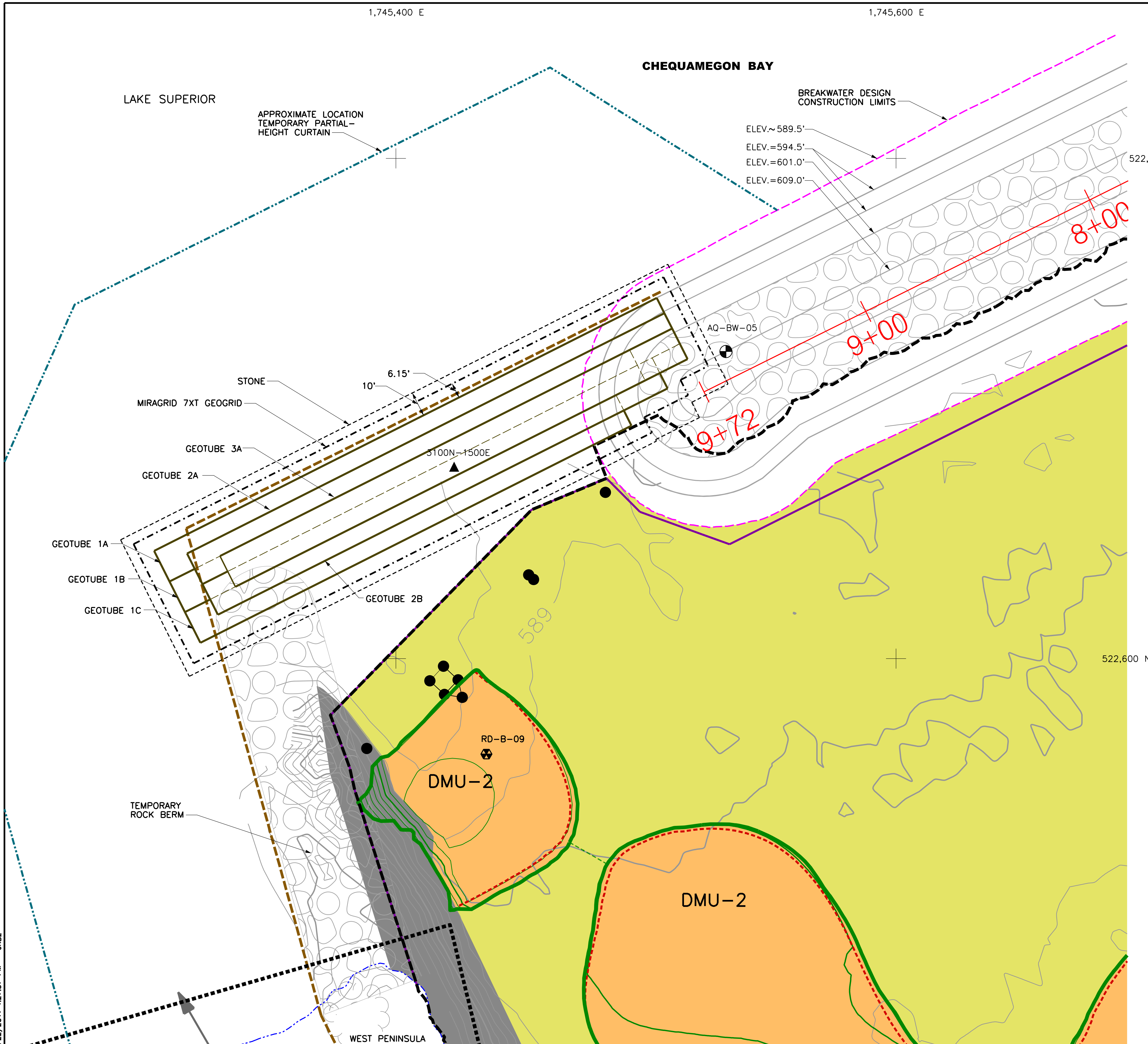
FIGURE 2

WEST GAP CLOSURE STABILITY RESULTS

Date: MARCH 2017 | Revision Date:

Drawn By: JOW | Checked By: JOS1 | Project: 16X002



LEGEND

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- APPROXIMATE GEOTUBE LOCATION
- APPROXIMATE MIRAGRID 7XT GEOGRID LOCATION
- APPROXIMATE STONE LIMITS
- AQ-BW-05 — GEOTECHNICAL BORINGS NUMBER AND LOCATION (GEOTECHNICAL INVESTIGATION\ REPORT ASHLAND BREAKWATER)
- 3100N-1500E — CHEMISTRY SAMPLE NUMBER AND LOCATION REMEDIAL INVESTIGATION REPORT APPENDIX J – HISTORICAL DATA (URS, 2007)
- RD-B-09 — GEOTECH BORING NUMBER AND LOCATION (FE JV, 2016)

NOTES:

1. HORIZONTAL DATUM: NAD83 WISCONSIN PLANE, NORTH ZONE, U.S. FEET.
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ASHLAND/NSP LAKEFRONT SITE
FINAL (100%) REMEDIAL DESIGN FOR PHASE 2 WET DREDGE
WEST GAP CLOSURE

NORTHERN STATES POWER COMPANY

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BY ____ DATE ____

DATE OF PREPARATION	BY	DATE
SURVEYED		
DRAWN	JOW	MARCH 2017
DESIGNED		
CHECKED	DMR	MARCH 2017

WEST GAP CLOSURE

HORIZONTAL SCALE: 0 20' 40'

PROJECT ID: 16X002

3

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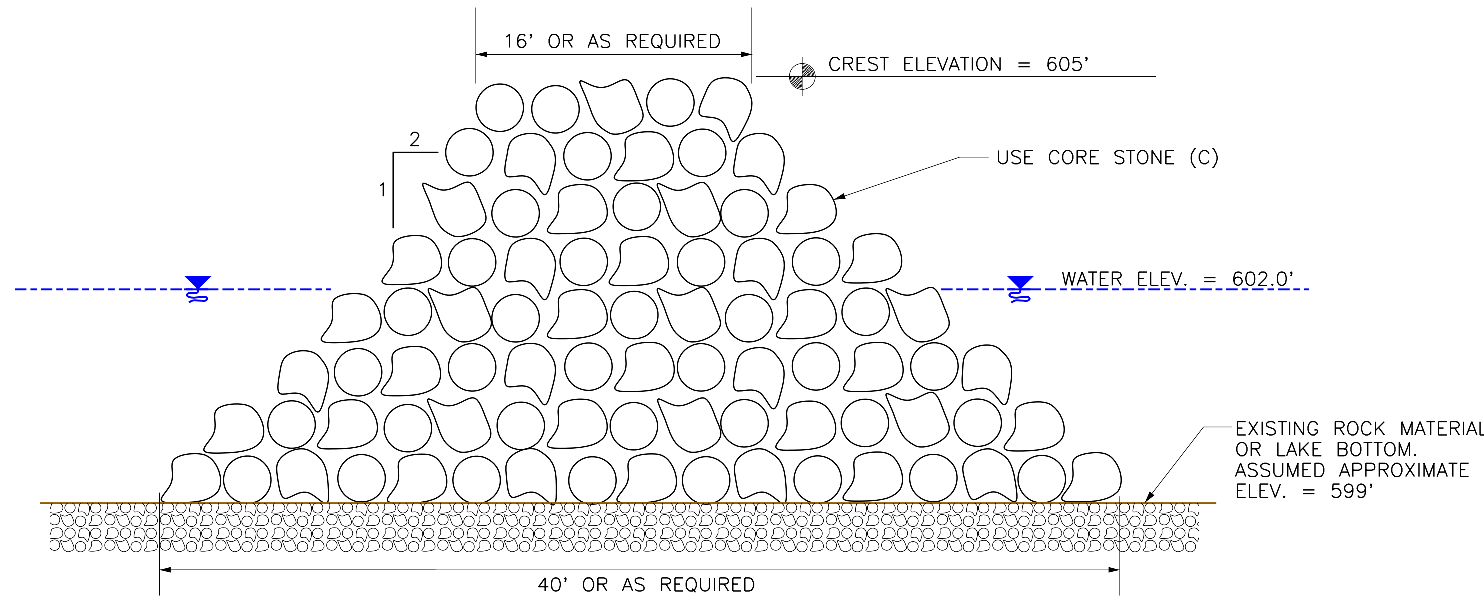

TEMPORARY ROCK BERM DETAIL

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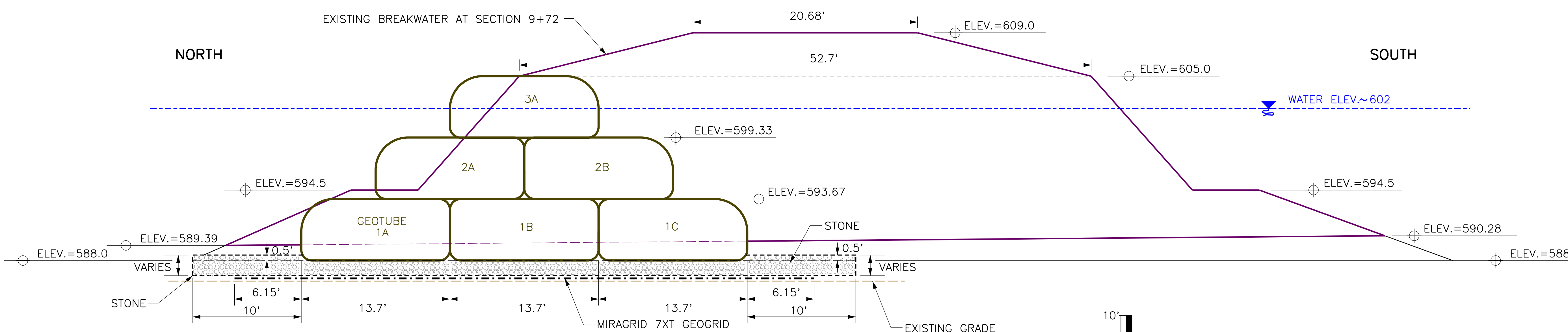
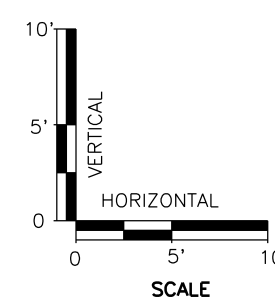


WEST GAP CLOSURE DETAIL

SCALE: AS SHOWN



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ASHLAND/NSP LAKEFRONT SITE
FINAL (100%) REMEDIAL DESIGN FOR
PHASE 2 WET DREDGE
WEST GAP CLOSURE

NORTHERN STATES POWER COMPANY

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REVISIONS			
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DESIGNED		
CHECKED	DMR	MARCH 2017

WEST GAP CLOSURE DETAILS

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PROJECT ID: 16X002

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Attachment 2

Baird & Associates Letter dated January 30, 2017

January 30, 2017

Baird & Associates
2924 Marketplace Drive, Suite 200
Madison, Wisconsin 53719 USA
T. 608 273 0592
F. 608 273 2010

Steve Garbaciak, PE
Foth Infrastructure & Environment, LLC
Glen Hill North Office Park
800 Roosevelt Road, Building E, Suite 412
Glen Ellyn, IL 60137

Baird

oceans
engineering
lakes
design
rivers
science
watersheds
construction

Re: Ashland/NSP Lakefront Site
95% Design for Phase 2 Wet Dredge
Baird Response to EPA Design Comment 3

Dear Mr. Garbaciak:

Further to our recent discussions, attached please find our formal technical response to Design Comment 3 in the EPA's letter of January 17, 2017 regarding the 95% design report for the Phase 2 West Dredge works. Specifically, the attached response provides our input regarding wave-structure interaction with the proposed geotube gap closure structures, including wave overtopping/transmission, wave loads and hydraulic stability.

We appreciate the opportunity to continue to support you on the execution of this important project.

Should you require further information or clarification on the attached information, please do not hesitate to contact the undersigned.

Sincerely,
Baird & Associates

Matthew J. Clark, PE
Senior Project Manager

Enc: As Stated

12361.102

Introduction

Further to Foth's request of 19Jan17, this document presents Baird's input regarding the EPA's comments on the proposed geotextile tube (subsequently referred to as "geotube") gap closure structures (Design Comment 3 in EPA letter of 17Jan17). More specifically, this document presents Baird's initial assessment of wave overtopping, wave transmission and wave loads on the geotubes.

The information presented herein is based on the site and metocean conditions defined in earlier studies related to the design of the breakwater (as reported in Foth|Envirocon JV, 2015) and a limited review of readily available published information regarding the design and performance of geotube structures. More detailed analyses could be undertaken if necessary; however, this was not possible within the limited time frame provided to prepare a response to the EPA's comments.

Water Levels

The following information summarizes water level data of relevance to the assessment of the geotube gap closure structures:

- Peak lake level in 2016 = +1.7 ft LWD
- Current WL (21Jan17) = 602.0 ft IGLD85 = +0.9 ft LWD
- 2017 forecast WLs expected to fall between long term average and record high (refer to Figure 1)

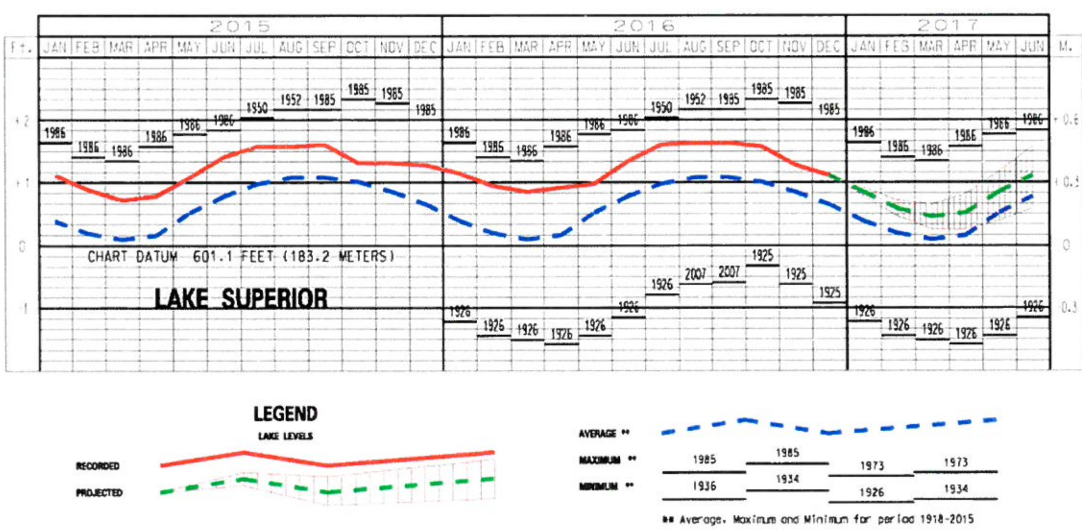


Figure 1 – Lake Superior – Six Month Lake Level Forecast
(Source: USACE, Detroit District, January 2017)

Based on the information presented in Figure 1, and considering the factors that cause long-term and seasonal fluctuations in lake levels, it is anticipated that lake levels may range from 0 to +2 ft LWD over the anticipated 18 month deployment period of the geotube structures (April 2017 through October 2018). Lower lake levels (0 to + 1 ft LWD) are generally expected during the winter months (Nov-Apr), while higher lake levels are generally expected during the summer months (May-Oct).

The lake levels presented above do not include storm surge due to individual storm events. Based on previous analyses by Baird, storm surge at the project site may be in the order of 1 to 3 ft during severe storms. The extreme water levels previously estimated by Baird to support the design of the breakwater are presented in Table 1, including lake level, storm surge and combined water level.

Table 1 - Extreme Water Levels

(Source: Foth|Envirocon JV, 2015)

Return Period (yrs)	Lake Level (ft LWD)	Storm Surge (ft)	Combined WL (ft LWD)
5	+1.61	-	-
10	+1.80	1.0 to 2.1*	-
20	+1.94		-
25	+2.03	-	-
50	+2.16	-	-
100	+2.26	1.6 to 3.2*	-
200	+2.36	-	+2.9 to +4.0**
Extreme WL used for breakwater design = 10 yr lake level + 100 yr Storm surge			+5.0

**Storm surge depends on wind speed and direction; range above considers N to NE winds*

***200 yr combined water level is based on 10/20 and 20/10 combinations of lake level and storm surge*

<u>Wave Conditions</u>

The following information, which was developed by Baird to supported detailed design development for the breakwater (as reported in Foth|Envirocon JV, 2015), summarizes wave data of relevance to the assessment of the geotube gap closure structures:

- Hs = 1.0 ft ~ 22% exceedance (average ~ 37 hrs/week)
- Hs = 2.0 ft ~ 4% exceedance (average ~ 7 hrs/week)
- Hs = 3.0 ft ~ 10 year event
- Hs = 4.0 ft ~ 100 year event

Structure Configurations

Cross-sections for the existing breakwater (designed by Baird) and the proposed geotube gap closure structures (designed by Foth|Envirocon JV) are summarized in Table 2 and illustrated in Figure 2.

Table 2 – Summary of Structure Configurations

Parameter	Breakwater	West Gap Closure	East Gap Closure
Design Life	Permanent	Temporary (18 mths)	Temporary (18 mths)
Structure Type	Rubble Mound	Geotubes	Geotubes
Structure Configuration (Layering/Composition)	A/B/C Stone	1/2/3 Stack (34 ft circumference)	1/2 Stack (45 ft circumference)
Outer Surface	Rough + permeable	Smooth + impermeable	Smooth + impermeable
Lakebed Elevation (ft LWD)	-12.0	-13.0	-11.0
Crest Elevation (ft LWD)	+8.0*	+4.0	+4.0
Crest Width (ft)	14.0	~ 8	~ 11
Side Slopes (H:V)	1.75:1	~ 1:1	~ 1:1

**Breakwater crest elevation includes 1 ft allowance for settlement*

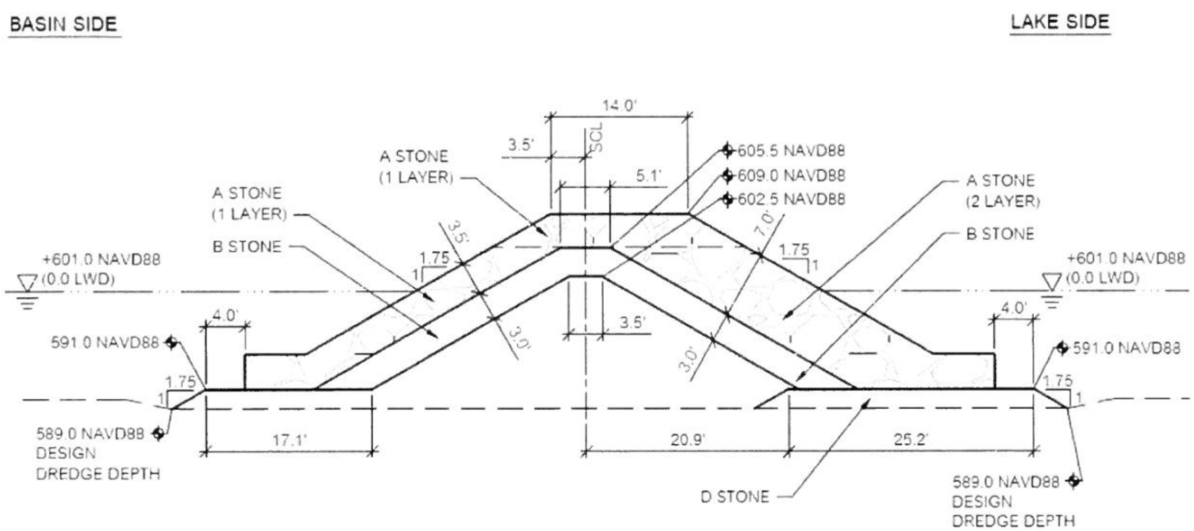
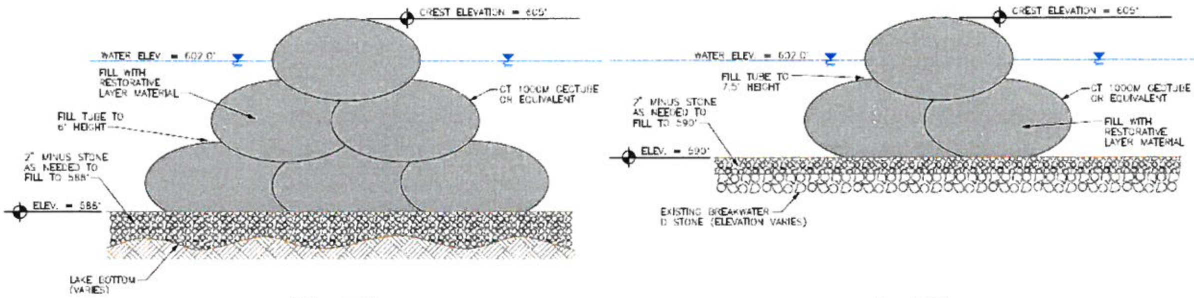


Figure 2a – Existing Breakwater Cross-Section
(Source: Foth|Envirocon JV, 2015)



West Gap **East Gap**

Figure 2b – Proposed Gap Closure Cross-Sections
(Source: Foth|Envirocon JV, 2016b)

It is understood that the proposed geotube gap closure structures will remain in place throughout the duration of the Phase 2 Wet Dredge program, as summarized below:

- April to October 2017 – marine dredging (seasonal high water levels)
- November 2017 to March 2018 – winter shutdown (seasonal low water levels)
- April to October 2018 – restorative layer placement (seasonal high water levels)

Wave Overtopping and Transmission

Wave overtopping occurs when the wave runup elevation exceeds the crest elevation of the structure. This process results in the transfer of a volume of water, as well as wave energy, over the structure and into the basin. Wave overtopping is quantified by the mean wave overtopping rate (Qot), while wave transmission is quantified by the wave transmission coefficient (Kt). Wave overtopping is not considered to be a specific concern to the Phase II wet dredge program. However, wave transmission may be, as wave action in the basin could result in damage to the dual barrier curtain system and/or otherwise compromise the dredging operation.

The following bullet points highlight some key considerations related to wave transmission of the proposed geotube gap closure structures:

- Wave overtopping and transmission are dependent upon relative freeboard, which is defined as the structure freeboard, F, divided by the incident wave height, Hs (Frel = F/Hs)
- Wave overtopping and transmission are greater for smooth (and impermeable) structures than for rough (and permeable) structures due to reduced energy dissipation
- Wave overtopping and transmission for the geotube structures (stepped outer surface, and impermeable) is expected to fall between that for smooth (and impermeable) and rough (and permeable) structures
- The proposed crest elevation of the geotube structures is 605 ft NAVD88 (+ 4.0 ft LWD), which is approximately 2 to 4 ft higher than the range in lake levels anticipated over the next 18 months
- The crest elevation of the breakwater is 609 ft NAVD88 (+8.0 ft LWD), which is approximately 6 to 8 ft higher than the range in lake levels anticipated over the next 18 months

Preliminary Estimate of Wave Transmission
A preliminary estimate of the wave transmission performance of the existing breakwater and proposed geotube gap closure structures has been estimated using published empirical methods presented in DELOS (2003). Table 3 presents estimates of wave transmission for the existing breakwater and the proposed west and east gap closure structures for a range of lake levels and storm events that may occur over the 18-month duration of the Phase 2 wet dredge project.

Table 3 – Preliminary Estimates of Wave Transmission for Various WL and Wave Conditions

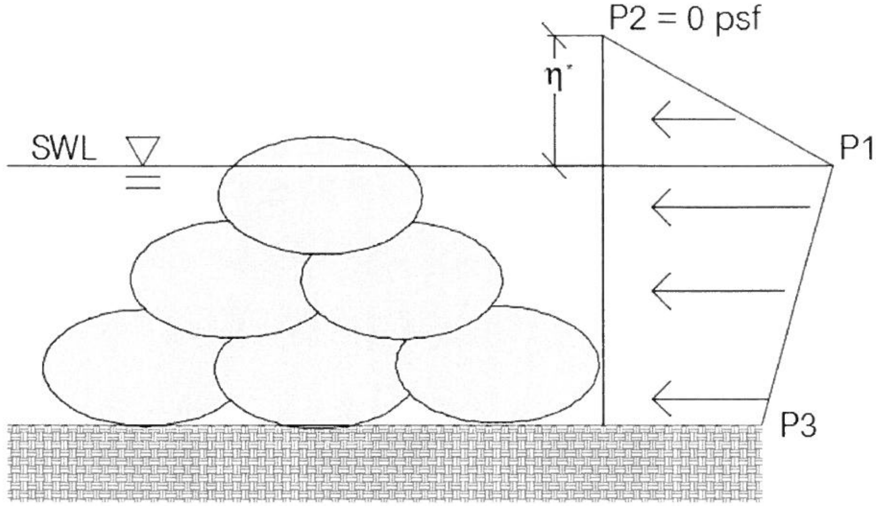
Description	WL (ft CD)	Incident Wave (Hs, ft)	Transmitted Wave (Hs, ft)		
			West Gap	Breakwater	East Gap
1. 2016 Summer High Lake Level + 22% Exc. Wave	+1.6	1.0	< 0.1	~ 0	< 0.1
2. 2016 Summer High Lake Level + 4% Exc. Wave	+1.6	2.0	0.15	~ 0	0.15
3. Case 1 with 0.4 ft Increase in Lake Level	+2.0	1.0	< 0.1	~ 0	< 0.1
4. Case 2 with 0.4 ft Increase in Lake Level	+2.0	2.0	0.3	< 0.1	0.2
5. Case 1 with 0.75 ft Increase in Lake Level	+2.35	1.0	< 0.1	~ 0	< 0.1
6. Case 2 with 0.75 ft Increase in Lake Level	+2.35	2.0	0.4	0.15	0.3
7. Current Lake Level + 10 Yr Surge&Wave	+3.0	3.0	1.2	0.2	1.1
8. 2016 Summer High Lake Level + 10 Yr Surge+Wave	+3.7	3.0	1.4	0.25	1.3
9. 2016 Summer High Lake Level + 100 Yr Surge+Wave	+4.8	4.0	2.5	0.3	2.3

This information indicates that wave transmission over the proposed gap closure structures will be significantly greater than that through/over the breakwater. This result is not surprising given the fact that the proposed gap structures are significantly lower and narrower than the existing breakwater structure. The following information should also be considered regarding the results in Table 4:

- The length of the west and east gap structures (~ 200 and 100 ft respectively) is considerably less than that of the breakwater (~ 840 ft); hence, the results for the breakwater are more indicative of average wave conditions in the basin;
- Transmitted wave heights (Hs) in excess of 0.5 ft will only occur during moderate to severe storm conditions at moderate to high lake levels, and only locally behind the gap closure structures;
- Although larger waves will exist directly behind the gap closure structures during storms, these conditions will decrease rapidly due to diffraction (lateral spreading of wave energy) as one moves away from the structures.

Wave Loads and Stability of Geocontainers

Preliminary Estimate of Wave Loads
A preliminary estimate of the wave loads on the geotubes has been developed using the Goda method for wave loads on a vertical wall (Goda 1974), as extended by Tanimoto et al (1976) and Takahashi et al (1994). For purpose of this analysis, the stacked geotube structure is treated as a stepped structure with a series of offset vertical faces. Figure 3 provides a schematic diagram of the theoretical wave pressure distribution on the structure.



P1 = maximum wave-induced pressure at the still water line (SWL)
P2 = 0 (the wave-induced pressure reduces to zero at the limit of wave runup in the structure, η*)
P3 = wave induced pressure at the lakebed

Figure 3 – Theoretical Wave Pressure Profile

The design wave conditions at the west and east gaps were estimated by Baird using numerical model simulations, as reported in Foth|Envirocon JV (2016a). Table 4 presents the estimated wave pressures (P1 and P3) for the west and east gap closure structures for the 10 and 100-year design wave conditions (H_{max}) for a representative range in water levels (note that the wave loads will be highest when the water level matches the crest elevation of the structure). The horizontal wave forces on the overall structure, and on the individual geotubes, can be estimated from these results to assess the global stability of the structure and the stability of individual geotubes.

Table 4 – Preliminary Estimates of Wave Pressures on Gap Closure Structures

Description		SWL (ft LWD)	Hmax (ft)	Angle of Incidence of Wave (degrees)	η* (feet)	p_1 (psf)	p_3 (psf)
West Gap	10 Year Wave	+1.0	3.1	45	4.0	106	61
		+4.0	3.1	45	4.0	99	49
	100 Year Wave	+1.0	5.0	45	6.5	174	101
		+4.0	5.0	45	6.5	164	80
East Gap	10 Year Wave	+1.0	3.7	0	5.6	151	92
		+4.0	3.7	0	5.6	142	74
	100 Year Wave	+1.0	5.4	0	8.1	242	144
		+4.0	5.4	0	8.1	205	104

Preliminary Estimate of Geotube Structure Stability Against Wave Loads

A literature review was undertaken to identify methods to estimate the stability of the proposed getotube gap closure structures. Several relevant references were identified, including Pilarczyk (1996, 2000), Oumeraci et al (2003) and Bezuijen et al (2005). At this time, due to the limited time available, only the methods of Pilarczyk (1996, 2000) and Bezuijen et al (2005) have been considered.

In all cases, physical model tests were used to assess the stability of the structures under wave attack, with the test results quantified using a stability number, Ns = $\frac{H_S}{\Delta T}$, where:

- H_S is the significant wave height
- T is thickness (height) of the geotubes
- $\Delta = (1-n)\frac{\rho_g-\rho_w}{\rho_w}$
- n is porosity (%) of the fill used in the geotubes
- ρ_g is the density of the grains of the fill
- ρ_w is the density of water

Pilarczyk (1996, 2000) refers to earlier tests by Delft Hydraulics (1973, 1994) of well-filled geotubes with thicknesses (T) in the order of 0.5 to 0.75 times their theoretical diameter (D) and base widths (B) in the order of 1.1D (i.e. thickness to width ratio, T/B ~ 0.45 to 0.68). He concludes that these structures were stable for Ns < 1. Bezuijen et al (2005) tested larger and much flatter geocontainers (T/B ~ 0.16), and found that these structures were stable for Ns < 2. Based on information presented in Foth|Envirocon JV (2016b), the thickness to width ratio of the geotubes proposed for the gap closure structures is in the order of 0.4 to 0.43; hence, the results of Pilarczyk (1996, 2000) appear to better suited to the Ashland case than the results of Bezuijen et al (2005).

The Pilarczyk (1996, 2000) method was used complete a preliminary assessment of the stability of the proposed gap closure structures under the 10 and 100-year return period wave events. The calculations were completed assuming geotube thicknesses (T) of 6.0 ft and 7.5 ft respectively for the west and east gaps. As noted in Foth|Envirocon JV (2016b), it is understood that the geotubes will be filled with imported clean sand that will be subsequently used as "Restorative Layer Material". The final density of the placed geotubes will be dependent on the porosity (n) of the sand fill. The stability calculations were repeated for porosities ranging from 20% to 50%. The results of these calculations are presented in Table 5, and show increasing stability (lower Ns) with decreasing porosity.

Table 5 – Preliminary Stability Calculations for Proposed Gap Closure Structures

	Estimated Stability Number, Ns = $H_S/\Delta T$			
Porosity	**West Gap**		**East Gap**	
(n)	10-year	100-year	10-year	100-year
50%	0.40	0.65	0.36	0.55
40%	0.34	0.54	0.30	0.46
30%	0.29	0.46	0.25	0.39
20%	0.25	0.40	0.22	0.34

The estimated $H_S/\Delta T$ values are all less than 1; hence, this methodology suggests that the proposed geotube gap closure structures should be stable under wave conditions up to (and exceeding) the 100-year design wave conditions. It is noted that Baird has no prior experience with this methodology, and has not evaluated the reliability of the method against practical experience. Hence, it is recommended that Foth|Envirocon JV undertake an independent assessment of the stability of the geotube structures using the estimated wave loads presented earlier.

Discussion

It is understood that the information presented above will be used by Foth|Envirocon JV to respond to the EPA's questions regarding wave overtopping, wave transmission and wave loads on the proposed geotube gap closure structures.

The information presented herein is based on the site and metocean conditions defined in earlier studies related to the design of the breakwater (as reported in Foth|Envirocon JV, 2015) and a limited review of readily available published information regarding the design and performance of geotube structures. More detailed analyses could be undertaken if necessary; however, this was not possible within the limited time frame provided to prepare a response to the EPA's comments.

Other potential issues related to the geotube gap closure structures that may warrant consideration include the following:

- Geotube stability against ice loads;
- Risk of damage (puncture) during placement over bedding material, by debris or by ice;
- Geotechnical considerations (bearing capacity, settlement, scour).

References

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Bezuijen, A., de Groot, M.B., Klein Breteler, M., and E. Berendsen, E. 2004. Placing Accuracy and Stability of Geocontainers. GeoDelft, WL Delft Hydraulics, Dutch Ministry of Transport and Public Works.

Foth Envirocon Joint Venture. July 2015 (Revised). Final Design for Ashland Breakwater, Ashland/NSP Lakefront Site, Project I.D. 15X001, Eau Claire, Wisconsin, USA.

Foth Envirocon Joint Venture. February 2016 (Revised) April 2016. Final Design for Phase 2 Wet Dredge Pilot Study, Ashland/NSP Lakefront Site, Project I.D. 15X002, Eau Claire, Wisconsin, USA.

Foth Envirocon Joint Venture. December 2016. 95% Design for Phase 2 Wet Dredge, Ashland/NSP Lakefront Site, Project I.D. 16X002, Eau Claire, Wisconsin, USA.

Attachment 3

Boring Logs

AQ-BW-05 Boring Log
RD-B-09 Boring Log
31N 15E Boring Log

SOIL BORING LOG INFORMATION

Foth | Envirocon Joint Venture

Facility/Project Name: Xcel Ashland	License/Permit/Monitoring No.		Boring Number: AQ-BW-05
Boring Drilled By: Name of crew chief (first, last) and Firm First: Scott Last: Strigel Firm:	Date Drilling Started: 2 / 24 / 15 (M M / D D / Year)	Date Drilling Completed: 2 / 24 / 15 (M M / D D / Year)	Drilling Method: HSA
WI Unique Well No. / DNR Well ID No. / Well Name	Final Static Water Level: Feet MSL	Surface Elevation: 601.7 Feet MSL	Borehole Diameter: 8" (3" Shelby T.)
Local Grid Origin (estimated: X) or Boring Location)	Lat ° ' "	Local Grid Location	
State Plane 522722.81 N 1745531.99 E	Long ° ' "	N E	
1/4 of 1/4 of Section 34 , T 48 N, R 4 W		Ft S W	
Facility ID / County: Ashland / County Code: 2	Civil Town/City/or Village: Ashland		

Sample: Number and Type	Length Att. & Recovered (in.)	Blow Counts	Depth in Feet (Below ground surface)	Soil/Rock Description and Geologic Origin For Each major Unit	USCS	GRAPHIC LOG	Well Diagram	PID/FID	Shear Strength Pocket Pen (tsf)	Moisture Content	Liquid Limit	Plasticity Index	P 200	RQD/Comments
			1											
				(30" Ice)										
			2											
			3											
			4											
			5											
				Water										
			6											
			7											
			8											

Sample									Soil Properties					
Number and Type	Length Att. & Recovered (in.)	Blow Counts	Depth in Feet (Below ground surface)	Soil/Rock Description and Geologic Origin For Each major Unit	USCS	GRAPHIC LOG	Well Diagram	PID/FID	Shear Strength Pocket Pen (tsf)	Moisture Content	Liquid Limit	Plasticity Index	P 200	RQD/Comments
			9											
			10											
1	8/24		11											
			11.8'											11.8'
			12	3" Wood Chips										
														12.1'
			13	Loose, reddish brown, silty sand, non-plastic	SM					W				
			14											
	14.3													14.3'
	Shelby Tube		15							20.6%			38.3	ST-1 γ = 106.5 pcf K = 2.8x10^{-4} cm/sec
	15.8		16											
		4	17							20.8%			14.6	
		5		Loose, reddish brown, fine sand, non-plastic with trace small cobble at 19.5' to 21.3'.						22.1%			16.1	G_s = 2.668
2	16/24	5	18		SP-SM									
		3												
			19											
		2								20.0%			5.4	
3	24/24	4	20											
		5												
		2	21											
			22	Medium soft to soft reddish brown lean clay, cohesive.	CL				0.05					

Sample: Number and Type	Sample: Length Att. & Recovered (in.)	Blow Counts	Depth in Feet (Below ground surface)	Soil/Rock Description and Geologic Origin For Each major Unit	USCS	GRAPHIC LOG	Well Diagram	PID/FID	Soil Properties: Shear Strength Pocket Pen (tsf)	Soil Properties: Moisture Content	Soil Properties: Liquid Limit	Soil Properties: Plasticity Index	Soil Properties: P 200	RQD/Comments
														22.3'
		3								20.7%	32	19		γ = 108.2 pcf
4	16/24		23											
		3							0.60	W				
		4							0.6	W				
			24											
	24.3													
										22.2%				ST-2
	Shelby Tube		25											γ = 104.7 pcf
														e_o = 0.634
														Cc = 0.24
														Cr = 0.03
			26											
	26.3								0.25					
			27											
			28											
			29	Medium stiff to soft reddish brown lean clay, medium plasticity cohesive with fat clay (CH) layer at 24 to 25'.	CL									
		1								23.4%				
5		2	30							19.5%	27	15		G_s = 2.730; γ = 114.8 pcf
		2							0.25	W				
		2	31						0.25	W				
			32											
			33											
			34											
		3								22.7%	26	15		
6	6/24	3	35											
		4							0.1	W				
		5	36											

Sample									Soil Properties					
Number and Type	Length Att. & Recovered (in.)	Blow Counts	Depth in Feet (Below ground surface)	Soil/Rock Description and Geologic Origin For Each major Unit	USCS	GRAPHIC LOG	Well Diagram	PID/FID	Shear Strength Pocket Pen (tsf)	Moisture Content	Liquid Limit	Plasticity Index	P 200	RQD/Comments
			37											
			38	Medium stiff to soft reddish brown lean clay, medium plasticity cohesive with fat clay (CH) layer at 24 to 25'.	CL									
			39											
		12								20.6%				
7	12/24	14	40											
			40.3											
		16		Dense reddish brown silt, with some fine sand, non-plastic.	ML					W				
		20	41											
			41.3	EOB @ 41.3'										
			42											
			43											
			44											
			45											
			46											
			47											
			48											
			49											
			50											

* Lab

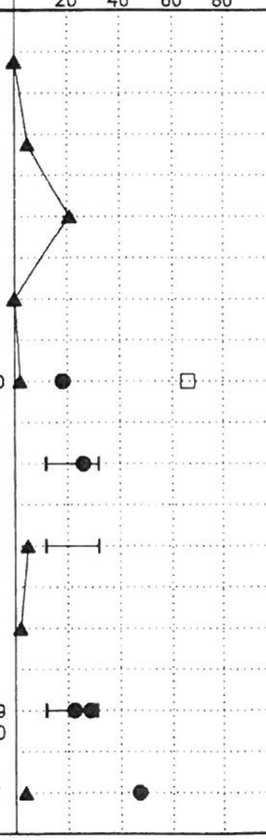
Foth | Envirocon
Joint Venture

BORING NUMBER RD-B-09

CLIENT Ashland NSP Lakefront
PROJECT NAME PDI, Phase II RA
PROJECT NUMBER 16X002
PROJECT LOCATION Ashland, WI
DATE STARTED 8/24/16 COMPLETED 8/24/16
GROUND ELEVATION 590.73 ft msl HOLE SIZE 3.25 inches
DRILLING CONTRACTOR Coleman Engineering
GROUND WATER LEVELS:
DRILLING METHOD Driven Casing
AT TIME OF DRILLING —
LOGGED BY RJM7 CHECKED BY JBH
AT END OF DRILLING —
NOTES Backfilled with 3/8" chipped bentonite. Est. 1 1/4 bags.
AFTER DRILLING —

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
0		Black Wood Debris, Wet.	SS 1	10	0-0-0-0 (0)		
			SS 2	39	0-0-5 (5)		
5			SS 3	33	7-17-4-2 (21)		
			SS	0	0-0-0-0 (0)		
		Very Loose Brown Fine Sand, Wet. (SP) Soft Brown Lean Clay with Sand, Wet. (CL)	SS 4	38	0-1-1-2 (2)		100
10		@10'-11 S.G. = 2.67, CU at Given Strain of 15%, Effective angle = 31.5 degrees, c' = 0.04 tsf @11'-12' S.G. = 2.72	SH 5	100		0.6 1	96
		Very Loose Brown Poorly Graded Sand with Silt, Wet. (SP-SM) Medium to Stiff Brown Lean Clay, Wet. (CL)	SS 6	100	2-2-3-3 (5)	1	
15			SS 7		0-0-2-3 (2)		
		@16'-17' S.G. = 2.699 @17'-18' S.G. = 2.687	SH 8	75		1	109 100
			SS 9	100	1-2-2-3 (4)		77
20		Note: All Pocket Pen Values are Compressive Strength.					

▲ SPT N VALUE ▲ 20 40 60 80
PL MC LL 20 40 60 80
☐ FINES CONTENT (%) ☐ 20 40 60 80



Bottom of borehole at 20.0 feet.

Note: No field olfactory or visual observation of environmental impact within boring, including staining, sheen, odor or visible NAPL, unless otherwise noted on log.

State of Wisconsin
Department of Natural Resources

Route To:
☐ Solid Waste
☐ Emergency Response
☐ Wastewater
☐ Haz. Waste
☐ Underground Tanks
☐ Water Resources
☐ Other

Soil Boring Log Information
Form 4400-122 7-91

Facility/Project Name	License/Permit/Monitoring Number	Boring Number
Ashland Lakefront Property		**31N 15E**

Boring Drilled By (Firm name and name of crew chief)	Date Drilling Started	Date Drilling Completed	Drilling Method
Enviroscan	**3/12/96**	**3/12/96**	**Geoprobe**

DNR Facility Well No.	WI Unique Well No.	Common Well Name	Final Static Water Level	Surface Elevation	Borehole Diameter
			Feet MSL	**588.9** Feet	**2.0** Inches

Boring Location		Local Grid Location (If applicable)
State Plane N, E	Lat **46° 35' 49"**	☒ N ☐ S **3100** Feet
1/4 of 1/4 of Section T N,R	Long **90° 52' 59"**	☒ E ☐ W **1500** Feet

County	DNR County Code	Civil Town/City/ or Village
Ashland	**02**	**Ashland**

Sample Number	Length (in) Recovered	Blow Counts	Depth In Feet	Soil/Rock Description And Geologic Origin For Each Major Unit	U S C S	Graphic Log	Well Diagram	PID/FID	Standard Penetration	Moisture Content	Liquid Limit	Plastic Limit	P 200	RQD/ Comments
1			1.5, 3.0	Brown, fine grained SAND; trace to little Silt, trace wood waste, no discernable odor	SP SM			ND/--						
2			4.5					ND/--						
3			6.0, 7.5					ND/--						
4			9.0	Brown, stiff, lean CLAY; little Sand and Gravel	CL			ND/--						
				End of Boring @ 10.0'										

I hereby certify that the information on this form is true and correct to the best of my knowledge.

Signature: John E. Guld

Firm: SEH
SEH 421 Frenette Drive
Chippewa Falls, WI. 54729
Tel: 715-720-6200, Fax: 715-720-6300

Attachment 4

Miragrid© 7XT Specification Sheet

TENCATE Mirafi®

TENCATE GEOSYNTHETICS
Americas

Miragrid® 7XT

SOIL REINFORCEMENT

Miragrid® 7XT geogrid is composed of high molecular weight, high tenacity polyester multifilament yarns which are woven in tension and finished with a PVC coating. Miragrid® 7XT geogrid is inert to biological degradation and resistant to naturally encountered chemicals, alkalis, and acids.

Miragrid® 7XT geogrid is used as soil reinforcement in MSE structures such as; segmental retaining walls, precast modular block walls, wire faced walls, geosynthetic wrapped faced walls and steepened slopes. Miragrid® 7XT is also used in MSE stabilized platforms for voids bridging, embankments on soft soils, landfill veneer stability, reducing differential settlement and for foundation seismic stability.

TenCate Geosynthetics Americas is accredited by Geosynthetic Accreditation Institute – Laboratory Accreditation Program (GAI-LAP).

Mechanical Properties	Test Method	Unit	Machine Direction Value
Tensile Strength @ Ultimate (MARV[1])	ASTM D6637 (Method B)	lbs/ft (kN/m)	5900 (86.1)
Tensile Strength @ 5% strain (MARV[1])	ASTM D6637 (Method B)	lbs/ft (kN/m)	2160 (31.5)
Creep Rupture Strength[2]	ASTM D5262/D6992	lbs/ft (kN/m)	4069 (59.4)
Long Term Design Strength[3]		lbs/ft (kN/m)	3370 (49.2)

[1] Minimum Average Roll Values (MARV) shown above are based on QC Testing per a defined lot not to exceed 12 months. Testing Frequency follows ASTM D4354, Table 1.
[2] 75-year design life based on NTPEP Report REGEO-2011-01-001 and REGEO-2015-01-002.
[3] Long Term Design Strength for Type 3 Backfill (Silty Sand), 6-inch lift / 25,000-lb roller. RF_{CR} = 1.45; RF_{ID} = 1.05; RF_D = 1.15 (Installation damage reduction factor for other soils available upon request.

Physical Properties	Unit	Roll Characteristic
Mass/Unit Area (ASTM D5261)	oz/yd² (g/m²)	9.4 (346)
Roll Dimensions[4] (width x length)	ft (m)	6 x 150 (1.8 x 46) 12 x 200 (3.6 x 61) 12 X 1000 (3.6 x 305)
Roll Area	yd² (m²)	100 (84) 267 (220) 1333 (1114)
Estimated Roll Weight	lbs (kg)	65 (29) 179 (81) 846 (383)

[4] Special order roll lengths are available upon request.

Miragrid 7XT and Tensile Strength direction are continuously printed in white on the edge of the roll.

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GMA Geosynthetic Materials Association