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LATHAM & WATKINS LLP

December 31, 2014

Craig Melodia
Associate Regional Counsel
United States Environmental Protection Agency
Region 5
77 West Jackson Boulevard
Chicago, IL 60604-3590

Re: Ashland Lakefront Superfund Site:
Response to September 23, 2014 Weston Dry Dredge Report

Dear Mr. Melodia:

On behalf of Northern States Power Company of Wisconsin ("NSPW" or the "Company"), we are writing to provide a further response to Weston Solutions, Inc.'s ("Weston") Technical Submittal, dated September 23, 2014 ("2014 Weston Report"). In the company's prior submittal, dated October 28, 2014, NSPW submitted a preliminary analysis of the 2014 Weston Report, which identified several serious concerns with Weston's unproven and untested approach. In that submittal, NSPW indicated that it intended to further review, in detail, the 2014 Weston Report (including the supplemental CD provided on October 8, 2014, which contained over 400 pages of exhibits), and that it would submit a supplemental report upon completion of its review. NSPW and its consultants have since reviewed these materials in greater detail, and this submittal constitutes our supplemental response. As described further below, Anchor QEA and Dr. Richard J. Finno, have both independently reviewed the 2014 Weston Report in detail, and both conclude that there are flaws with Weston's unproven and aggressive approach, that the dry dredge is not safe or implementable at this Site, and that there are other better alternative remedies that would safely and effectively remediate the sediments at this Site. We also summarize below the history of our discussions over this issue as we think it provides helpful context for NSPW's determination that the dry dredge remedy is too dangerous to implement at this Site.

I. EXPERIENCED TECHNICAL EXPERTS PREVIOUSLY CONCLUDED THAT THE DRY DREDGE IS NOT SAFE OR IMPLEMENTABLE AT THIS SITE

A. Numerous Technical Concerns With The Dry Dredge Proposal Were Raised in 2008-2010

The possibility of implementing a dry dredge remedy at the Site has been the subject of numerous reports and analyses on the part of the U.S. Environmental Protection Agency

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("EPA"), NSPW, and others. In 2008, a feasibility study assessed dry excavation as a possible remedial approach, and concluded that a dry dredge would present "potentially greater risk to human health, because of the need to work behind barriers engineered to keep out the waters of Lake Superior."[1] Foth Infrastructure and Engineering, LLC ("Foth"), reached similar conclusions in its 2009 evaluation of a potential dry excavation, noting that elevated artesian pressures within the Copper Falls formation could result in instability, basal heave, and failure of the dry excavation system.

Notwithstanding the feasibility study and Foth's conclusions, the Record of Decision ("ROD") for the Site issued in September 2010 selected a wet-dry hybrid remedy for the sediments, while allowing for the potential of a remedy change (via an explanation of significant differences ("ESD")) to a wet dredge only approach, following the successful completion of a pilot study.[2] When the ROD was issued, NSPW was made aware of a report for the first time prepared by EPA's consultant, Weston Solutions, Inc. ("Weston"), titled "Conceptual Geotechnical Assessment For Sediment Removal at the Ashland/Northern States Power Lakefront Site in Ashland, Wisconsin" ("2009 Weston Report"). It appears that the remedy selection in the ROD was based, at least in some material part, on the analysis in the 2009 Weston Report.

Although Weston's "preliminary and conceptual" analysis concluded that "near-shore, bay bottom sediments likely can be safely removed using dry excavation techniques, *assuming that conceptual planning, final design engineering and implementation of the construction work are all properly executed*," it provided no specific guidance for safe implementation. Further, Weston acknowledged that the "structural stability of the sheet pile wall, excavation bottom blowout, and piping of bay bottom sandy sediments are *significant worker/equipment safety concerns and represent potential '**fatal flaw**' failure mechanisms*" unique to the near shore dry excavation remedy. 2009 Weston Report, at 2 (emphasis added). Weston further recommended that, in order to even attempt a dry dredge, the Site should be divided up into a matrix of interior sheet pile walls, which was not reflected in the remedy described in the ROD. Those modifications to the dry dredge remedy were not evaluated under the National Contingency Plan ("NCP") criteria in the ROD.

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[1] Note that other remedies have also been identified as possible alternative remedies. For example, as early as the late 1990s, a consultant for the Wisconsin Department of Natural Resources ("WDNR") recommended a CDF remedy for the Site. *See* Short Elliot and Hendrickson (SEH), Sediment Investigation Report (1996). An engineered shoreline remedy identified in the feasibility study also scored well on the National Contingency Plan criteria, except for the mistaken belief at that time that there was not a way to permit a CDF under Wisconsin law. EPA, WDNR, and NSPW now understand that a CDF can be permitted under Wisconsin law and, in fact, there is a CDF within the Site already that was created and permitted in the late 1980s.

[2] Based on previous discussions with EPA and the text of the ROD itself, NSPW understands that, if the wet dredge pilot is successful, EPA will issue an Explanation of Significant Differences authorizing the Company to perform a full-scale wet dredge at the Site

B. In 2012 Multiple Nationally-Recognized Sediment Engineering Firms Concluded That The Dry Dredge Was Not Safe Or Implementable

In 2012, multiple technical experts with particular expertise in sediments—including Anchor, Gradient, and Burns & McDonnell—separately reviewed the 2009 Weston report and expressed serious concerns about the safety, environmental risks, and feasibility of a dry dredge.[3] In fact, each consultant concluded that a dry dredge is an inappropriate remedy for the Site and could result in catastrophic and irreparable harm to human health and the environment. *See* Anchor QEA, *Independent Evaluation of Sediment Removal Alternatives: Ashland/NSPW Lakefront Superfund Site* (October 2012); Gradient, *Critique of the National Contingency Plan Consistency of US EPA's September 2010 Record of Decision for the Ashland/Northern States Power Lakefront Site* (October 2012); and Burns & McDonnell, *Technical Assessment of EPA's Comparative Analysis of Near Shore Dry Excavation and Site-Specific Failure Mechanisms* (October 2012).

Among other things, the experts concluded that a dry dredge creates a significant risk of "bottom uplift," a catastrophic failure of the bay floor that would threaten the safety of the workers performing the remedy and cause wide distribution of the contaminants in the bay sediments. The dry dredge is also based on unrealistic expectations regarding the ability of a sheet pile wall to hold back Lake Superior, among other potential failure mechanisms. A dry dredge would also cause significant community disruption and potentially expose the community to greater impacts from noise, air emissions, odors, and the long-term closure of Kreher Park. These experts concluded that there are less expensive, less dangerous, and more effective alternatives to the selected dry dredge. As such, they concluded the dry dredge is inconsistent with the NCP and would be an unsafe and inappropriate remedy for this Site.

C. After Collecting Additional Site Data In 2012 And 2013, Anchor Further Concluded That The Additional Data Demonstrated That The Dry Dredge Was Not Safe Or Implementable

As a result of the concerns identified by NSPW's consultants in 2012, EPA requested that NSPW perform additional sampling along the shoreline in 2012, and additional sampling in the Bay in 2013. NSPW's consultants gathered this additional data in 2012 and 2013. This additional data was then evaluated by NSPW's consultants. The new data further confirmed prior concerns about the safety and implementability of the dry dredge. *See* Anchor QEA, *Shoreline and Offshore Geotechnical Evaluation Report* (December 2013). In particular, the data showed significant variability in site conditions that did not follow predictable trends, and therefore would not permit a contractor to "design around" problem areas. *Id.*, at ES-2.

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[3] Three additional consultants, AECOM, URS Corporation and Foth Infrastructure & Environment LLC, also expressed serious concerns with the dry dredge, prior to the release of the Weston Report.

D. In 2014, Distinguished Academic Expert, Dr. Finno, Independently Concluded That The Dry Dredge Is Not Safe Or Implementable At The Site

NSPW sought a second opinion of the conclusions reached by Anchor in Anchor's 2013 report. In 2014 NSPW asked Dr. Richard J. Finno[4], a distinguished Professor of Civil Engineering at Northwestern University, specializing in geotechnical engineering to review the data for the Site and Weston and Anchor's reports and to provide his input. Dr. Finno's independent evaluation of the proposed dry dredge remedy, submitted on October 28, 2014, concluded that the dry dredge is not safe or implementable at the Site due to the potential for bottom heave, global instability, and numerous design and constructability concerns. Dr. Finno further concluded that a wet dredge remedy would eliminate the risks associated with the dry dredge, and would be a far better solution to the geotechnical challenges at the Site. Dr. Finno also performed a preliminary review of Weston's 2014 report, as did Anchor, and both identified serious concerns with Weston's unproven, novel and aggressive approach.

II. ANCHOR AND DR. FINNO HAVE IDENTIFIED SERIOUS CONCERNS UPON A DETAILED REVIEW OF WESTON'S 2014 REPORT AND BOTH CONCLUDE THAT THE DRY DREDGE IS UNSAFE

Based on the experts' detailed reviews of the 2014 Weston Report, NSPW remains concerned that the dry dredge is still unsafe. As set forth below, Anchor and Dr. Finno's supplemental reviews of the 2014 Weston Report and backup analyses, confirm that: (i) Weston has employed novel and aggressive technical approaches that overestimate the safety and implementability of a dry dredge; (ii) Weston has proposed factors of safety below industry standards; and (iii) a wet dredge or engineered shoreline would be more appropriate for this Site.

A. Anchor's Supplemental Evaluation Concludes That The 2014 Weston Report Does Not Resolve The Safety And Implementability Concerns Previously Identified

Anchor's supplemental evaluation has identified a number of significant concerns related to Weston's analytic methods, Weston's failure to appreciate implementability and construction

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[4] As described in our October 28, 2014 submittal, Dr. Finno has conducted substantial research in many areas directly applicable to the dry dredge remedy proposed for the Site, including research related to excavation support, tunnels, failure processes, soils, and ground movements. Dr. Finno's work has been widely recognized in the civil engineering community. He has received eight major awards from the American Society of Civil Engineers, including the Karl Terzaghi Award, which is considered to be the most prestigious award for a geotechnical engineer in the United States, and he has been awarded numerous National Science Foundation grants. Dr. Finno served as a member of the EPA's Land Application Peer Review Committee. Dr. Finno has also served as Chair of the Earth Retaining Structures Committee of the American Society of Civil Engineers, as well as an editor of its Journal of Geotechnical and Geoenvironmental Engineering.

safety concerns, and the incomplete nature of Weston's key data and information. Anchor's supplemental evaluation is attached hereto as Attachment 1.

In particular, Anchor's evaluation concludes that Weston has relied on aggressive, non-conservative assumptions and newly developed and untested formulas to reach the conclusion that a dry dredge remedy would be implementable at the Site. Even under Weston's unproven and aggressive approach, Weston itself acknowledges that there would still be a potential for instability to occur. They downplay this risk, however, by suggesting that it could be mitigated by advancing unrealistic field practices during construction. Among other things, Anchor has identified the following concerns with the 2014 Weston Report:

- Weston has developed new, experimental formulas for calculating stability that have not been vetted or tested in the geotechnical industry and lack a track record of being applied and implemented successfully. Because the consequences of a failure at the Site are severe, it would be inappropriate to experiment with novel and untested theories here;

- Weston also makes several aggressive and non-conservative assumptions throughout their analysis. The risks of these assumptions compound, one upon another, each eroding the overall margin of safety further. If success hinges upon many assumptions all turning out favorably, the risk of failure increases if even one of those assumptions is incorrect. For this reason Weston's overall approach is overly aggressive and unsafe;

- Since 2009, Weston's formulas and calculations continue to change, resulting in inconsistencies in Weston's analysis. Weston has seemingly modified its approach to address new data that weighs against dry-dredging, rather than analyzing the new data using standard engineering methods or even their own prior approaches to drive conclusions. For example, in the 2014 Weston Report, Weston recommends a minimum factor of safety of 2.0 for the "piping" failure mechanism, which is both inconsistent with standard engineering practice, and also much more aggressive than their own work in 2009, in which they proposed a minimum factor of safety of 4 to 5 for the piping evaluation;

- Weston has selected the lowest possible (i.e., the least conservative) factors of safety for their analyses, despite clear guidance that higher values should be selected in complex projects with the potential for catastrophic disaster, as is the case here. In fact, a number of elements in Weston's analysis are actually *less conservative* than the design assumptions that are being made for the wet dredge pilot study containment system, which are based on real world observed conditions at the site from last Fall. For example, Weston has underestimated the size and force of waves that could impact the Site, which is particularly surprising given the experiences this past fall when significant waves were experienced during the 2014 wet dredge pilot program. As a result, Weston's proposed wall is thinner and simpler than would actually be required to withstand a storm event and possible wave forces;

- Given the high degree of heterogeneity of the Site's lithology and soil conditions, the presence of a substantial artesian condition underlying the Site, the large scale of the excavation, and the potential consequences of failure, Anchor and Dr. Finno have recommended a factor of safety of at least 1.5 for the evaluation of bottom uplift. Weston supports a factor of safety of only 1.25. Notably, Weston calculated a factor of safety of 1.4 for at least one known location, indicating that even under their approach, if they applied the common industry standard factor of safety of 1.5, there would be remedy failure in at least some known areas at the site;

- Weston has not fully accounted for the potential for drained soil conditions to develop in the excavation area, which results in overestimated stability levels;

- Weston's approach adds new elements to the dry dredge that were not contemplated in the 2010 ROD, nor presented during the public comment period on the Proposed Plan, such that an ESD or ROD Amendment is now needed to even allow the dry dredge remedy to move forward. For example, Weston recommends (i) the use of numerous small dry dredge cells, which would increase the length of sheet pile to be driven many times over with resulting adverse community impacts, including noise, cost and schedule time; (ii) the installation of a network of roads to support the segmented design approach, and (iii) the installation of groundwater wells to dewater the aquifer, in the event that thin portions of the aquitard are detected during dredging. None of these proposed modifications have been evaluated under the NCP criteria, despite posing significant impacts to the community and substantial impacts on cost.

- Weston does not dispute the significant negative community impacts that would result to the community from the dry dredge, given the duration of the project, and the impacts from noise and odors; and

- Weston's proposed dry dredge remedy is not cost effective (even if it were implementable). A failed dry dredge could impose significant economic hardship on gas utility customers and community residents who might ultimately bear the costs of the cleanup.

Based on their review of the 2014 Weston Report and independent calculations, Anchor recommends that a dry dredge ***not*** be performed at the Ashland Site because a dry dredge is too risky and other, better alternative remedies exist that are safer and more appropriate for this Site.

B. Dr. Finno's Supplemental Evaluation Has Also Identified Flaws In Weston's Analysis

Like Anchor, Dr. Finno has also prepared a supplemental report analyzing the proposed dry dredge remedy in greater detail, attached hereto as Attachment 2, and has similarly concluded that the proposed dry dredge is not appropriate for this Site. After reviewing the data and analyses presented in the 2014 Weston Report, Dr. Finno has independently concluded that:

- The proposed dry dredge remedy is not safe or implementable at the Site due to the potential for bottom uplift, global instability, and numerous and insurmountable design and constructability concerns;

- A dry dredge excavation would not be stable because bottom uplift due to artesian water pressures will likely occur at some locations after soil has been excavated. When bottom uplift occurs, the excavation area likely will be flooded by ground water from the underlying aquifer;

- The factors of safety used by Weston do not appropriately account for variability in subsurface conditions, engineering parameters, and loading conditions, and even Weston's proposed factors of safety would not be met at certain locations at the Site;

- The effects of the upward flow of water adjacent to the sheet pile wall for the actual subsurface conditions encountered offshore need to be considered. When these effects are considered, Dr. Finno's analysis shows that there are several locations where the factor of safety against piping is about one-half of the industry standard of 4 to 5. If piping were to occur, support provided by the soil adjacent to the sheet pile wall would be removed and the wall would collapse, flooding the excavation;

- A failure mode that encompasses the entire sheet pile wall needs to be considered but was not considered by Weston. If the analysis is performed, the results indicate that failure (i.e., a mass of soil encompassing the wall sill slides into the excavation area, subsequently flooding it), would occur in the long term condition;

- The concept of using a sheet pile wall for the dry dredge is ill-founded. Weston's analyses do not adequately account for expected loading conditions, including wave loadings, development of water-filled gaps during periods of high water, or the directional effects of wave loading;

- Weston should have provided details regarding how the movements of the sheet pile wall were computed in order to determine whether the sheet pile wall would be overtopped during a storm, resulting in the flooding of the excavation;

- Weston dismisses the potential for leaking through the sheet pile wall, stating only that the leakage would be reduced by using a cell-by-cell approach; and

- Weston ignores a number of significant construction-related difficulties associated with the dry dredge remedy such as maintaining an impervious barrier and structural integrity and installing sheeting to required depth in hard portions of glacial tills.

In light of these concerns, Dr. Finno likewise concludes that a wet dredge remedy would eliminate the risks associated with the dry dredge, and would be a far better solution to the geotechnical challenges at the Site.

III. THE REMEDY SELECTION PROCESS SHOULD BE FAIR, OPEN, AND TRANSPARENT

NSPW continues to believe that all parties would benefit from an open technical dialogue regarding the formulas and methodologies utilized by all consultants. As described in our October 28, 2014, submittal, NSPW, EPA and WDNR initially agreed that the parties would

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exchange this technical information (formulas, assumptions, minimum safety factors, etc.) *before* commencing the offshore sampling program in 2013, and then proceed to have an open, scientifically-focused dialogue among the parties' technical teams regarding the safety and implementability of the dry dredge once the offshore sampling was completed. NSPW provided its consultants formulas and methodologies to EPA and WDNR on October 15, 2012 (as well as an updated analysis on December 17, 2013, after the offshore sampling was completed). EPA did not share its own consultant's formulas or methodologies with NSPW until the fall of 2014, long after the data was collected and NSPW's own consultant reports were submitted. This was not the open and collaborative approach the parties contemplated.[5]

IV. NSPW REMAINS WILLING TO PERFORM A REASONABLE ALTERNATIVE REMEDY

When safe for its workers and the community, NSPW has not hesitated to perform extensive site investigation and remediation work, all at substantial costs. The company has been successfully implementing the onland (Phase 1) remediation, at a cost estimated to exceed $50 million. The company has invested significant funds into further studying conditions in the bay and pursuing alternative approaches to the bay sediment cleanup. The company stands ready to implement a sediment remedy that is safe and meets the NCP criteria. To this end, the Company has indicated its willingness to implement a reasonable full-scale wet dredge remedy, or a hybrid wet-dredge/engineered shoreline remedy, or, to the extent the agencies want to perform what we view as an unsafe, unimplementable dry dredge remedy, that the agencies accept a cashout from NSPW.

Thank you for your attention to these important matters. We look forward to discussing these issues with you further at your convenience.

Sincerely,

Kelly E. Richardson
of LATHAM & WATKINS LLP

[5] In an effort to obtain the technical information underlying that portion of the Weston analysis that has been communicated to the Army Corps, NSPW also submitted numerous Freedom of Information Act ("FOIA") requests to the Army Corps, but has yet to receive a complete response. In fact, *more than one year after it was issued*, one such request is still pending on appeal before the Army Corps' Engineer Research and Development Center in Vicksburg, Mississippi.

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cc: Kristen Carney
Tom Benson, U.S. DOJ
Sumona Majumdar, U.S. DOJ
Lacey Cochart, WDNR
Scott Hansen, EPA
John Robinson, WDNR
Jamie Dunn, WDNR

Enclosures

POINTS OF CONCERN REGARDING WESTON 2014 DRY DREDGING EVALUATION

ASHLAND LAKEFRONT SUPERFUND SITE

Prepared for

Northern States Power - Wisconsin

Prepared by

Anchor QEA, LLC

December 2014

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LIST OF ACRONYMS AND ABBREVIATIONS

ASCE	American Society of Civil Engineers
CDF	Confined Disposal Facility
CFF	Copper Falls Formation
EPA	U.S. Environmental Protection Agency
FOIA	Freedom of Information Act
Foth	Foth Infrastructure and Engineering, LLC
FS	Feasibility Study
MCF	Miller Creek Formation
NBC	Negative Bearing Capacity
NRRB	National Remedy Review Board
NSPW	Northern States Power Company, a Wisconsin corporation
ROD	Record of Decision
Site	Ashland/Northern States Power Lakefront Superfund Site
USACE	U.S. Army Corps of Engineers
Weston	Weston Solutions, Inc.

EXECUTIVE SUMMARY

This report discusses several key points of concern regarding an evaluation that was recently commissioned by the U.S. Environmental Protection Agency (EPA) for dry dredging, which they have proposed as a possible means of remediating contaminated sediment at the Ashland/Northern States Power Lakefront Superfund Site (Site) in Ashland, Wisconsin. Weston's evaluation, and the subject of this report, is presented in the *Ashland Lakefront Superfund Site Technical Submittal*, by Weston Solutions, Inc. (Weston), dated September 2014, and referred to hereafter as the "2014 Weston Report" (Weston 2014).

Weston has developed new calculation formulas for basal heave and bottom upheaval. They attempt to defend dry dredging as an appropriate remedial approach for this Site, but their conclusions improperly rely on multiple aggressive assumptions and newly developed formulas that have not been tested in actual practice. Their report describes, defends, and ultimately relies on aggressive assumptions and decisions each step of the way, each of which degrades safety and increases risk, and in total, leads to a dangerous underestimation of the problems that dry dredging would encounter at this Site.

Even given all of these aggressive steps, Weston acknowledges that there would still be a potential for instability to occur, stating in their report:

> *It must be noted that unanticipated subsurface conditions may be encountered during the dry excavation field work which would have the effect of reducing one or more of the calculated FS (factor of safety) values for the five instability scenarios to unacceptable values.*

They then attempt to sidestep this conclusion by advocating unrealistic field practices to try to control or mitigate the instabilities during construction.

The Ashland Lakefront project is no place for an aggressive and uncertain remedial approach. The combination of debris, unpredictable weather and lake conditions, and highly variable soil and groundwater properties demands a thoughtful and careful approach. The Site, its chemical and geotechnical characteristics, and the environment and climate of

Chequamegon Bay itself pose variable conditions that significantly affect the technical design and construction approach.

Figure 1 presents a representation of how Weston's analysis combined a number of aggressive choices and assumptions, resulting in a conclusion that pushes an experimental, unproven, and risky approach to the remediation of this Site. The figure presents six separate rows, each representing a choice or assumption used in evaluating dry dredging. For each subject of analysis, Weston has made aggressive assumptions, and in some cases the most aggressive assumption possible. The subjects of analysis listed in Figure 1 and discussed in detail later in this report, are as follows:

- **Aquitard thickness.** In cases where there is some question about the relevant thickness of the stabilizing Site aquitard layer, Weston has selected the thickest possible assumption. If the average, or thinner end, of the observed thickness is used in the calculations, actual risks of failure are greater than Weston's method.
- **Waves and wave forces.** Weston has selected relatively low wave loads—lower, in fact, than those experienced at the Site in the fall of 2014—and has also derived relatively low loading forces from the waves acting on the surrounding sheet pile walls. For example, Weston assumes the following: (1) pressure from waves breaking against the sheet pile wall will be less than the pressure from non-breaking waves; and (2) significant waves will never break against the sheet pile wall.
- **Formulas used for analysis.** Weston has developed new formulas for their stability analyses, rather than relying on tested, peer reviewed, and published formulas already present in standard engineering literature.
- **Artesian pressures.** In cases where there is some question about the magnitude of the artesian groundwater pressure applied to the basal heave analysis, Weston has applied reduced artesian pressures (lower than measured) acting below the surface. If observed artesian pressures are used in the calculations, actual risks are greater than Weston's hypothetical scenario.
- **Drainage conditions in soil.** Weston has not fully accounted for the potential that drained soil conditions may arise during the dry dredging process.
- **Factors of Safety.** In cases where different Factors of Safety are cited in engineering literature, Weston has targeted the lowest, or least protective, possible values for their analyses. Given the severe consequences of failure (potential loss of life and

irreparable environmental impacts), varying site conditions, and complexity of the project, conservative, accepted Factors of Safety are warranted.

As depicted on Figure 1, the end result of Weston's evaluation, when each step is compounded, is an aggressive and risky approach, rather than an appropriately cautious and safe approach, that recognizes the very real and severe consequences of failure at this Site.

Even given all of these aggressive steps, Weston still admits that there would be a potential for instability to occur. However, they attempt to downplay the significance of this risk, and describe highly unrealistic ways of "controlling" or "mitigating" possible instabilities during construction. Among other things, Weston has proposed modifying the remedy to be performed in a segmented approach with numerous sheet pile cells, with correspondingly significant schedule impacts, costs, and complexity. Weston has also proposed utilizing untested and challenging dewatering operations in an attempt to decrease the artesian aquifer pressure, with further unsafe assumptions about the homogeneous nature of the aquifer, which could potentially impact the extent of groundwater contamination in the uplands areas.

Weston's analysis presents an unconvincing and dangerous argument. Anchor QEA continues to recommend that dry dredging not be performed at this Site because it is too risky—there are far too many dangers associated with variability in the subsurface, the surrounding conditions of Chequamegon Bay and Lake Superior pose numerous challenges to construction and safety, and the results of failure are too potentially dangerous and environmentally catastrophic. Furthermore, other, better alternative remedies exist that are safer and more appropriate, including wet dredging (and development of an engineered shoreline (where sediments are stabilized and could potentially be treated and used for Site shoreline improvements).

1 INTRODUCTION

This report discusses several key concerns regarding an evaluation recently commissioned by the U.S. Environmental Protection Agency (EPA) for dry dredging at the Ashland/Northern States Power Lakefront Superfund Site (Site) in Ashland, Wisconsin. EPA's evaluation was undertaken by Weston Solutions, Inc. (Weston), dated September 2014, and referred to hereafter as the "2014 Weston Report" (Weston 2014).

1.1 Background and Nature of Concerns

Dry dredging is a construction technique that in a lakefront or offshore setting such as this, involves installing a wall or cutoff structure capable of retaining the surrounding water to create an enclosed area from which water can be pumped out, leaving the previously submerged subgrade exposed and available for excavation using land-based, earth moving equipment and methods.

At this Site, dry dredging poses concerns for implementability, safety, and environmental protectiveness, most notably due to the presence of artesian (elevated) groundwater pressures below the Site. Dry dredging also has logistical challenges that make it more complicated, which will result in more negative impacts to the community than would result from alternative cleanup approaches. Dry dredging is also a very expensive endeavor that is not a cost-effective solution for cleanup of this Site.

1.2 Review of Previous Evaluations of Dry Dredging

Dry dredging as a possible remedial measure at the Site has been the subject of numerous reports and analyses in the recent past. It is helpful to review the history of these efforts, both on the part of EPA, Northern States Power Company, a Wisconsin corporation (NSPW), and others, to put the current state of the discussion into context, and to frame the basis for Anchor QEA and NSPW's continued concern with the concept. In summary, the dry dredge evaluation can be roughly divided into three key periods, per the following:

- Early Feasibility Study (FS) work
- Data gaps-related Site investigation work and updated analyses

- 2014 Weston Report (and reviews by the U.S. Army Corps of Engineers [USACE])

Each of these is discussed in more detail below.

1.2.1 Multiple Parties Conducted Early Feasibility-Study-Level Evaluations

A project FS (URS 2008) assessed dry excavation as a possible remedial approach, and concluded that it would present "potentially greater risk to human health, because of the need to work behind barriers engineered to keep out the waters of Lake Superior." Meanwhile, the FS identified other potential alternative techniques as being technically implementable, including the following:

- Wet dredging—a far more customary means of removing contaminated offshore sediment—was found to "provide the most long-term benefit at the least cost and with the fewest short-term technical implementation issues."
- On-site sediment confinement in a nearshore confined disposal facility (CDF), or engineered shoreline, would "provide the most long-term benefit with the fewest short-term implementation issues and short-term impacts" (Possible concerns expressed in the FS about the ability to permit this option have been reduced by more recent inquiries and discussions.)

The engineered shoreline/CDF concept was also mentioned as a recommended option for this Site as early as 1996, by consultants working for the Wisconsin Department of Natural Resources on early Site evaluations (SEH 1996).

In 2009, the EPA requested that the National Remedy Review Board (NRRB) conduct a review of the proposed cleanup action for the Site (NRRB 2009). The NRRB indicated its support of a dry excavation concept for the Site, based on the information presented to it at that time. However, after completion of the FS and the NRRB's review, Foth Infrastructure and Engineering, LLC (Foth; Foth 2009) performed geotechnical studies of the Site that identified potential catastrophic risks associated with the potential for dry excavation destabilization and failure due to underlying artesian groundwater pressures pushing upward.

In September 2010, EPA's Record of Decision (ROD; EPA 2010) identified dry excavation as the preferred alternative for nearshore sediment and wood debris at the Site, despite the conclusion of the FS and the concerns raised by Foth (2009). EPA's conclusions in favor of dry excavation appear to have been driven largely by the conclusions of a preliminary engineering analysis conducted by Weston in 2009. Although it was not provided to the responsible parties or the public until well after the ROD was published, Weston had prepared a technical memorandum entitled *Conceptual Geotechnical Assessment for Sediment Removal* (Weston 2009) that presented a preliminary assessment of the dry excavation method as applied to the Site. Unlike the FS, Weston (2009) concluded that dry excavation could be a feasible means of removing contaminated sediments from the offshore area of the Site.

Weston's 2009 report recognized the potential for fatal flaws with the dry dredge approach, and described an approach wherein the Site would be subdivided into numerous smaller excavation enclosures. This concept was not anticipated or evaluated in either the FS or the ROD; and thus, represented a significant deviation from the original concept of dry excavation at this Site.

Several independent technical consultants reviewed Weston's 2009 report and disagreed with its conclusions. Anchor QEA, LLC conducted a detailed review of the Weston Report, as well as field and laboratory data from the Site at the time; the analyses and conclusions are documented in Anchor QEA (2012). Many calculations conducted by Weston were independently performed by Anchor QEA using refined (and, in some cases, revised) assumptions, where appropriate. Anchor QEA's findings were consistent with those of Foth in 2009, and indicated that dry dredging at this Site would be subject to uncertainty and dangerously low predicted levels of stability, as discussed further below.

Meanwhile, independent analyses conducted by Burns and McDonnell (2012) and Gradient (2012), both concluded, like Anchor QEA, that the dry dredge remedy would be unsafe at this Site, and that Weston's conclusions were not representative of the true risks of the remedy concept.

1.2.2 2012/2013 Data Gaps Investigations and Updated Analyses

In 2013, NSPW commissioned an offshore geotechnical investigation, from which Anchor QEA prepared an updated, more detailed geotechnical evaluation titled *Shoreline and Offshore Geotechnical Evaluation Report* (Anchor QEA 2013). This report used offshore data to further confirm serious concerns about the safety and implementability of the "dry dredge" remedy, and concluded the following key points, confirming Anchor QEA's conclusions in the 2012 report (Anchor QEA 2012):

- The Site subsurface conditions are highly variable and cannot be fully predicted with sufficient accuracy. It would be impractical to "design around" potential problem spots.
- At multiple locations, use of established engineering formulas indicates unsafe conditions for dry excavation.
- The use of modified and newly developed analysis formulas would add risk to understanding the dangers of dry dredging.
- Weston's proposal to subdivide the Site into individual "cells" is impractical and not feasible.
- Dry excavation would pose adverse impacts to the community due to duration, cost, noise, and air quality.

During and following the development of Anchor QEA's 2013 report, NSPW maintained a stance of proactive transparency with the EPA regarding calculation procedures and formulas, endeavoring to make sure that the engineering analysis assumptions and approach were understood and vetted by all relevant and involved parties.

1.2.3 2014 Weston Report and Input from the U.S. Army Corps of Engineers

During and following NSPW's efforts to better understand offshore conditions and their implications on dry dredging, and after NSPW submitted Anchor QEA's 2013 report, it appears that EPA solicited a further evaluation from Weston, and ultimately a partial peer review of Weston's approach by USACE. The 2014 Weston Report purports to demonstrate the adequacy and stability of dry dredging as a remedial approach for nearshore sediments at the Site. The USACE peer-review letters dated February 21 and September 23, 2014

(USACE 2014a, 2014b) critiqued certain aspects of Weston's earlier work, and suggested several modifications to their approach.

Despite NSPW's requests for information via the Freedom of Information Act (FOIA) process, neither the USACE peer review letters nor the interim documents by Weston that they appear to have reviewed were made available to the NSPW team. Thus, it is unclear which version of Weston's formulations for basal heave and bottom uplift were peer reviewed by the USACE. It is evident, however, that the USACE peer review did not take into account the 2013 analyses by Anchor QEA (2013), or, for that matter, the similar but independently derived analyses and conclusions developed by Burns and McDonnell (2012) and Gradient (2012). In fact, Weston's 2014 Report gives only a brief recognition of Anchor QEA's 2013 work, but does not present any detailed review comments; and does not cite Burns and McDonnell (2012) and Gradient (2012) at all. USACE's peer review process therefore appears to be incomplete, as does Weston's understanding of conclusions reached by other engineering consultants.

1.3 Summary of Concerns

Anchor QEA has identified a number of concerns regarding the analyses and conclusions described in the 2014 Weston Report. The purpose of this report is to present and describe the bases for these technical concerns, and their implications on the safety and implementability of dry dredging at the project Site. The key concerns fall into several general categories, as follows:

- The multiple levels of aggressive assumptions built into Weston's analyses
- The creation of new, untested an unproven formulas
- Weston's underestimation of construction challenges and practical real-world implementation issues

Each of these is discussed below.

1.3.1 Weston's Evaluation Is Based on Aggressive and Unsafe Assumptions

The evaluation presented by Weston involves a number of individual analytical steps and assumptions that are aggressive. While the critical and complex nature of this project demands a reasonably cautious approach, Weston's individual aggressive assumptions build upon each other, and end up being compounded and combined such that the final conclusions end up being extremely aggressive. In their report, numerous destabilizing factors are dismissed or ignored, all of which would degrade safety and increase risk. The end result of their evaluation process is a dangerous underestimation of the problems that implementing a dry dredge remediation would present.

The implications of instability or failure in a dry dredging scenario at this Site are much too serious for an aggressive and untested approach. Even small dislocations or damages to the surrounding sheet pile wall could lead to quick water entry, which would be difficult to stop once it starts. Of greater concern is the potential for subgrade failure, either through basal heave, bottom uplift, rotational instability, or uncontrolled groundwater piping, or a combination of them. Each of these failure modes, once it begins to take place, could have the effect of amplifying and compounding the others. A failure or collapse of the containment system could not only result in irreparable harm to the environment, but would immediately threaten lives, while exposing and worsening the existing contaminants and environmental impacts to Chequamagon Bay and the community of Ashland. Therefore, a failed remedy would not only be very costly, but could be catastrophic to human health and the environment. Thus, this project demands a reasonably cautious and conservative approach and strict reliance on well-tested evaluation and construction methods.

Figure 1 presents a representation of how Weston's analysis combined a number of aggressive choices and assumptions, resulting in a conclusion that pushes an experimental, unproven, and risky approach to the remediation of this Site. The figure presents six separate rows, each representing a choice or assumption used in evaluating dry dredging. For each subject of analysis, Weston has made aggressive assumptions, and in some cases the most aggressive assumption possible. The subjects of analysis listed in Figure 1 and discussed in detail later in this report, are as follows:

- **Aquitard thickness.** In cases where there is some question about the relevant thickness of the stabilizing Site aquitard layer, Weston has selected the thickest possible assumption. If the average, or thinner end, of the observed thickness is used in the calculations, actual risks of failure are greater than Weston's method. See Section 2 for further discussion.
- **Waves and wave forces.** Weston has selected relative low wave loads—lower, in fact, than those experienced at the Site in the fall of 2014—and has also derived relatively low loading forces from the waves, acting on the surrounding sheet pile walls. For example, Weston assumes the following: (1) pressure from waves breaking against the sheet pile wall will be less than the pressure from non-breaking waves; and (2) significant waves will never break against the sheet pile wall. See Section 4 for further discussion.
- **Formulas used for analysis.** Weston has developed new formulas for their stability analyses, rather than relying on tested, peer reviewed, and published formulas already present in standard engineering literature. See Section 3 for further discussion.
- **Artesian pressures.** In cases where there is some question about the magnitude of the artesian groundwater pressure applied to the basal heave analysis, Weston has applied reduced artesian pressures (lower than measured) acting below the surface. If observed artesian pressures are used in the calculations, actual risks are greater than Weston's hypothetical scenario. See Section 3.2 for further discussion.
- **Drainage conditions in soil.** Weston has not fully accounted for the potential for drained soil conditions to arise during the dry dredging process. See Sections 3.5 and 4.3 for further discussion.
- **Factors of safety.** In cases where different Factors of Safety are cited in engineering literature, Weston has targeted the lowest, or least protective, possible values for their analyses, despite clear guidance that complex projects with potential for catastrophic public effects demand the selection of higher values. See Section 3.4 and 3.5 for further discussion. Given the consequences of failure (potential loss of life and irreparable environmental impacts), varying Site conditions, and complexity of the project, conservative and accepted Factors of Safety are warranted.

As depicted on Figure 1, the end result of their evaluation, when each step is compounded, is an aggressive and risky approach, rather than an appropriately cautious and safe approach, that recognizes the very real and severe consequences of failure at this Site.

1.3.2 Weston's Analysis Underestimates, and in Some Cases Ignores, Inevitable Construction Challenges and Impacts to the Community

Aside from the details of the technical analyses, there are serious issues regarding the implementation and constructability of dry dredging. Section 5 of this report provides further discussion of the following basic points:

- The dry dredging project concept has become much more involved and complicated than was ever contemplated in the Feasibility Study or ROD.
- Dry dredging will cause problems with noise, odor, and landside disturbance—much more than wet dredging would.
- Weston has underestimated the length of time that dry dredging-related construction would require. The work would have to occur over numerous construction seasons, extending its community impacts.
- Weston unwisely discounts the effects and challenges that piping and bottom uplift would cause to the construction.
- Weston's proposed mitigation techniques for responding to possible instabilities are neither realistic nor implementable.
- Implementability issues aside, the dry dredging remedy concept would prove far more costly than was projected in the ROD.

2 AGGRESSIVE ASSUMPTIONS REGARDING SUBSURFACE CONDITIONS

Weston has re-evaluated both the subsurface stratigraphy and the selection of soil strength properties at the Site, and has focused their efforts on three selected previous explorations: offshore borings AQ-SB-02 and AQ-SB-04; and shoreline SB-185. They chose these explorations because they were identified by Anchor QEA (2013) as the locations where the lowest Factors of Safety were calculated. Inherent variability in the Site subsurface could, in fact, result in the presence of even more critical locations, beyond those that have already been identified.

Both the aquitard thickness and the assumed strength values are described in more detail below.

2.1 Weston Made Aggressive Assumptions Regarding Aquitard Thickness

Although the soil strength properties that they derived for these locations do not differ significantly from those used by Anchor QEA (2013), their interpretation of aquitard thickness is an example of their application of the most aggressive possible interpretation when faced with more than one possible assumption.

For explorations AQ-SB-02 and AQ-SB-04, seams of relatively sandy material deep in the Miller Creek Formation (MCF) were concluded by Anchor QEA's analysis (Anchor QEA 2013) to be capable of allowing for groundwater flow and elevated groundwater pressures, and should therefore be considered as part of the Copper Falls Formation (CFF) aquifer. This reasonable assumption resulted in thickness of the MCF aquitard being less than would be required to resist basal heave and bottom upheaval. Weston, on the other hand, attempts to argue in favor of a more aggressive assumption of a thicker aquitard, using an argument based primarily on the sandy material's behavioral properties, and the clay content effects on these properties. They conclude that the sandy layers should be considered as part of the aquitard. This has the effect of increasing the interpreted thickness of the aquitard, which leads to higher Factors of Safety in stability calculations. In both cases, as summarized in Table 1, Weston has interpreted the sand seam to be a continuation of the aquitard, which allows them to apply the highest possible interpretation of aquitard thickness.

Table 1
Comparison of Aquitard Thicknesses Assumed

Exploration Number	Range of Possible Aquitard Thickness* (per Anchor QEA 2013)	Selected Aquitard Thickness (Weston 2014)
AQ-SB-02	28.7 to 37 feet	37.4 feet (thickest possible interpretation)
AQ-SB-04	41.4 to 47.4 feet	47.3 feet (thickest possible interpretation)

Note:
*Based on the presence of a sand seam at depth

2.2 Weston Did Not Account for Effects of Construction on Strength Loss

Another area where Weston has disregarded potential uncertainty is in their selection of strength properties for the subsurface soil types. They have not attempted to account for any possible effects of pre-existing irregularities or fractures in the soil layers, nor for any soil strength degradation that may develop in response to disturbance from sheet pile installation. Even if the potential for existing discontinuities or fractures were to be disregarded, as Weston has done here, it is important to bear in mind the potential for the soil layers to undergo disturbance in response to sheet pile wall installation, dry dredging, and artesian groundwater pressures, each of which could lead to further disturbance and strength degradation as the work proceeds. For this reason, in light of these potential issues, the Factor of Safety needs to be selected with these uncertainties taken into account.

2.3 Weston Did Not Fully Account for the Effects of Soil Drainage State

Because the water's interaction with soil is so important to the soil's strength and behavior, it is essential for geotechnical engineering analysis to account for the state of soil drainage that would apply. Typically, when forces act upon soils (such as new surface loads, or unloading due to excavation), the soils initially respond in a "non-drained" manner before the water bound up in the soil mass has been able to flow in or out in response to the forces to normalize pore pressures. This represents a short-term, "undrained" stress condition in the soil. If the forces remain for a prolonged period of time, allowing the water pressure of the soil to adjust by flowing in or out of the soil, then "drained" conditions can develop. The

time required for drained conditions to take effect depends on the nature of the soil. A granular soil will drain far faster than a fine-grained soil such as clay.
Anchor QEA's 2013 geotechnical evaluation of the Site included a series of porewater dissipation tests within various subunits of the Miller Creek Formation aquitard, which indicated that pore pressures dissipated in a number of hours or less in the MCF upper silt layer, and therefore concluded that:

> *It is expected that dry excavation will subject underlying soils to loading conditions gradually enough for a state of partial drainage to occur, particularly in the slightly more permeable upper silt layer of the aquitard.* (Anchor QEA 2013)

The deeper, more clayey portion of the MCF drains slower and would require a significantly longer time period to behave in a drained manner.

The importance of accounting for possible development of drained conditions during excavations has been documented in geotechnical literature, as the drained case can be more critical than the undrained case:

> *Stability is therefore controlled by the drained strength of the soil corresponding to equilibrium (long-term) pore pressures. This case represents the critical condition for unloading problems that generate negative excess pore pressures (u_e<0) during construction (e.g. excavations in stiff clays) since the factor of safety decreases with time due to swelling.* (Ladd 1986)

And:

> *When a slope in clay is created by excavation, the pore pressures in the clay decrease in response to removal of the excavated material. Over time, the negative excess pore pressures dissipate… the effective stresses in the clay around the excavation decrease, and the factor of safety of the slope decreases with time.* (Duncan and Wright 2005)

The selection of undrained versus drained parameters is dictated by the type of soils encountered and the duration of construction. The application of undrained conditions assumes that construction (draining within sheet pile wall, excavation, backfill, and refilling of the excavation with water) occurs relatively quickly enough so that the soils do not experience significant pore pressure dissipation that would decrease the strength of the soil. However, if the excavation is left open for a period of several weeks, then drained conditions may be able to develop in part or all of the soil mass. It would be risky not to consider this possibility, as Weston appears to have done in some of their analyses, as described below.

3 STABILITY ANALYSES ARE BASED ON UNCONSERVATIVE AND/OR UNTESTED ASSUMPTIONS

Perhaps the most significant feature of Weston's 2014 analyses is their formulation of new recommended quantitative analysis procedures for basal heave (Appendix B of the 2014 Weston Report) and bottom uplift (Appendix C of the 2014 Weston Report). Although they utilize several sources and theoretical considerations in their formulations of these recommended formulas, the resulting equations for basal heave and bottom uplift are, at this point, recommended procedures only, and have yet to be tested by industry peers, practical design use, or field experience. These newly derived Factor of Safety equations are essentially experimental in nature, as they have not been vetted or tested in the geotechnical industry, and lack a history and track record of being applied and implemented successfully. Because the consequences of failure are severe, the dry dredge scenario at this Site is neither the time nor the place to experiment with the application of novel and untested theories.

Aside from that overarching concern, we have found flaws in logic with their application of the various stability formulas. This section presents details and concerns with the various geotechnical analyses that were conducted by Weston.

3.1 Basal Heave Analysis

For their evaluation of basal heave, Weston provides a detailed derivation and discussion of their proposed Factor of Safety equation as well as a step-by-step application for the Ashland Site (Appendix B of their report). The principle formulation of their Factor of Safety equation is based on a procedure presented in a 2002 paper by Goh and Wong, which, in itself, is a variation of a procedure presented by Terzaghi in 1943.

This equation represents a significant shift in calculation technique compared to their previous evaluations in 2009 (Weston 2009). In their original Technical Memorandum published in November 2009 for which the EPA ROD is based, Weston utilized a procedure known as the Negative Bearing Capacity (NBC) method. In Appendix D of that memorandum, Weston stated the following as justification for selecting the NBC method for assessing basal heave:

The NBC method, in particular the Bjerrum and Eide's modification to the NBC theory for 2 cohesive layers, appears to yield reasonable results and are widely used.

Weston has a history of modifying their analysis approach for this project. In 2009, they first established an initial precedent by originally selecting the NBC method, and the ROD was based on conclusions Weston derived from the NBC method, which has the advantage of being a standard formula that is well known to the geotechnical industry, and has been for decades. While we have not been afforded the opportunity to review Weston's interim work (apparently the subject of USACE peer review, as discussed in Section 1.2.3), it seems clear that they proposed alternative calculation approaches to USACE, and then made further adjustments for the 2014 Weston Report.

Besides the fact that Weston has continued to modify their approach to the calculations, it is difficult for us to envision any professional engineer relying on untested equations and analytical procedures such as those proposed by Weston to design and implement a sediment remedy in the field. This would be a practice that is essentially experimental in nature. When worker safety and community environmental protectiveness is at stake, the design should not be reliant on an untested method that would be used experimentally for the proposed dry dredge application. The analysis should be performed with more widely used analytical procedures that offer a track record of success.

3.2 Determination of Artesian Groundwater Pressures for Basal Heave

Another area where Weston has proposed a new calculation approach is the selection of groundwater pressure for use in the basal heave formula. Weston has modeled the pore pressure through the MCF as a linear distribution. The pore pressure at the base of the MCF is consistent with the artesian pressure measured at the top of the CFF. The pore pressure at the top of the MCF is zero for areas within the excavation and consistent with the hydrostatic pressure from surface water of Chequamegon Bay for areas outside of the excavation. Weston appears to have assumed the magnitude of the artesian head (labeled as parameter H_{AH}) to be 11.13 feet, 11.48 feet, and 11.10 feet for AQSB-02, AQSB-04, and SB-185, respectively. Anchor QEA was unable to discern Weston's methodology for determining the pore pressure values at the base of the MCF. It appears neither the highest nor the lowest pore pressure dissipation results were assumed, but rather some average or weighted average of nearby Cone Penetration Test results and/or monitoring wells were used.

Anchor QEA strongly believes that the artesian head assumed for determining the pore pressure at the base of the MCF should be chosen carefully, bearing in mind the need for stability and safety, and recognizing the documented subsurface variability that characterizes this Site (Anchor QEA 2013). This is due to the fact that pore pressures can fluctuate seasonally and without the benefit of long-term pore pressure measurements of the CFF, it is not possible to determine the level of conservatism for the dissipation test results with respect to the most critical case. Even if stability analyses assumed the maximum value from the pore pressure dissipation results at locations near the area of interest, the pore pressure value would only represent a very discrete period of time and would be very unlikely to be that of the critical case. The actual artesian pressure could still in fact be significantly higher at the time of construction should dry excavation be performed during a time of year when artesian pressures are highest.

3.3 Bottom Upheaval

For the evaluation of bottom uplift, Weston provides a detailed derivation and discussion of their proposed recommended Factor of Safety equation, as well as a step-by-step application for the Ashland Site (Appendix C of their report). The principal formulation of their Factor of Safety equation is based on the commonly used Factor of Safety equation for bottom uplift in excavations, which evaluates the ratio of soil overburden to uplift from porewater pressure. Citing a 2012 technical paper by Demetrious Koutsoftas (i.e., the Koutsoftas paper), Weston has chosen to include adhesion between the soil and sheet pile wall as an additional stabilizing force (i.e., shear resistance).

The Koutsoftas paper, titled *Excavation Stability against Hydrostatic Uplift: Islais Creek Contract D – Army Street Segment*, and pertinent to braced excavations and focusing on specific project examples in San Francisco, is prominently cited by Weston as justification for inclusion of shear resistance in their bottom uplift evaluation. In Figure 5 of the Koutsoftas paper, which Weston references, shear resistance has been included for a narrow (11-meter-wide) braced excavation with a very stiff shoring system. However, at the Ashland Site, the excavation is much wider, with a flexible shoring system (i.e., a non-braced, cantilevered sheet pile wall), so its behavior would be expected to differ from the long and narrow geometry investigated by Koutsoftas, in that the greater distance between walls would lessen their influence and introduce more of a potential for upward movement in the center of the larger excavation envisioned for the dry dredge scenario. Because of their narrow geometry and controlled nature, the project descriptions of the Koutsoftas example are not directly applicable to the Ashland Site. The Koutsoftas paper does not recommend or imply that shear resistance is reasonable to include in all bottom uplift evaluations. Weston's inclusion of shear resistance is aggressive, and plays a significant role in the stability evaluation for bottom uplift.

Weston originally attempted to include shear resistance in a FS equation they derive for their 2009 report. At the time, the Koutsoftas paper had not yet been published, and Weston's 2009 equation included shear resistance using a different formulation than Koutsoftas. Weston had essentially self-derived an equation that lacked literary supported justification for the inclusion of shear resistance which inflated the Factor of Safety against bottom uplift. While some degree of wall adhesion may well exist between soil and sheetpiling, Weston overstates the reliability of shear resistance as a contributing factor for bottom uplift. It is not reasonable to assume that soil shear strength will mobilize uniformly over the entire Site, or even over the 200-by-150-foot area under Weston's revised segmented approach. In fact, it will be further degraded by potential groundwater upwelling or piping (mentioned below in Section 4.4). The insistence that shear resistance be included in the evaluation, and Weston's inconsistency in Factor of Safety equations between the one derived for their original 2009 report to their more current 2014 report further demonstrates the impractical and experimental nature of Weston's analysis, and the concept of performing dry dredge at the Ashland Site.

3.4 Weston Has Selected Aggressive Factor of Safety Levels for their Analyses

Weston's selection of a minimum allowable Factor of Safety is another area where they have selected a decidedly unconservative approach on more than one occasion. Given the high degree of heterogeneity of Site lithology, soil conditions (see Anchor QEA 2013 for discussion on site lithology and soil condition variability), the presence of a substantial artesian condition underlying the Site, the large scale of the excavation, and the potential consequences of failure, Anchor QEA recommends a Factor of Safety of at least 1.5 for the evaluation of bottom uplift. Weston, meanwhile, supports a minimum Factor of Safety of 1.25 for evaluations of bottom uplift. This is particularly important given the fact that Weston, for example, determined a Factor of Safety value of 1.4 for location AQ-SB-02.

The range of Factors of Safety observed in literary references for the evaluation of bottom uplift range from 1.2 to 1.5, with 1.5 being a more commonly recommended value. However, all of these references recommend the simple form of the Factor of Safety equation that does not include the modifications proposed by Weston, such as shear resistance, and does not account for uncertainties associated with artesian conditions. No engineering references support applying a Factor of Safety of 1.25 across the board for all projects—especially given the unique conditions of this Site, where the subsurface variability and uncertainty are combined with serious risks to human health and the environment that would result from failure.

Weston's rationale for targeting the 1.25 Factor of Safety value rests on apparent consistency with the USACE's peer review letter dated February 21, 2014 (USACE 2014a), which itself briefly cites a passage from Bowles' *Foundation Analysis and Design* (Bowles 1996) related to analysis of temporary excavations. However, no further context is provided for their use of Bowles' text. This is important because the context of the project, the risks, the extent of subsurface knowledge, and the ability to respond to unknowns in the field all need to be taken into account when determining an appropriate Factor of Safety. It is not merely a matter of pulling a number out of a single reference without consideration of the overall context and project-specific conditions. Anchor QEA has recommended that a Factor of Safety of 1.5 be utilized for bottom uplift, based on recommended practice by the United States Navy (NAVFAC 1986), work by Fang (1991), and consideration of the inherent risks of performing dry dredge excavation at the Ashland Site. Therefore, the minimum Factor of Safety value of 1.5 is well supported and justified given the importance and risks associated with dry dredging, and it remains the prudent minimum value in our view. This would mean that Weston's recalculation of the Factor of Safety for bottom uplift at AQ-SB-02, is, in fact, below (less safe than) the appropriate threshold level.

3.5 Piping Instability and Rotational Instability

Groundwater piping represents a quickly-developing erosive condition that cannot be effectively controlled once initiated. Weston discounts the importance of piping, saying simply that it can be controlled by "active dewatering of the excavation bottom using readily available bottom suction pumps." Other effects of groundwater piping that are not accounted for here could include loss of the soil adhesion that they include in their bottom upheaval analysis. Again, Weston has chosen to neglect an effect that could prove important and damaging to the successful execution of the work.

Weston has applied an aggressively low Factor of Safety to the condition of groundwater piping, much as they have done for bottom uplift (Section 3.4). In the 2014 Weston Report, the minimum Factor of Safety is stated as being 2.0, but this is not only inconsistent with standard engineering practice, but is much more aggressive than their own work in 2009, in which they proposed a minimum Factor of Safety of 4 to 5 for the piping evaluation. A minimum Factor of Safety of 2.0 for piping is less than half of the industry accepted range of values of 4 to 5, which was the range that Weston themselves originally recommended in their 2009 work. The reasons for the industry-standard Factor of Safety being relatively high include the following:

- Piping is difficult to predict in advance
- Piping develops progressively (starting small but growing)
- Piping is virtually impossible to stop once it has started
- The increased flow of water into and through soil layers can weaken soil strength, can loosen pre-existing irregularities and fractures, and can decrease or eliminate frictional forces acting along embedded sheet piling

A dramatic and well-known example of piping in action was the disastrous piping-caused failure of the Teton Dam in Idaho in 1976, in which an area of water seepage out of the face the dam appeared visually innocuous at first, but quickly grew such that the entire dam face collapsed later that day. That event led to 11 deaths and is one of the key justifications for the engineering industry applying such a "high" Factor of Safety requirement for groundwater piping.

Meanwhile, Weston also utilized slope stability software (SLOPE/W) to evaluate rotational stability as an additional evaluation of possible failure modes. In their analysis, as for the sheet pile wall analysis discussed in Section 3.3, Weston did not account for the possibility that drained soil conditions could develop during the period of excavation. Instead, they conducted the analysis as an evaluation of undrained conditions, applying the artesian water pressure as a surcharge load, reflecting a short-term, temporary scenario only. The equivalent analysis using drained soil conditions result is a Factor of Safety that is somewhat lower, and illustrates again the repeated use of unconservative assumptions in Weston's work.

3.6 Weston Has Changed their Recommended Factors of Safety over Time, which Greatly Affects their Conclusions

Between their work in 2009 and in 2014, Weston has changed not only the engineering formulas that they are using to conduct the analysis, but also their recommendations of Factors of Safety for two of the three key analyses discussed here—Basal Heave, and Piping Instability. Table 2 summarizes the Factors of Safety that they recommended in 2009 and in 2014, as well as the Factors of Safety that they calculated for the corresponding analyses in their 2014 report.

Table 2
Comparison of Factor of Safety Selections

	Weston (2009)	Weston (2014)	Factors of Safety calculated by Weston (2014)	Factors of Safety calculated by Anchor QEA (2013)	Anchor QEA recommendation (2013)
Basal Heave (Section 3.1)	1.2	1.50	1.8 – 4.7	0.55 – 6.92	1.50
Bottom Uplift (Section 3.3)	1.25	1.25	1.4 – 1.5	0.95 – 1.48	1.50
Piping Instability (Section 3.5)	4 to 5	2.0	3.7 – 5.1	(not calculated)	4 to 5

Table 2 demonstrates that for both bottom uplift and piping instability, their selection of Factors of Safety makes a tremendous difference in how their results are interpreted. The industry standard Factors of Safety that Anchor QEA has continued to recommend would imply the potential failure of both modes. Weston's more aggressive Factors of Safety allow them to present the misleading conclusion that these analyses indicate stability.

4 SHEET PILE WALL STRUCTURE HAS BEEN UNDER-DESIGNED

Weston has taken an aggressive approach to designing sheet pile structures for the dry dredging scenario. In fact, they have selected wave sizes for their analysis that are in fact smaller than the waves that were experienced at the Site in September 2014, when 3- to 4-foot- wave were observed. Their underestimation of wave heights has translated into under-represented forces acting on the wall, which is particularly dangerous given the challenging conditions of Chequamagon Bay and the importance of maintaining sheet pile wall stability and water-tightness while dry dredging is implemented.

At the time of this writing, NSPW is developing a design for a water quality containment system to be used during a wet dredge pilot study (in progress). Because the current design involves the use of sheet pile walls, the design process is very similar to what Weston presents for a dry dredging scenario. We have found that a number of elements in Weston's analysis, described below, are actually less conservative than the design assumptions that are being made for the wet dredge pilot study containment system. In fact, the opposite should be true—a dry dredge wall design should be more conservatively designed than a wet dredge wall design—because the consequences of damage, sheet pile movement, ground movement, and instability are so much greater for dry dredging than they would be for wet dredging. In other words, failure of the wall during a wet dredge condition will not have the consequences of potential loss of life and/or irreparable environmental harm that would be associated with failure of the wall during a dry dredge project.

With a wet dredging scenario that utilizes a sheet pile wall, there is water behind the wall that helps balance pressures on both sides compared to a wall in a dry dredge scenario, where water behind the wall is removed causing an unbalanced pressure condition. Perhaps the most obvious example of this is a comparison of the effect of a small gap opening up between individual sheet piles (a possible result of wall deflection). Under a wet dredging scenario, a small gap would likely be unnoticeable, and certainly of little consequence. However, under a dry dredging scenario, even a small opening between sheet piles would allow pressurized water from the surrounding Bay to enter the excavation immediately, and once water begins to enter through the wall, it could be very difficult to stop or even stabilize it. Overall, the consequences of underestimating pressures, forces, and effects would be much more dangerous under the dry dredging scenario.

The following sections present further detail regarding Weston's evaluation of sheet piling for dry dredging, demonstrating the aggressive and unrealistic approach that Weston has taken with their analysis.

4.1 Estimation of Wind-generated Waves

Wave forces and lake levels experienced at the Site would have a profound effect on the stability of the sheet pile barrier walls. Experiences during the Wet Dredge Pilot Study that was set up in September of this year provided a vivid example of how challenging conditions can be in Chequamegon Bay and at this Site. Wind-generated wave forces were identified as the dominant force in the structural design of the 200-foot offshore cantilevered sheet piling retaining structure.

Appendix A of Exhibit 4 of the 2014 Weston Report presents details on the wave force calculations, although it provides no technical background regarding how the wave values were derived, so it would be difficult to evaluate the basis for their predictions. From the report, it appears that a 50-year return-interval event was selected for the preliminary structural design. However, there was no discussion of why this event was selected as the basis of design. In addition, no information was provided regarding how the 50-year return-interval event was determined (wind speeds, lake levels, model inputs, etc.). Graphical output of two-dimensional wave transformation model results for the 50-year event were included in the Exhibit (which were included as a PowerPoint briefing packet, not a detailed description.) The results indicate that the significant wave heights ranged between 1.75 feet and 2.5 feet for the 50-year event.

Table 3 summarizes the wave heights and lake levels assumed by Weston, compared to Anchor QEA's design assumptions, and to actual conditions observed in Chequamegon Bay during the storm that occurred on September 10, 2014 during Pilot Study work.

Table 3
Comparison of Significant Wave Height Assumptions

	Weston (2014)	Anchor QEA (2014) Pilot Study Design	Site Observations, September 2014
Storm Event Recurrence Interval	50-year	100-year	Approx. 100-year
Significant Wave Heights	1.75 to 2.5 feet	Greater than 3.3 feet (1.0 m)	3 to 4 feet[1]
Static Lake Level	602.0 feet	603.0 feet (1 foot above normal lake levels)	603.0 feet (1 foot above normal lake levels)

Notes:
1 Note that these wave heights were observed wave heights and not necessarily significant wave heights.
m = meter

For comparison, for Anchor QEA's design of a water quality protection system for the Wet Dredge Pilot Study, a 100-year return interval storm event was used as the basis of design, per discussions with agency representatives on April 9, 2014, and as documented in the *Pilot Study Design Package* (Anchor QEA 2014). Our predictions for the 100-year storm event predict significant wave heights greater than 3.3 feet, which is consistent with findings by URS (2007) and CHE (2012).

During the original implementation of a wet dredge pilot study by NSPW and Anchor QEA in fall 2014, a storm event on September 10 sent 3- to 4-foot-high waves toward the Site, in conjunction with a seiche (temporary lake-level rise) of 1 foot. This event verified the importance of preparing for sizable storm and wave events at the Site.

The suggested wave heights used in Weston's sheet pile analysis are smaller than what has been observed at the Site, and what was used for design of the Wet Dredge Pilot Study. Furthermore, Weston has failed to take potential seiche into account at all. If anything, the wave and lake levels assumed for a dry dredging scenario should be equal to or greater than those used for wet dredging design, since wall damage or failure under a dry dredge scenario would be far more problematic, and possibly catastrophic, compared to the effects of damage in a situation where lake levels are maintained on both sides of the wall.

It is not apparent whether or not Weston incorporated the effects of seiche or other lake level variations in their analysis. Lake level variations could occur both from seiche effects and from other, longer-term climactic and regional factors. As important examples, a 1- to 2-foot temporary seiche effect accompanied the storm event of September 9 and 10, 2014, and lake levels over the past year (2014) have been observed as approximately 1 foot higher than Weston's presumed lake level elevation of 602 feet.

4.2 Development of Wave Forces

Weston's analysis of wave forces included both non-breaking and breaking waves for the design of the sheet pile wall system. The method used for calculating non-breaking wave forces was clearly defined, but the method used for calculating the breaking wave forces was neither discussed nor referenced. Therefore, the method for determining the breaking wave could not be evaluated. However, the forces actually applied by breaking wave forces are likely to be higher than the values provided in Weston's report. We are concerned that they have seriously underestimated the forces applied by breaking waves. Evidence for this is that the total breaking wave forces as shown on page E4.A2-9 of the 2014 Weston Report are actually smaller than the design non-breaking wave force on page E4.A-14, therefore yielding smaller sections of sheet pile. This is very unusual, considering that the energy from a breaking wave is greater than a non-breaking wave condition, and that the water depth used for the breaking wave condition is greater than the non-breaking wave condition.

For comparison, the currently ongoing development of a potential containment system for the Wet Dredge Pilot Study uses methods and procedures prescribed by the standard reference by the American Society of Civil Engineers (ASCE) 7-10 (ASCE 2011) for the calculation of the breaking wave force. Using equation 5.4-6 in Section 5.4.4.2 ,*Breaking Wave Loads on Vertical Walls* in ASCE 7-10, with the same water depth, top of sheet pile wall elevation and using a dynamic pressure coefficient, C_P, of 2.8 yields a total wave force *four times greater* than the breaking force given on page E4.A2-9 provided by Weston.

4.3 Design of Cantilevered Sheet Pile Wall Does Not Correctly Account for Soil Drainage State

The use of the ProSheet computer program, as Weston used, is a standard industry practice for sheet pile selection. However, in their use of the program, Weston has not fully accounted for the possibility that drained conditions could develop during the excavation process. The importance of this consideration is described above, in Section 2.3. While Weston applied drained soil parameters on the retained (outer) side of the wall, they used undrained soil parameters on the excavated (inner) side, and did not evaluate the possibility that drained parameters could also apply on the inner, excavated side of the wall.

When the same ProSheet sheet pile analysis is conducted using drained soil parameters on both sides of the wall, the Factor of Safety is much lower. The results are included as Appendix A to this report.

4.4 Implications to Construction Schedule

Because of Weston's apparent under-estimation of wave heights and forces (described in the previous section), a finalized sheet pile design would indicate that a much more robust structure would be needed. The end result of Weston's sheet pile wall analysis is a cantilevered sheet pile section, but cantilevered sheet piling alone would not be sufficient to remain safe, stable and water-tight. The more complex wall would add complexity and time to the installation process, reducing the amount of construction season available for the dredging work. In fact, given the timeframes necessary to install, and later remove, the sheet piling, there would be limited time available for in-water construction on Chequamegon Bay during each construction season. The overall duration of the dry excavation work would need to extend over multiple construction seasons, greatly expanding the timeframe of the project as well as its overall costs.

5 WESTON HAS UNDERESTIMATED THE PROBLEMS THAT IMPLEMENTATION WILL FACE

Aside from their geotechnical analyses, Weston's argument in favor of dry dredging fails to recognize the actual field challenges that the remedial action would present. They adopt an idealized and simplified concept of dry dredging implementation, whereas in reality the work would pose a number of difficulties in its construction and community impacts.

5.1 Dry Dredging Would Pose Harmful Effects on the Community

Installation of sheetpiling is going to be a much more difficult process than envisioned and described by Weston, with debris, rock, and natural variability of soil strengths in the MCF presenting areas where hard driving or refusal are likely to be encountered. Borings 88-5 and AQ-SB-03 are examples of explorations that encountered rock or similar obstructions. These variable subsurface conditions and driving difficulties would cause a very difficult, if not impossible, situation in attempting to achieve a consistently "impermeable" wall, in which each connection between sheets remains perfectly sealed. Thus, a reliably impervious and structurally intact wall is an unrealistic expectation.

The issue of time duration and schedule is not addressed in Weston's analysis, but each episode of sheet pile installation is likely to require a period of several months. It is likely that more than 3 months would be needed just to install sheet piling around a single excavation cell, including penetration of the sheets, removal of debris and obstructions, and structural details and seam preparation. Similarly, sheet pile removal at the end of the construction season would also occupy several weeks. Therefore, much of the available 7- to 8-month construction season in Ashland, Wisconsin, would be occupied by installation and removal of the sheet pile system. With so little time available for remedial work during each construction season, a full accounting of the dry dredge option needs to account for numerous construction seasons.

Community impacts from dry dredging could be significant—both from the noise of sheet pile installation, and the possible odors that would be produced once the lakebed is exposed after water removal. In wet dredging, the water has the advantage of covering the contaminated sediments, and oil sheen on the water surface can be readily controlled. With the lakebed exposed to the open air, controlling air quality will be much more difficult.

Another way that dry dredging would cause negative community impacts stems from the fact that dry dredging would be heavily reliant on landside activity and on-shore equipment staging and traffic access, thus causing noise, traffic, and visual impacts to the community. These impacts would occur for the duration of the project, which would span multiple seasons and may potentially also include multiple episodes of sheet pile wall installation. For comparison, wet dredging equipment is mobilized on water and does its work offshore, which helps to mitigate the on-shore visual and noise impacts from the work. The long duration of the project may also mean that the City cannot move forward with its waterfront redevelopment plans for many years to come.

5.2 Suggested In-field Mitigation Steps Are Unrealistic and Dangerous

Weston states in their report that some upheaval and fracturing is common within dry dredge excavations. While this may be true, the practicality and timeframe in which "field" remedies could be constructed and implemented, coupled with the shallow embedment design depths of the cantilever sheet pile system may result in a much higher risk to worker safety, project costs, project delays, and neighborhood impacts.

Weston acknowledges the potential for soil instability within the dry dredge area:

> *It must be noted that unanticipated subsurface conditions may be encountered during the dry excavation field work which would have the effect of reducing one or more of the calculated FS values for the five instability scenarios to unacceptable values. (p. T.19)*

However, Weston has overestimated the ability to detect such dangers in the field, mentioning only the possible appearance of a thicker impacted sediment layer, and failing to address the fact that a thinner area of the MCF layer would be completely undetectable to construction crews, yet would be even more impactful to stability Factors of Safety than incidences of thicker impacted sediment would. Also, they underestimate the difficulty of achieving a reliable approach to mitigating the condition, if it were recognized at all. If, for example, the bottom of the MCF were at a higher elevation than expected, the reduced aquitard thickness would degrade the Factor of Safety—but there would be no way of knowing this in the field prior to installation of the sheet pile. Hence, it would not be possible, in practice, to recognize that the design needed modification. Rather, the remedy failure would be observed before any awareness that the estimated aquitard thickness is incorrect.

Another example is the idea of using wells to reduce water pressures in the underlying aquifer, a misleadingly simple concept that would be very difficult, if not impossible, to implement successfully. These wells would be installed offshore, so their installation would be much more complicated and prone to imprecision than on-land well installation methods. Furthermore, the amount of time needed to do the following: 1) identify the existence and bounds of problematically variable soil layers; 2) devise and design a solution, including well locations, screened intervals, and the like; 3) install and develop the wells; and 4) achieve and verify the desired degree of depressurization—all in an area with significant spatial gaps between known subsurface conditions, requiring guesswork and approximations each step of the way—renders such an idea thoroughly unworkable within the timeframes of the dry dredging concept, and certainly not in enough time to stave off developing instabilities. Attempting to depressurize the aquifer could also lead to other problems, such as settlement of adjoining land areas and disruption to groundwater flow (potentially leading to further spread of the upgradient groundwater contamination plume below the Phase 1 area).

Weston acknowledges the potential for bottom instability, but dismisses the risks and downplays the potential consequences of failure:

> *It is common, especially when working in wet environments, to experience localized non-critical manifestations of what may be considered quasi bottom uplift or exit gradient instabilities where the excavation bottom may deflect slightly upward on the order of several inches… these developments are non-serious and commonplace in excavation work, and are typically and easily remedied by staged excavation and*

> *bottom grading activities, and/or active dewatering of the excavation bottom using readily available bottom suction pumps. (p. T.20)*

This statement under-represents the importance of incipient basal heave and bottom uplift, which would be indicators of soil layer disturbance, entry of pressurized groundwater into subsurface fractures and along the embedded sheet pile walls, and degradation of overall excavation stability. It also oversimplifies the implementation challenges that such evidence of growing instabilities would represent, as it mentions only the most obvious, surficial response possible (pumping out the water) while ignoring the fact that underlying soil conditions would be worsening and becoming less stable for the excavation.

Finally, Weston understates the effort associated with performing dry dredging in a segmental fashion, saying that:

> *…it is common practice for projects of this nature to be completed by instead creating smaller sheeted cells… this remedial concept permits a much more easily managed, efficient and cost effective staged construction approach to be used instead, in which one cell at a time can be remedied. (p. T.18)*

Weston, however, provides no justification for this opinion. In reality, breaking the project into individual dry dredging segments would greatly add to project duration and cost, as each segment would need to undergo sheet pile driving, dewatering, and excavation separately from the others. The total amount of sheet pile driven is naturally and inevitably significantly greater in a segmental approach. Time, cost, and community impacts would increase proportionately. Additional rows of driven sheet piling would also increase the disturbance of soils within the MCF, further worsening the stability concerns.

5.3 The Dry Dredging Concept Is Now Far Different and Less Cost-effective than Was Envisioned by the Feasibility Study and Record of Decision

The final point to be considered regarding implementability of dry dredging is its overall cost. Following Weston's analyses, the dry dredging remedy can now be seen to include a host of construction elements that would inflate costs well beyond what the FS anticipated. These include the use of temporary earthen roads into the excavation areas, continuous pumping of water inflow, and, most importantly, the potential for numerous episodes of sheet pile installation, removal, reinstallation, and so forth, as the project proceeds in a segmental fashion. Furthermore, as is documented herein, Weston has underestimated the nature and necessary strength of the structural wall, inflating the remedy cost even more. Overall the dry dredging project has gradually become a far larger and more difficult, expensive, and dangerous concept than was expected at the time of the FS.

6 CONCLUSIONS

The Ashland Lakefront project is no place for an aggressive and uncertain remedial approach. The combination of debris, unpredictable weather and lake conditions, and highly variable soil and groundwater properties demands a thoughtful and careful approach. The Site, its chemical and geotechnical characteristics, and the environment and climate of Chequamegon Bay pose variable conditions that significantly affect the technical design and construction approach.

For many key steps of their analysis, Weston has used aggressive assumptions or overlooked important areas of concern. They have selected the thickest possible interpretation of aquitard layer thickness in cases where the aquitard appears to actually be thinner; they have underestimated the size and forces of waves that could impact the Site (and therefore underestimated the size of wall that would be needed); they have developed new formulas for calculating stability that are untested by the engineering industry; and they have selected overly aggressive Factors of Safety.

Even with all of these aggressive assumptions, they recognize that a potential for instability still exists, but attempt to justify this result by describing possible in-the-field mitigation techniques, such as localized deep aquifer dewatering, that are not reasonable nor implementable in reality.

Altogether, Anchor QEA continues to strongly recommend that NSPW and EPA not move forward with a dry dredging remedy at this Site. There are far too many dangers associated with variability in the subsurface, the surrounding conditions of Chequamegon Bay and Lake Superior pose numerous challenges to construction and safety, and the results of failure are too dangerous and environmentally catastrophic.

Dry dredging is the wrong choice for this Site. Other proven methods are better suited to the work, including wet dredging (the subject of a Pilot Study being designed by NSPW) and engineered shoreline (where sediments are stabilized and used for Site shoreline improvements).

7 REFERENCES

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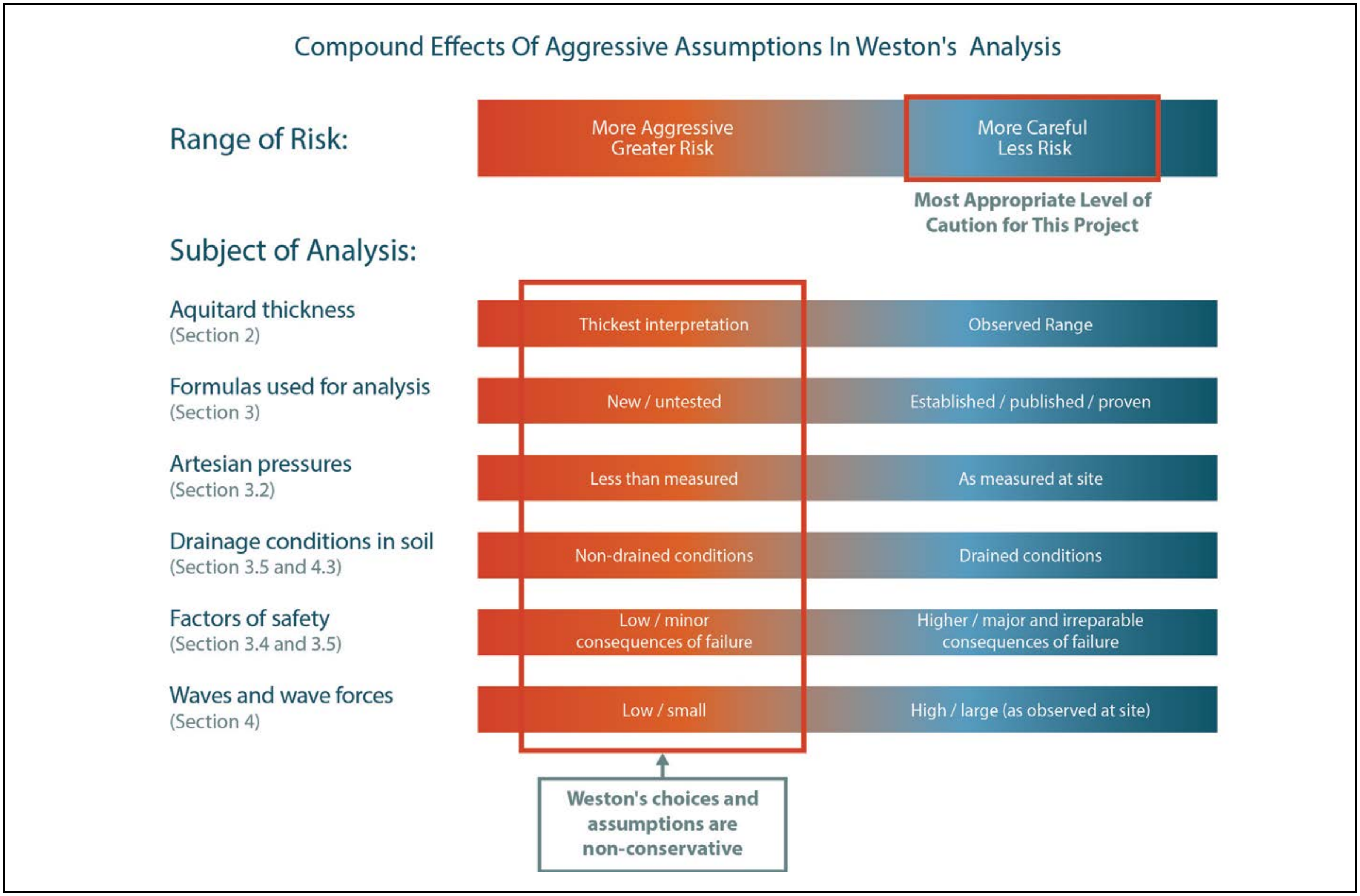
FIGURE



ANCHOR QEA

Figure 1
Compound Effects of Aggressive Assumptions in Weston's Analysis
Points Of Concern Regarding EPA 2014 Dry Dredging Evaluation
Ashland Lakefront Superfund Site

APPENDIX A
SHEET PILE ANALYSIS USING PROSHEET™

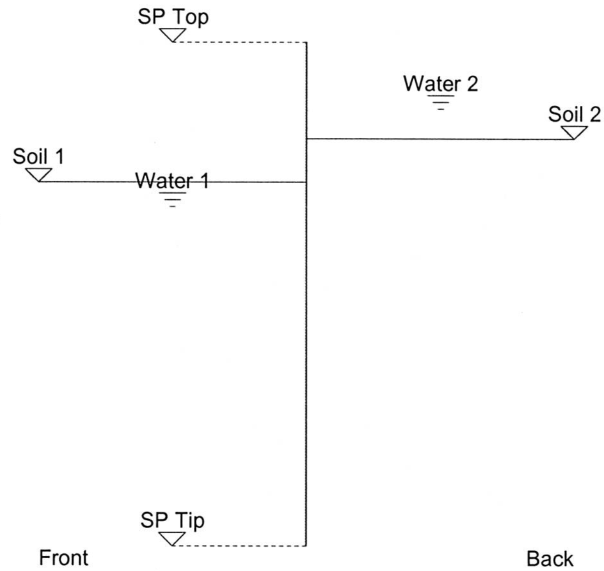
Sheet Pile Design According to Blum-Method

Project Name:	Weston Drained Both Sides AQ-SB-04
Date:	12/11/2014
Author:	JTaylor
Company:	Anchor QEA, LLC
Comment:	Trial run of Weston design using drained conditions on both sides.

AQ-SB-04
Drained Conditions

Geodata

	Unit
Sheet Pile Top Level [ft]	0.000
Sheet Pile Tip Level [ft]	93.932
Soil Level in Front [ft]	26.200
Soil Level behind [ft]	18.100
Anchorlevel [ft]	0.000
Water Level in Front [ft]	28.200
Water Level behind [ft]	10.000
Soil Surface Inclination in Front [Deg]	0.000
Soil Surface Inclination behind [Deg]	0.000
Caquot Surcharge in Front [kip/ft2]	0.000
Caquot Surcharge behind [kip/ft2]	0.000
Anchor Inclination [Deg]	0.000
Earth Support	Cantilever



Soil Layers

Layers in Front

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	27.400	0.135	0.073	1.706	25.000	12.500	0.000
Layer 2	29.600	0.124	0.062	1.706	25.000	12.500	0.000
Layer 3	43.000	0.135	0.073	1.706	25.000	12.500	0.000
Layer 4	55.700	0.117	0.054	1.643	23.000	11.500	0.000
Layer 5	69.000	0.123	0.061	1.643	23.000	11.500	0.000
Layer 6	75.800	0.129	0.067	1.643	23.000	11.500	0.000
Layer 7	81.100	0.135	0.073	1.706	25.000	12.500	0.000
Layer 8	100.000	0.136	0.074	2.201	40.000	20.000	0.000

Layers behind

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	24.200	0.115	0.053	0.440	20.000	10.000	0.000
Layer 2	27.400	0.135	0.073	0.359	25.000	12.500	0.000
Layer 3	29.600	0.124	0.062	0.359	25.000	12.500	0.000
Layer 4	43.000	0.135	0.073	0.359	25.000	12.500	0.000
Layer 5	55.700	0.117	0.054	0.389	23.000	11.500	0.000
Layer 6	69.000	0.123	0.061	0.389	23.000	11.500	0.000
Layer 7	75.800	0.129	0.067	0.389	23.000	11.500	0.000
Layer 8	81.100	0.135	0.073	0.359	25.000	12.500	0.000
Layer 9	100.000	0.136	0.074	0.187	40.000	20.000	0.000

Userdefined Pressures

	Pressure Top [kip/ft2]	Pressure Tip [kip/ft2]	Depth Top [ft]	Depth Tip [ft]
Strip 1	0.198	0.198	3.720	18.100
Strip 2	0.000	0.716	24.200	81.100

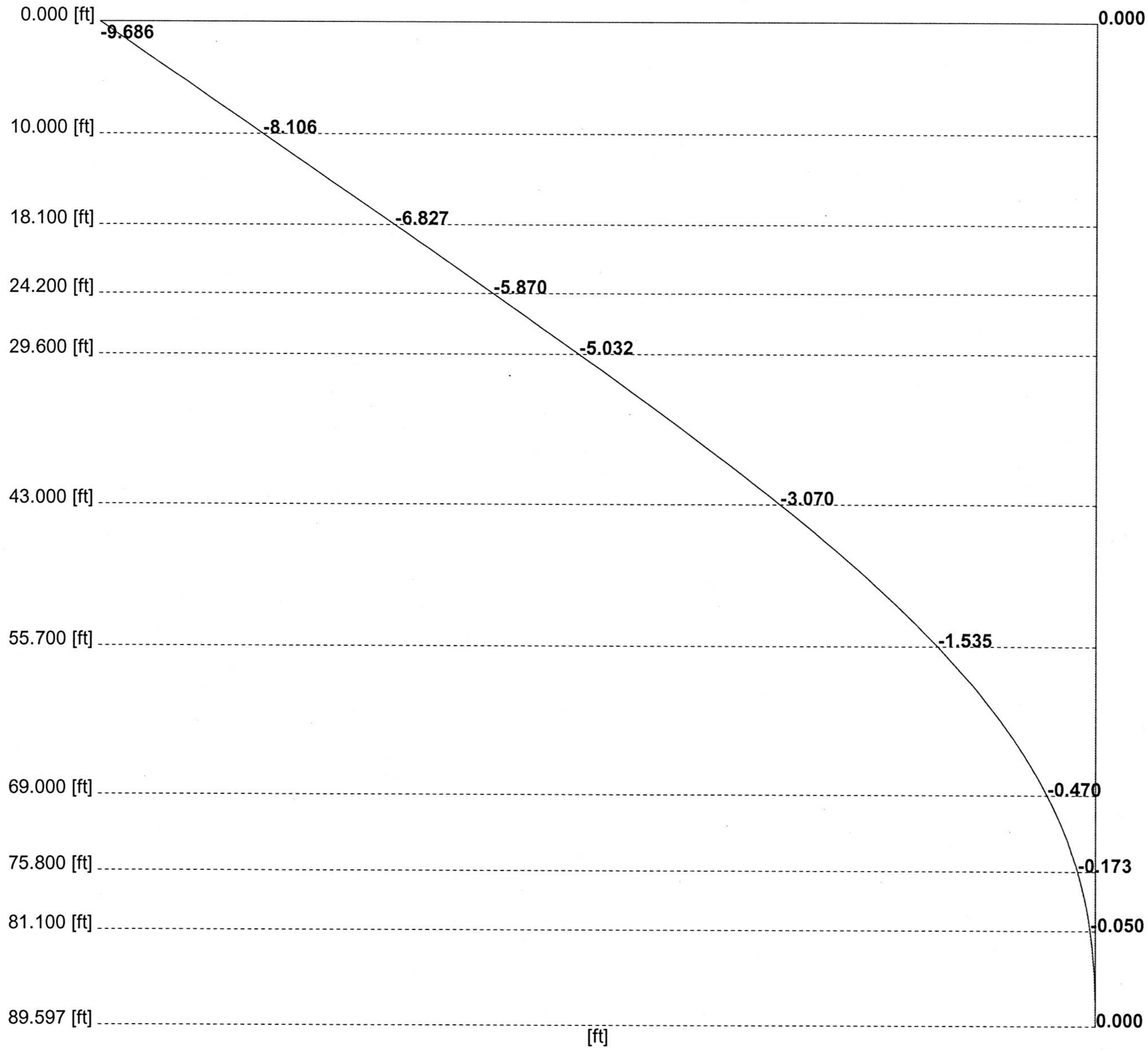
Pile Section

Name	AZ 50
Inertia [in4/ft]	886.494
Modulus [in3/ft]	93.279
Area [in2/ft]	15.222
Mass [lbs/ft2]	51.798
Steel Grade [lb/in2]	66692.078
Requested Safety	1.500

Pile Check

		Depth [ft]
Name	AZ 50	
Inertia [in4/ft]	886.494	
Modulus [in3/ft]	93.279	
Area [in2/ft]	15.222	
Mass [lbs/ft2]	51.798	
Steel Grade [lb/in2]	66692.078	
Minimal Moment [kipft/ft]	-4.915	89.630
Maxmimal Moment [kipft/ft]	690.645	66.953
Normal Forces at Max. Moment [kip/ft]	75.731	89.630
Normal Forces at Min. Moment [kip/ft]	26.708	66.953
Deflection at Min. Moment [ft]	0.000	89.630
Deflection at Max. Moment [ft]	-0.578	66.953
Min. Stress at Min. Moment [lb/in2]	4342.710	89.630
Max. Stress at Min. Moment [lb/in2]	5607.215	89.630
Min. Stress at Max. Moment [lb/in2]	-89076.313	66.953
Max. Stress at Max. Moment [lb/in2]	92585.273	66.953
Safety < Req. Safety = 1.500	0.720	
Sheet Pile Top Level [ft]	0.000	
Sheet Pile Tip Level [ft]	93.932	
Sheet Pile Length [ft]	93.932	
Included OverLength [ft]	4.335	
Vertical Equilibrium [kip/ft]	107.961	
Anchor Force (horiz.) [kip/ft]	0.000	

Deflection Diagram



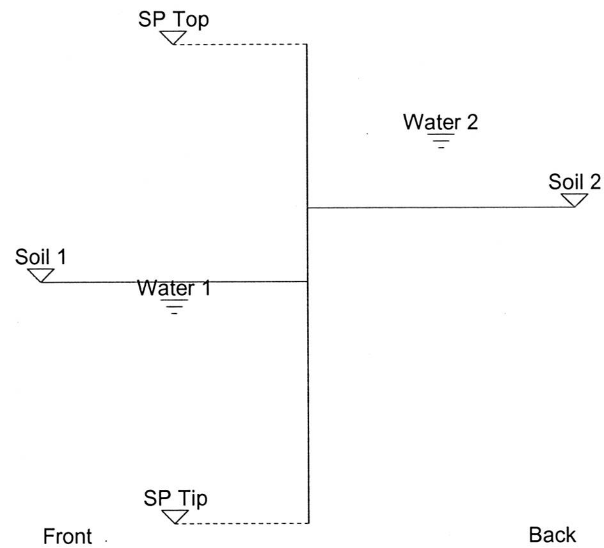
Sheet Pile Design According to Blum-Method

Project Name:	Weston Undrained Both Sides AQ-SB-04
Date:	12/11/2014
Author:	JTaylor
Company:	Anchor QEA, LLC
Comment:	Trial run of Weston design using undrained conditions on both sides.

AQ-SB-04
Undrained Conditions

Geodata

	Unit
Sheet Pile Top Level [ft]	0.000
Sheet Pile Tip Level [ft]	52.789
Soil Level in Front [ft]	26.200
Soil Level behind [ft]	18.100
Anchorlevel [ft]	0.000
Water Level in Front [ft]	28.200
Water Level behind [ft]	10.000
Soil Surface Inclination in Front [Deg]	0.000
Soil Surface Inclination behind [Deg]	0.000
Caquot Surcharge in Front [kip/ft2]	0.000
Caquot Surcharge behind [kip/ft2]	0.000
Anchor Inclination [Deg]	0.000
Earth Support	Cantilever



Soil Layers

Layers in Front

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	27.400	0.123	0.060	1.000	0.000	0.000	3.962
Layer 2	29.600	0.112	0.049	8.808	38.000	-19.000	0.000
Layer 3	43.000	0.123	0.060	1.000	0.000	0.000	1.318
Layer 4	55.700	0.105	0.042	1.000	0.000	0.000	0.570
Layer 5	69.000	0.111	0.048	1.000	0.000	0.000	0.752
Layer 6	75.800	0.117	0.054	1.000	0.000	0.000	0.944
Layer 7	81.100	0.123	0.060	1.000	0.000	0.000	2.243
Layer 8	100.000	0.136	0.074	10.400	40.000	-20.000	0.000

Layers behind

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	24.200	0.115	0.053	0.440	20.000	10.000	0.000
Layer 2	27.400	0.123	0.060	1.000	0.000	0.000	3.962
Layer 3	29.600	0.112	0.049	8.808	38.000	-19.000	0.000
Layer 4	43.000	0.123	0.060	1.000	0.000	0.000	1.318
Layer 5	55.700	0.105	0.042	1.000	0.000	0.000	0.570
Layer 6	69.000	0.111	0.048	1.000	0.000	0.000	0.752
Layer 7	75.800	0.117	0.054	1.000	0.000	0.000	0.944
Layer 8	81.100	0.123	0.060	1.000	0.000	0.000	2.243
Layer 9	100.000	0.136	0.074	10.400	40.000	-20.000	0.000

Userdefined Pressures

	Pressure Top [kip/ft2]	Pressure Tip [kip/ft2]	Depth Top [ft]	Depth Tip [ft]
Strip 1	0.198	0.198	3.720	18.100
Strip 2	0.000	0.716	24.200	81.100

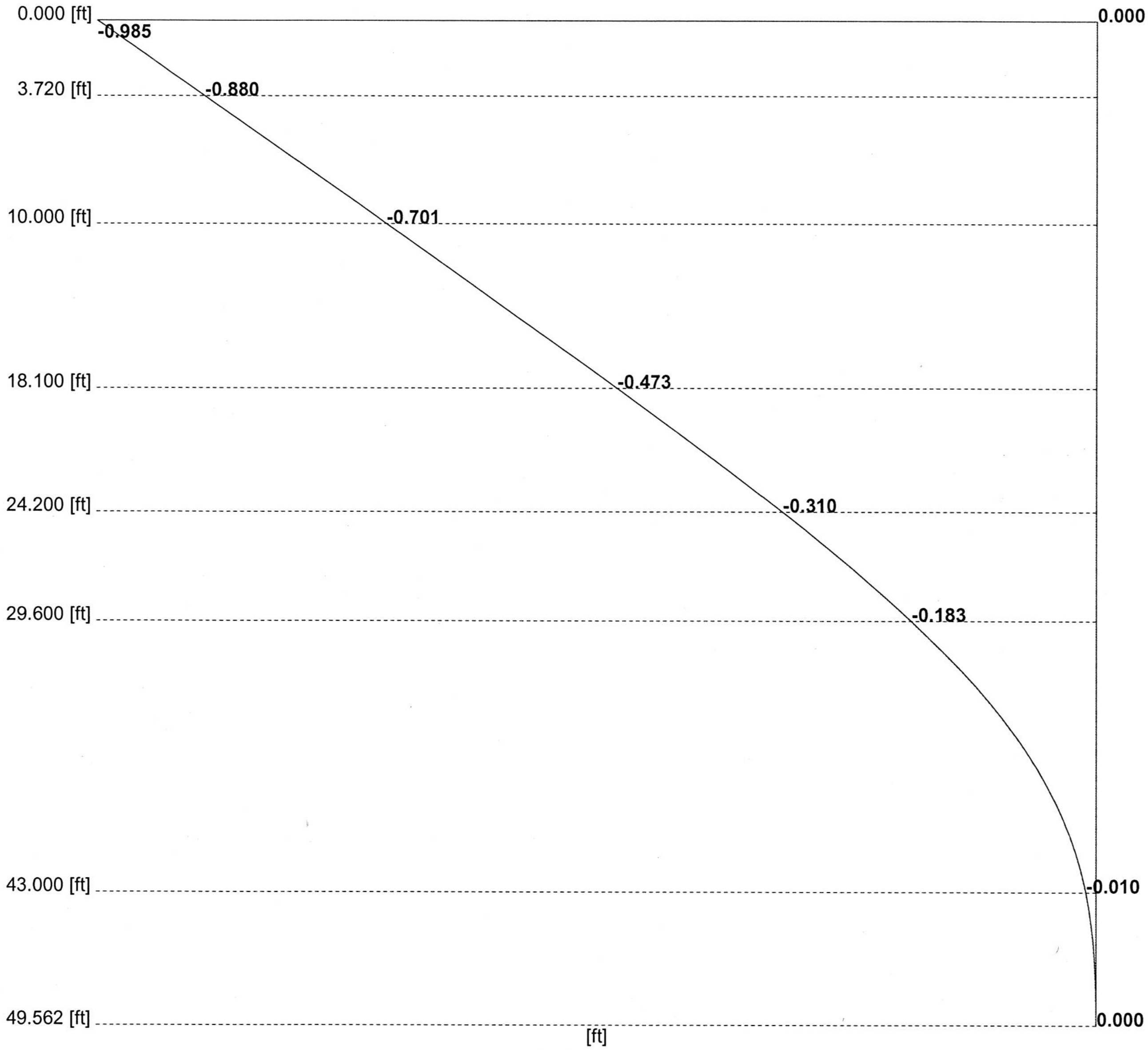
Pile Section

Name	PU 32
Inertia [in4/ft]	529.582
Modulus [in3/ft]	59.520
Area [in2/ft]	11.433
Mass [lbs/ft2]	38.956
Steel Grade [lb/in2]	50000.000
Requested Safety	1.500

Pile Check

		Depth [ft]
Name	PU 32	
Inertia [in4/ft]	529.582	
Modulus [in3/ft]	59.520	
Area [in2/ft]	11.433	
Mass [lbs/ft2]	38.956	
Steel Grade [lb/in2]	50000.000	
Minimal Moment [kipft/ft]	-0.609	49.595
Maxmimal Moment [kipft/ft]	158.747	36.162
Normal Forces at Max. Moment [kip/ft]	0.000	49.595
Normal Forces at Min. Moment [kip/ft]	-5.341	36.162
Deflection at Min. Moment [ft]	0.000	49.595
Deflection at Max. Moment [ft]	-0.068	36.162
Min. Stress at Min. Moment [lb/in2]	-122.751	49.595
Max. Stress at Min. Moment [lb/in2]	122.751	49.595
Min. Stress at Max. Moment [lb/in2]	-32398.316	36.162
Max. Stress at Max. Moment [lb/in2]	31464.064	36.162
Safety > Req. Safety = 1.500	1.543	
Sheet Pile Top Level [ft]	0.000	
Sheet Pile Tip Level [ft]	52.789	
Sheet Pile Length [ft]	52.789	
Included OverLength [ft]	3.227	
Vertical Equilibrium [kip/ft]	-5.341	
Anchor Force (horiz.) [kip/ft]	0.000	

Deflection Diagram



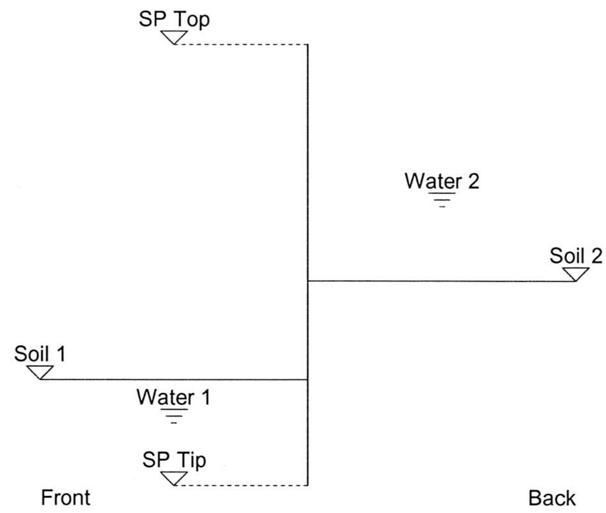
Sheet Pile Design According to Blum-Method

Project Name:	Weston Undrained Both Sides AQ-SB-02
Date:	12/11/2014
Author:	JTaylor
Company:	Anchor QEA, LLC
Comment:	Trial run of Weston design using undrained conditions on both sides.

AQ-SB-02
UnDrained conditions

Geodata

	Unit
Sheet Pile Top Level [ft]	0.000
Sheet Pile Tip Level [ft]	29.587
Soil Level in Front [ft]	22.500
Soil Level behind [ft]	15.900
Anchorlevel [ft]	0.000
Water Level in Front [ft]	24.500
Water Level behind [ft]	10.000
Soil Surface Inclination in Front [Deg]	0.000
Soil Surface Inclination behind [Deg]	0.000
Caquot Surcharge in Front [kip/ft2]	0.000
Caquot Surcharge behind [kip/ft2]	0.000
Anchor Inclination [Deg]	0.000
Earth Support	Cantilever



Soil Layers

Layers in Front

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	28.500	0.118	0.056	1.000	0.000	0.000	3.588
Layer 2	30.000	0.117	0.055	1.000	0.000	0.000	4.607
Layer 3	36.750	0.099	0.037	1.000	0.000	0.000	0.820
Layer 4	44.750	0.105	0.043	1.000	0.000	0.000	0.790
Layer 5	47.250	0.113	0.051	1.000	0.000	0.000	1.125
Layer 6	49.750	0.118	0.056	1.000	0.000	0.000	3.600
Layer 7	57.900	0.118	0.056	1.000	0.000	0.000	3.620
Layer 8	90.000	0.136	0.074	4.600	40.000	0.000	0.000

Layers behind

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	20.500	0.115	0.053	1.000	0.000	0.000	3.588
Layer 2	28.500	0.118	0.056	1.000	0.000	0.000	3.588
Layer 3	30.000	0.117	0.055	1.000	0.000	0.000	4.607
Layer 4	36.750	0.099	0.037	1.000	0.000	0.000	0.820
Layer 5	44.750	0.105	0.043	1.000	0.000	0.000	0.790
Layer 6	47.250	0.113	0.051	1.000	0.000	0.000	1.125
Layer 7	49.750	0.118	0.056	1.000	0.000	0.000	3.600
Layer 8	57.900	0.118	0.056	1.000	0.000	0.000	3.620
Layer 9	90.000	0.136	0.074	0.217	40.000	0.000	0.000

Userdefined Pressures

	Pressure Top [kip/ft2]	Pressure Tip [kip/ft2]	Depth Top [ft]	Depth Tip [ft]
Strip 1	0.211	0.211	2.680	15.900
Strip 2	0.000	0.694	20.500	57.900

Pile Section

Name	AZ 26-700
Inertia [in4/ft]	437.316
Modulus [in3/ft]	48.360
Area [in2/ft]	8.844
Mass [lbs/ft2]	30.087
Steel Grade [lb/in2]	50000.000
Requested Safety	1.500

Pile Check

		Depth [ft]
Name	AZ 26-700	
Inertia [in4/ft]	437.316	
Modulus [in3/ft]	48.360	
Area [in2/ft]	8.844	
Mass [lbs/ft2]	30.087	
Steel Grade [lb/in2]	50000.000	
Minimal Moment [kipft/ft]	-1.397	28.175
Maxmimal Moment [kipft/ft]	61.846	23.320
Normal Forces at Max. Moment [kip/ft]	0.000	28.175
Normal Forces at Min. Moment [kip/ft]	0.000	23.320
Deflection at Min. Moment [ft]	0.000	28.175
Deflection at Max. Moment [ft]	-0.006	23.320
Min. Stress at Min. Moment [lb/in2]	-346.532	28.175
Max. Stress at Min. Moment [lb/in2]	346.532	28.175
Min. Stress at Max. Moment [lb/in2]	-15345.883	23.320
Max. Stress at Max. Moment [lb/in2]	15345.883	23.320
Safety > Req. Safety = 1.500	3.258	
Sheet Pile Top Level [ft]	0.000	
Sheet Pile Tip Level [ft]	29.587	
Sheet Pile Length [ft]	29.587	
Included OverLength [ft]	1.445	
Vertical Equilibrium [kip/ft]	0.000	
Anchor Force (horiz.) [kip/ft]	0.000	

Deflection Diagram