New York State Department of Environmental Conservation
Division of Environmental Remediation
Remedial Bureau D, 12th Floor
625 Broadway, Albany, New York 12233-7013
Phone: (518) 402-9676 • **Fax:** (518) 402-9020
Website: www.dec.ny.gov

Joe Martens
Commissioner

August 1, 2014

Mr. John P. McAuliffe, P.E.
Program Director, Syracuse
Honeywell
301 Plainfield Road, Suite 330
Syracuse, NY 13212

Re: Onondaga Lake Technical Support Document for Remediation Area E Shoreline, Dated July 2014

Dear Mr. McAuliffe:

We have received and reviewed the above-referenced document, a copy of which was attached to Edward Glaza's August 1, 2014 email to my attention, and find that the document has addressed our previous comments. Therefore, the Onondaga Lake Technical Support Document for Remediation Area E Shoreline, dated July 2014, is hereby approved. Please see that copies of the final document, including this approval letter, are sent to the document repositories selected for this site.

Sincerely,

Timothy J. Larson, P.E.
Project Manager

ec: B. Israel, Esq, - Arnold & Porter
J. Davis - NYSDOL, Albany
M. Schuck - NYSDOH, Albany
D. Babcock - Parsons
R. Nunes - USEPA, NYC
M. Sergott - NYSDOH, Albany
E. Glaza – Parsons
M. McDonald - Honeywell

ONONDAGA LAKE REMEDIATION AREA E SHORELINE TECHNICAL SUPPORT DOCUMENT

Prepared For:

Honeywell

301 Plainfield Road, Suite 330
Syracuse, NY 13212

Prepared By:

PARSONS

301 Plainfield Road, Suite 350
Syracuse, NY 13212

and

Geosyntec consultants

JULY 2014

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ONONDAGA LAKE REMEDIATION AREA E SHORELINE TECHNICAL SUPPORT DOCUMENT

1.0 INTRODUCTION

NYSDEC[1] and USEPA[2] issued a Record of Decision (ROD) in July 2005 that selected a remedy for the Onondaga Lake Bottom Subsite of the Onondaga Lake Superfund Site (NYSDEC and USEPA, 2005). This included establishment of sediment cleanup criteria. These criteria and the results of extensive sediment sampling completed prior and subsequent to the ROD were used to define the area of the lake requiring sediment dredging and/or capping. This included an area in the southeastern part of the lake referred to as Remediation Area E (RA-E), as shown in Figure 1.

Dredging was incorporated into the remedy for this portion of the lake (SMU 6 and eastern portion of SMU 7) to allow placement of the cap without loss of lake surface area and to achieve post-capping elevations that are consistent with the habitat goals detailed in the ROD. However, unlike the In-Lake Waste Deposit (ILWD) in SMU 1 and portions of SMUs 2 and 7, dredging was not required in RA-E to remove contaminant mass or reduce sediment contaminant concentrations prior to capping based on lower contaminant levels.

The remedy called for in the ROD for this area, including both dredging and capping, cannot be carried out in the area adjacent to the shoreline from east of Harbor Brook to just north of Onondaga Creek, as shown in Figure 1. Three active rail lines are located immediately adjacent to part of the RA-E shoreline (Figure 2). Detailed geotechnical analysis (Appendix A) indicates that dredging along this shoreline could result in shoreline and rail line instability, which could cause movement of the rail lines. Therefore, a buffer zone where no dredging will occur has been established to prevent shoreline and rail line instability. This buffer zone extends to approximately 130 to 200 ft from the shoreline and impacts an area of approximately 10 acres (approximately 2% of the total area dredged and/or capped as part of the overall remedy). The water in this area is relatively shallow, ranging from 0 to 3 ft (Figure 3). The shallow nature of this area combined with the high wind/wave energy levels inhibits productive habitat conditions. Placement of a sediment cap without dredging would result in loss of lake surface, which is contrary to ROD requirements. However, since the levels of contamination in this area are relatively low, implementing a revised remedial approach that includes measures to improve habitat and promote natural recovery provides more environmental benefit than losing lake surface area through placement of a cap without prior dredging.

No unacceptable human health risks are presented by the relatively low contamination levels present in this area. Human health risks related to direct exposure to sediments in the southern basin nearshore area, including sediments in RA-E, through activities such as wading and swimming were evaluated as part of the baseline risk assessment. The results were found to be within the USEPA target risk range for cancer risks and below the target threshold for non-

1 NYSDEC - New York State Department of Environmental Conservation
2 USEPA – United States Environmental Protection Agency

cancer risks (NYSDEC and USEPA, 2005). Potential ecological risks for the lake are generally related to direct toxicity to organisms (such as aquatic insects) living within the sediment and bioaccumulation of contaminants (including mercury) into fish. Contaminant concentrations in some sample locations in the buffer zone exceeded the mean probable effects concentration quotient (mean PECQ) of 1, which is the threshold used to delineate potential direct toxicity. However, the relatively small size of this area would not substantively affect populations. The mean PECQ in surface samples in this area ranges from 0.4 to 27. The elevated mean PECQ in this area is primarily driven by heavy polyaromatic hydrocarbons (PAHs), which are not very mobile (compared to volatile organic contaminants, chlorinated benzenes and naphthalene) in sediment porewater and therefore have less potential to migrate to clean material which may accumulate above the existing sediment surface. Among the 17 surface sediment samples within the area, eight exceeded the site-specific mercury probable effect concentration (PEC) of 2.2 mg/kg; at only two of the eight locations was the mercury PEC exceeded by more than a factor of two. The bioaccumulation-based sediment quality value (BSQV) of 0.8 mg/kg total mercury would continue to be achieved in the "South Corner area" (relevant BSQV area defined in the Final Design, Appendix N, 2012). PCBs were generally not detected at elevated concentrations in the samples collected from this area, and therefore this area does not present a lake-wide bioaccumulation concern for PCBs.

2.0 SHORELINE REVISED REMEDIAL APPROACH

The revised approach incorporates measures to improve habitat and promote natural recovery of sediments in the RA-E area (Figure 3). The prevailing wind direction and the associated high wave action result in low vegetation density and habitat value. Aquatic plant coverage is sparse, and the number of fish nests documented is lower than almost any other shallow water area of the lake (Onondaga County, 2013). Therefore, a wave damper will be constructed along approximately 1000 feet of the buffer zone to reduce the wave energy along this shoreline (Figure 4). The wave damper will be a raised linear mound of cobble with a top width of approximately 10 ft. The design elevation for the top of the wave damper is 361.5 ft NAVD88, which is approximately 1 ft. below the lake design elevation of 362.5. This will provide significant wave energy reduction while still allowing free exchange of water and preventing stagnation of the area on the shoreward side of the wave damper.

The wave damper will reduce shoreline wave energy and thus allow for better growing conditions for aquatic vegetation. In addition, active planting of primarily emergent wetland species will be implemented in the areas behind the wave damper and behind areas with a shallow cap which would serve as a wave damper. The wave damper and increased vegetation will improve the area's habitat value for fish and other organisms. It will also help stabilize sediments and promote natural recovery through deposition and retention of new clean sediments such as those entering the lake from Onondaga Creek and/or resulting from decay of vegetation. Deposition rates are expected to be low.

No wave damper is included in the area in front of the Metro shoreline discharge pipe or in the areas immediately north and south of the discharge to avoid impeding effluent dispersion into the lake, and to prevent nutrient-rich water from negatively impacting water quality behind the wave damper. No wave damper is included at the southern end of the off-set area because the post-capping bathymetry in the Final Design already includes shallow water in this area to serve as a wave damper for the Wastebed B/Harbor Brook outboard area wetland.

The wave damper will be constructed of natural materials (cobble), consistent with the habitat/erosion protection substrate in this area. Baseline and long-term surface sediment sampling will be implemented to monitor conditions over time. Dredging and capping will be performed at the location of the wave dampers prior to placement of the additional cobbles for the wave damper, such that full dredging and capping will be implemented immediately outboard of the wave dampers. Additional details on the design of the wave damper will be provided in an addendum to the final design.

The remedial program in this area will also include the following.

- Baseline surface sediment sampling at approximately the same density as sampled during the pre-design investigation for the full list of mean PECQ parameters plus benzene, toluene and phenol; total organic carbon (TOC); and grain size.
- Characterization of existing substrate, structure, and vegetation. The vegetation present will be surveyed during baseline sampling to gain a better understanding of the need for additional plantings/types of plantings as well as the presence of invasive species. Additional information on success criteria and the potential need for adaptive management regarding the plantings will be submitted as part of the forthcoming monitoring plan.
- Post-remedy surface sediment sampling and vegetation monitoring at/near baseline locations (frequency to be determined) to confirm natural recovery and restoration success.

Details pertaining to monitoring and maintenance activities in this area, including the wave damper, will be included in the Onondaga Lake Monitoring and Maintenance Plan.

3.0 CONSIDERED ALTERNATIVES

In addition to the recommended revised approach described above, the following alternatives were evaluated in detail for this area:

- Modified cap
- Sequential dredging/capping
- Sheetpile to improve stability
- Temporary surcharge
- *In situ* treatment

A description of each of these alternatives and the basis for determining that they were not appropriate are provided below.

Modified Cap. Placement of a thinner cap without prior dredging would not be an appropriate approach because it would result in loss of lake surface. The ROD-specified cap includes a chemical isolation layer with a minimum thickness of one foot overlain by a habitat layer with a minimum thickness of one foot. Allowances must also be made for mixing with underlying sediment and the overplacement that result during construction, as shown in Figure 5. This results in an average cap thickness of approximately 3 ft. Thinner caps were considered for this area. The minimum constructible cap that would provide chemical isolation and erosion protection would be approximately 17 inches thick, as shown in Figure 5. However, the water depth in this area remains shallow for a significant distance from the shore, and placement of a 17-inch thick cap would result in a loss of lake surface area of approximately five acres.

Sequential Dredging/Capping. Sequential dredging and capping would not be an acceptable approach because it could result in shoreline and rail line instability. Sequential dredging and capping would involve dredging a portion of the shoreline area and then capping this dredged area before proceeding with dredging and capping the adjacent area. The entire shoreline area would ultimately be dredged and capped through a series of such steps. This would reduce the potential impacts to the shoreline and rail stability, but it would not eliminate them. Geotechnical analysis (Appendix A) indicated that any activity that reduces the shoreline stability from its current state would not be acceptable.

Sheetpiling to Improve Stability. Driving sheetpile along the shoreline prior to dredging was evaluated as a potential method of improving stability in this area. Two options were considered, neither of which would be an appropriate approach based on multiple considerations. The first option consisted of a continuous sheetpile along the length of the shoreline (Figure 6). The second option consisted of a series of smaller cells along the shoreline totally enclosed by sheetpiles (Figure 7). There is significant debris and rip-rap along the shoreline that would have to be removed through excavation prior to driving the sheetpile, as shown in Figure 8. Excavation of debris is consistent with the required installation methods for other sheetpile installations along the shoreline, such as the sheetpile installed as part of the Willis/Semet and Wastebed B/Harbor Brook barrier walls. However, as discussed in Section 2, any excavation in this area would unacceptably reduce shoreline and railroad stability; therefore, installation of a shoreline sheetpile wall or a series of sheetpile cells would not be an acceptable approach.

Installing the barrier wall approximately 30 feet from the shoreline was also evaluated under the assumption that there would be less debris to manage. However, geotechnical analysis indicated that, even if the barrier wall were driven to 80 feet deep, dredging outboard of the barrier wall would result in unacceptable movement of the railroad tracks (Appendix A). Construction of a 30-foot-wide soil buttress on the lake side of the wall would be required to prevent unacceptable deflection of the wall during dredging. This 30-foot-wide buttress would result in permanent loss of lake surface area, as shown in Figure 9. Therefore, installation of a sheetpile wall offset from the shoreline would not be an acceptable approach.

Temporary Surcharge. Placement of a temporary surcharge would not be an appropriate approach based on multiple considerations. This approach would involve placing the cap without prior dredging along the shoreline area adjacent to the rail lines, and then covering the cap with a large temporary soil pile (berm), as shown in Figure 10. Over time, the weight of the soil would result in compression of the sediments underlying the cap by an amount equal to the cap thickness, thereby lowering the cap surface. The goal would be to create enough settlement so that when the temporary soil pile was removed, the cap surface would be below the lake surface so that there was no loss of lake surface area.

The average cap thickness based on ROD-specified minimums plus average overplacements is approximately 3 ft. Geotechnical analysis indicated that placement of a 10-foot-tall temporary soil pile over the impacted area along the entire shoreline would result in a settlement of approximately 3 ft within approximately five years (Appendix A). However, this would result in a loss of lake surface area during the 5-year settlement period. It would also present a significant negative visual impact due to the length and height of the soil pile. Most significantly, detailed geotechnical modeling indicates it could also result in unacceptable settlement of the adjacent rail lines.

In situ Treatment. *In situ* treatment would consist of applying a treatment media to the surface of the sediment. The media would either be actively mixed or allowed to mix naturally with the upper layer of contaminated sediment, thereby reducing potential risks. Powdered activated carbon (PAC) and granular activated carbon (GAC) are the most widely used treatment media at other sediment sites and have the greatest potential for success in this area. Since activated carbon does not reduce the contaminant concentration within the sediment, it would not achieve the sediment criteria established in the ROD. However, the high sorptive capacity of activated carbon may reduce the contaminant concentration within the sediment porewater and thus may reduce contaminant bioavailability.

PAC and GAC are susceptible to disturbance and movement by currents or wave action because they are less dense then typical sediment particles. They are most commonly applied in relatively low energy environments, such as wetlands or in deep water. The shoreline area of RA-E is the highest energy shoreline within the lake. Even with the wave damper discussed in Section 1 in place, this area will still be subject to waves and ice scour that would periodically displace and move surface sediments and any activated carbon. In-situ treatment would therefore not provide long-term effectiveness in this area and would not be appropriate.

4.0 CONCLUSIONS

As discussed above, a capping and dredging offset has been established for the RA-E shoreline adjacent to the active railroad lines to ensure that the shoreline and rail stability is not impacted. A detailed evaluation of potential alternatives was completed, resulting in selection of a revised approach that incorporates measures to improve habitat and promote natural recovery of sediments in this area. The low concentrations of contaminants in this area present minimal risks if not actively remediated. The reduced wind/wave energy along the shoreline following construction of a wave damper and planting of emergent wetland species will allow habitat recovery and natural recovery of the sediments in the RA-E shoreline area. Details documenting the final design in this area will be provided in an addendum to Onondaga Lake Capping, Dredging, Habitat and Profundal Zone (SMU8) Final Design.

5.0 REFERENCES

New York State Department of Environmental Conservation and United States Environmental Protection Agency Region 2. 2005. *Record of Decision. Onondaga Lake Bottom Subsite of the Onondaga Lake Superfund Site*. July 2005.

Onondaga County. 2013. Ambient Monitoring Program: 2011. 2011 Annual Report Final, February 2013.

FIGURES

Remediation Area Boundary
Isolation Cap Area
SMU 8 Thin-layer Cap Area
Dredge Area
Sediment Management Unit (SMU) Boundary
Extent of ILWD in Littoral Zone
Willis/Semet IRM Barrier Wall
West Wall Portion of the WB-B/HB IRM
East Wall Portion of the WB-B/HB IRM
Eastern Shoreline Groundwater Collection Trench

N

New York State Digital Orthoimagery from 2009

650 325 0 650
Feet

FIGURE 1

Honeywell
Onondaga Lake
Syracuse, New York

Remediation Areas C, D, & E
Dredge & Cap Areas

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Date Revised: 1/31/2014 11:56:42 AM
User: 40189

FIGURE 2

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Onondaga Lake
Syracuse, New York

Shoreline Railroad Lines

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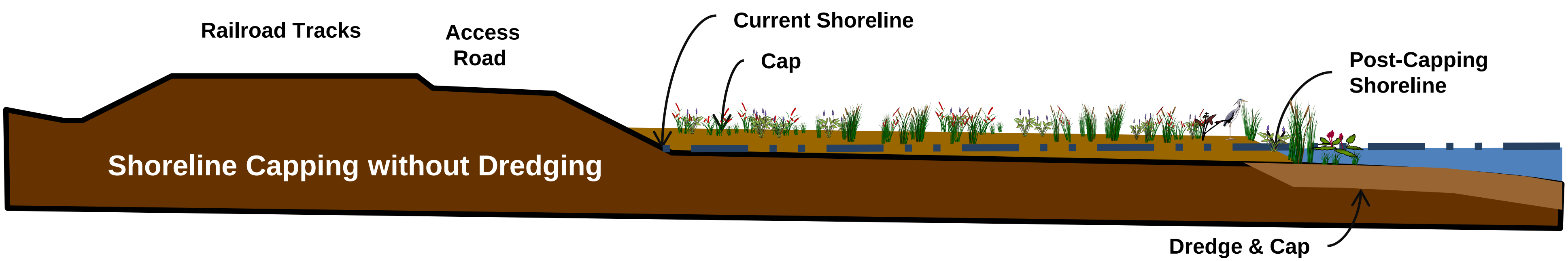

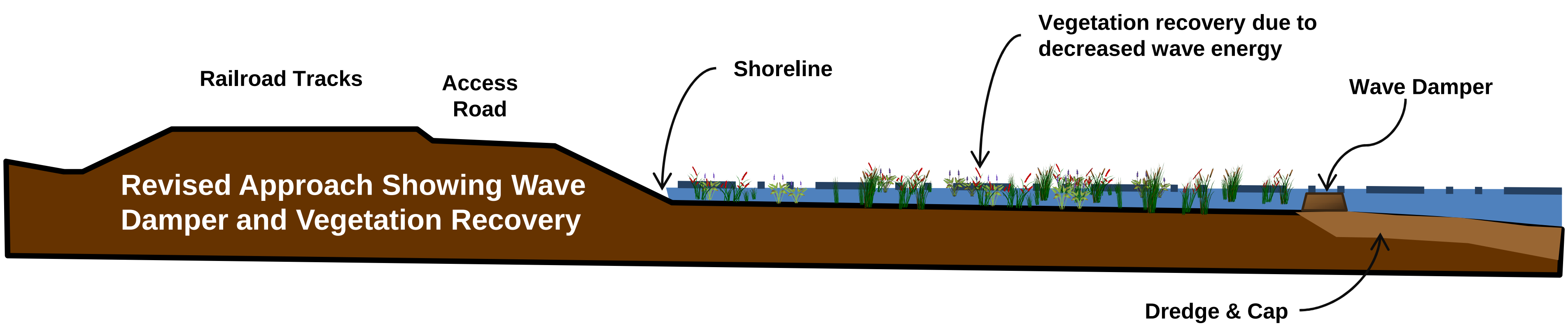

FIGURE 3

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Onondaga Lake
Syracuse, New York

Conceptual Cross-Sections

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FIGURE 4

Honeywell Onondaga Lake Syracuse, New York

Remediation Area E Shoreline Offset and Wave Damper

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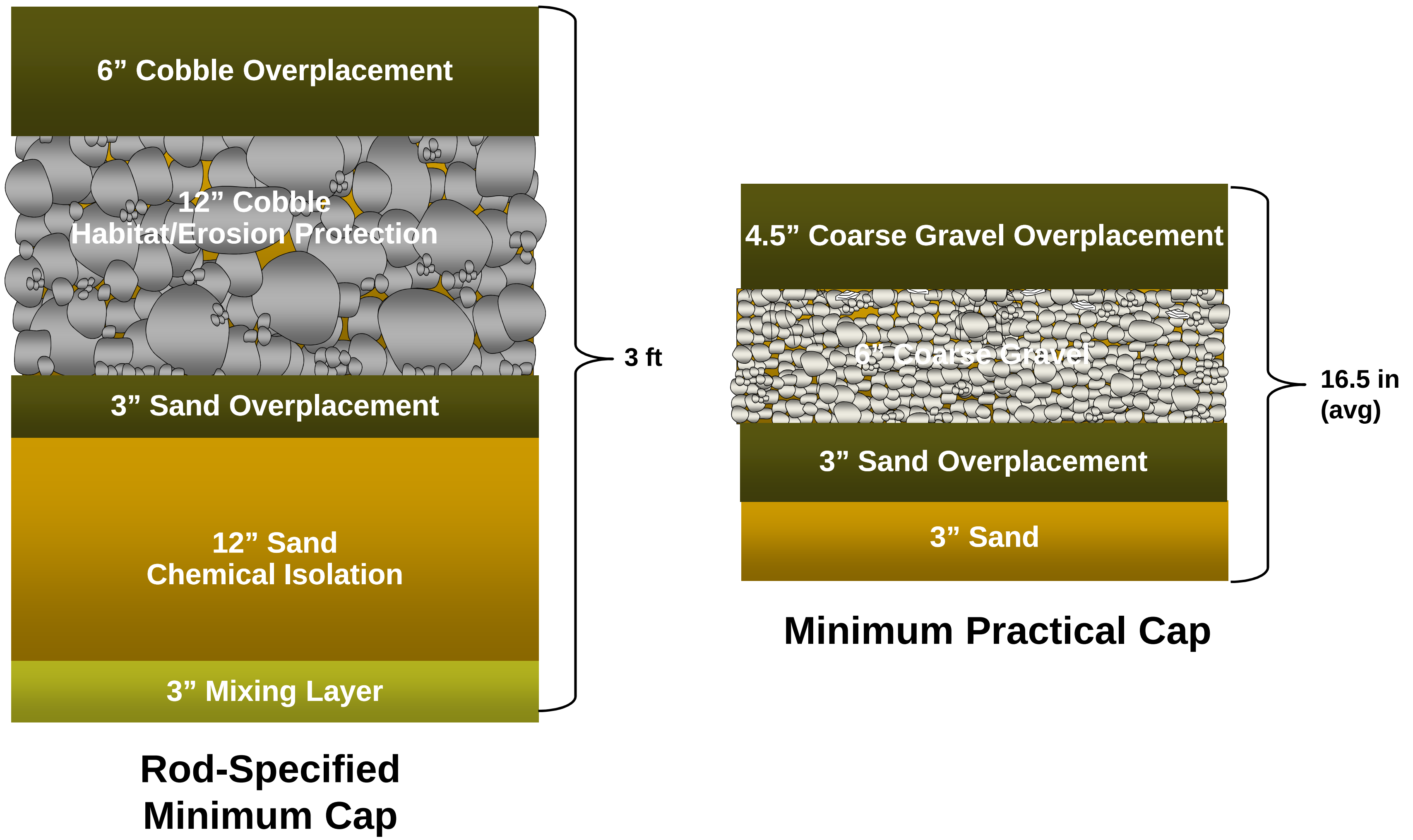

FIGURE 5

Honeywell

Onondaga Lake
Syracuse, New York

ROD-Specified Minimum Cap and Minimum Practical Cap

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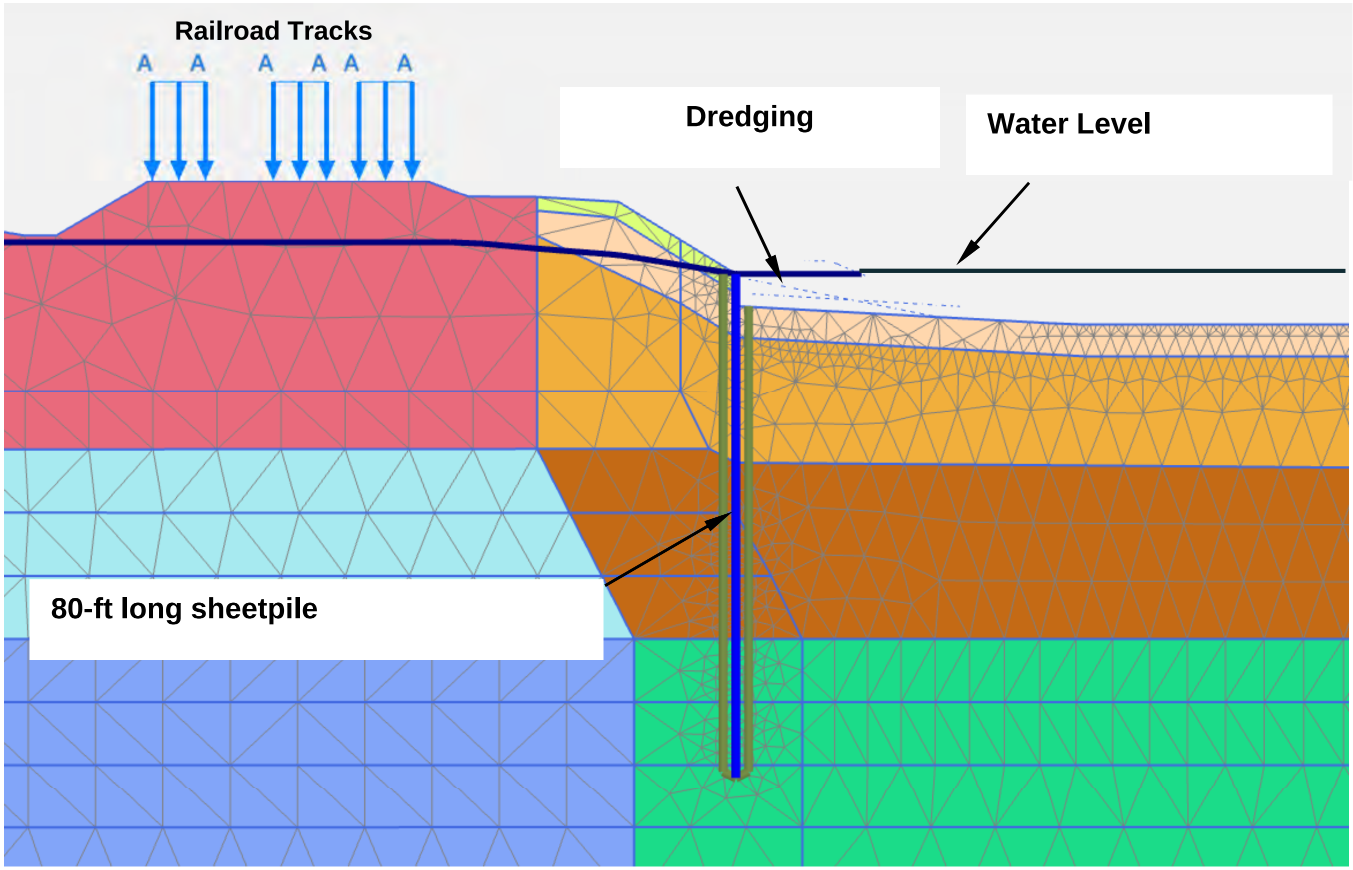

Finite Element Mesh (Cross Section)
Used for Sheetpile Stability Modeling

FIGURE 6

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Onondaga Lake
Syracuse, New York

Shoreline Sheetpile

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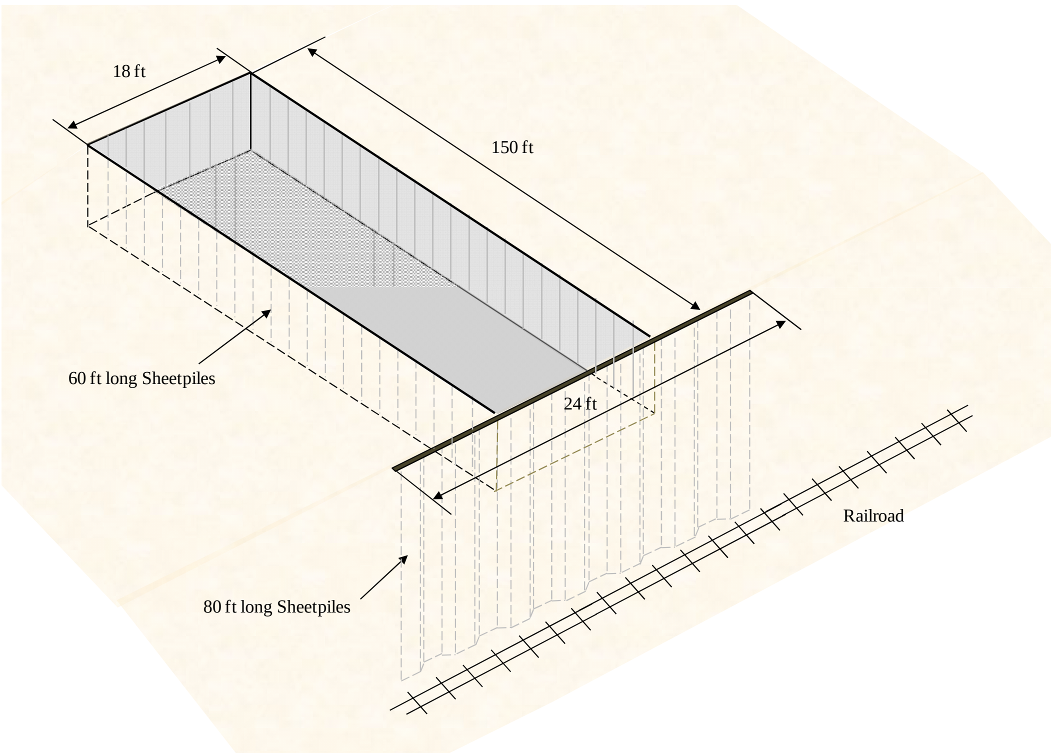

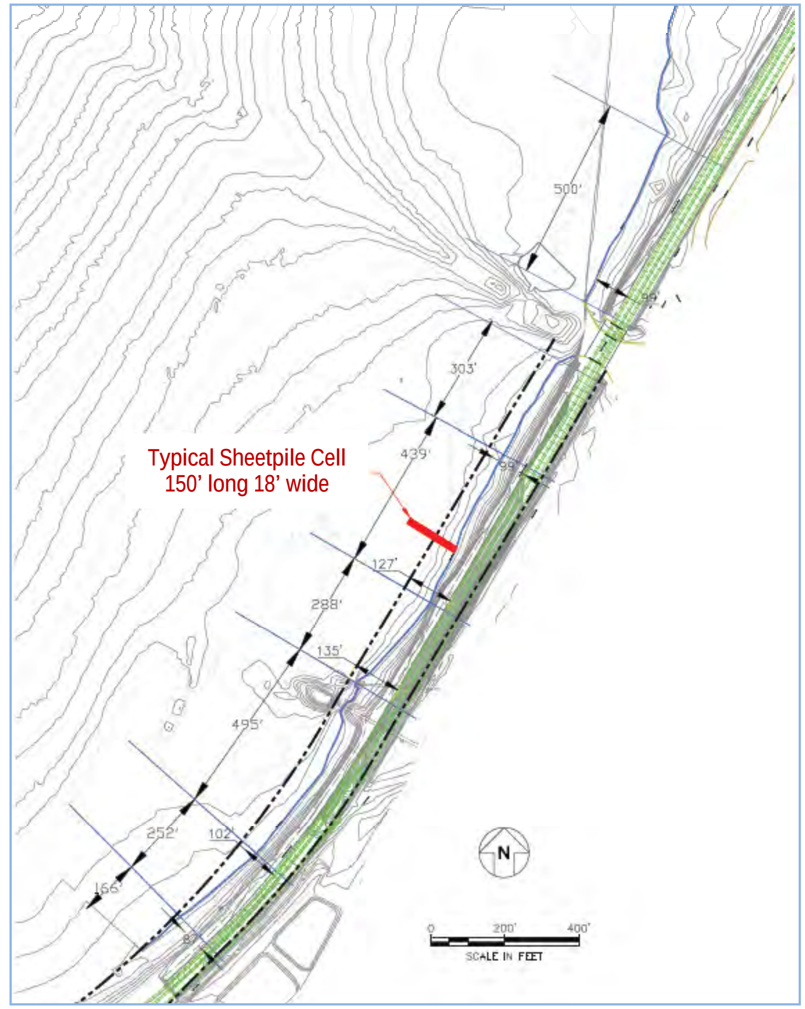

FIGURE 7

Honeywell

Onondaga Lake
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Sheetpile Cell Conceptual Design

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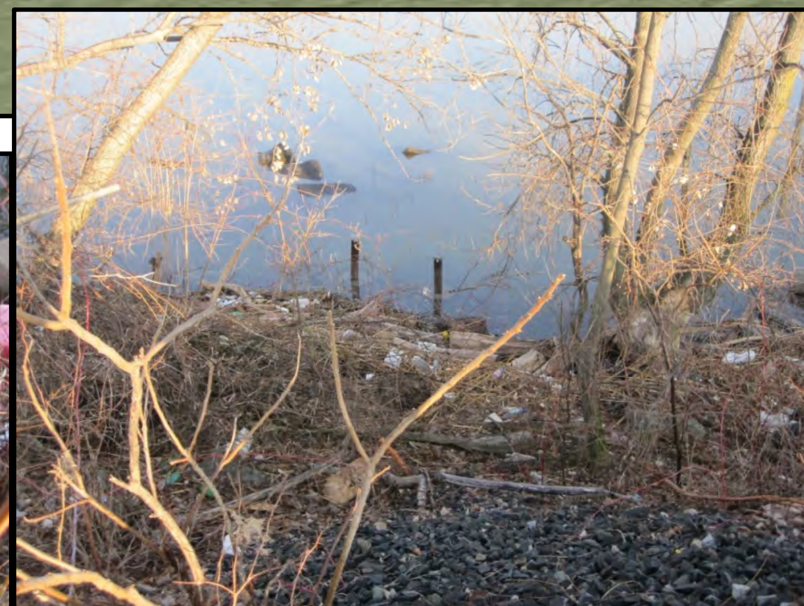

FIGURE 8

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Onondaga Lake
Syracuse, New York

Shoreline Debris

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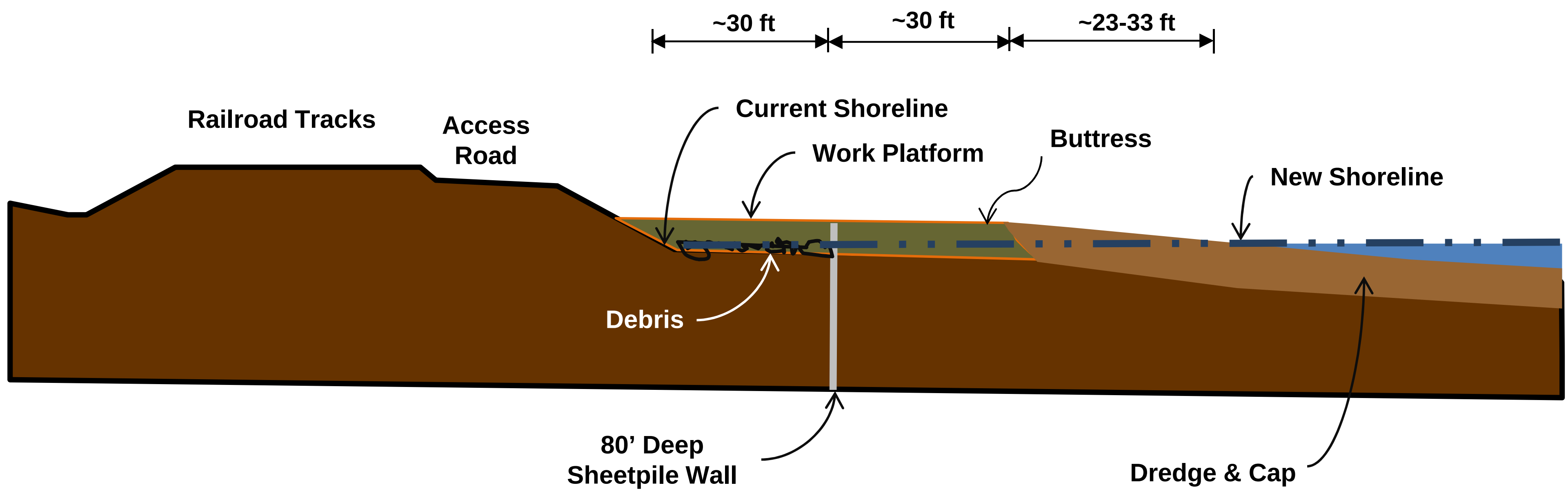

FIGURE 9

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Onondaga Lake
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Offshore Sheetpile – Cross-Section

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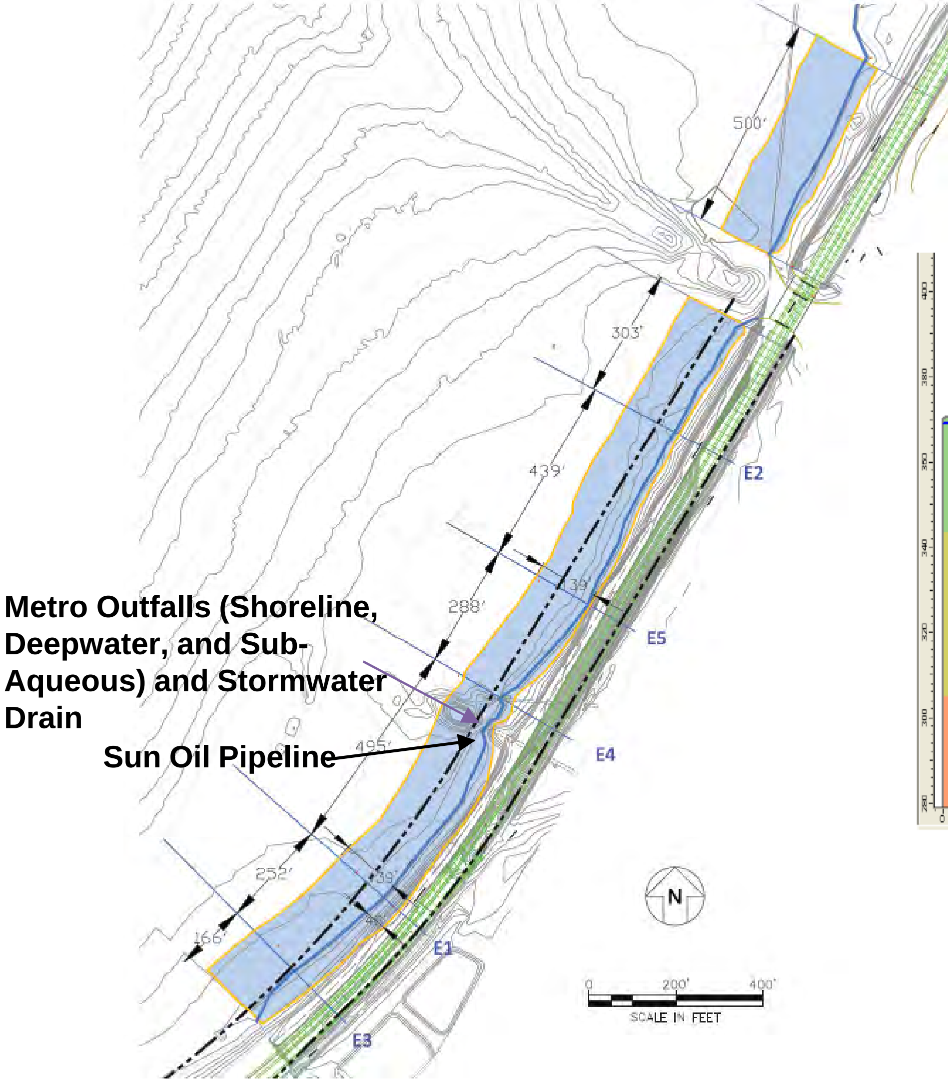

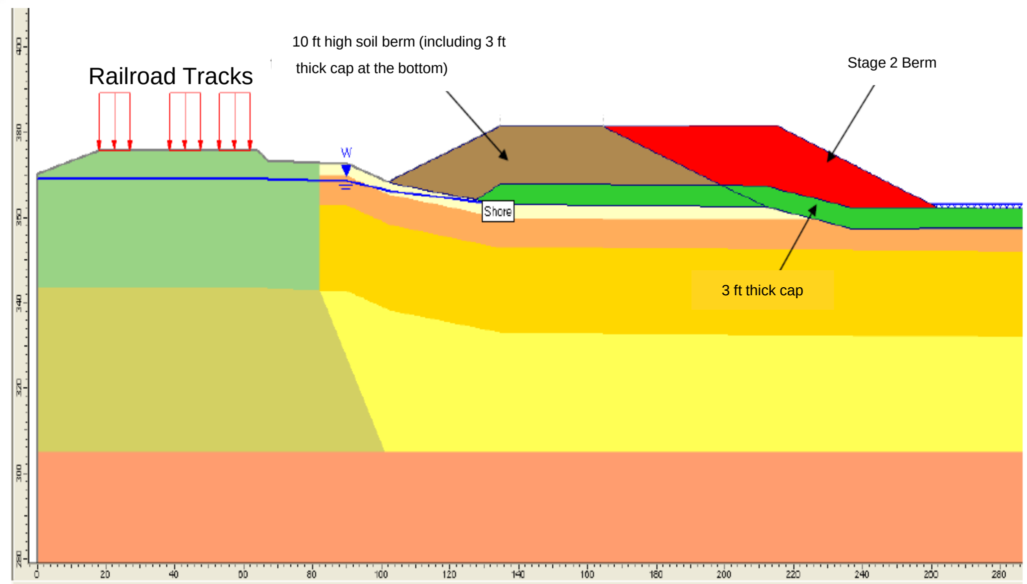

Uncertainty associated with potential settlement under the railroad tracks

Legend

- Surcharge Area
- Shoreline

FIGURE 10

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Onondaga Lake
Syracuse, New York

Surcharge Conceptual Design

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APPENDIX A

SHORELINE GEOTECHNICAL EVALUATION

Written by:	Mustafa Erten	Date:	4/18/2014	Reviewed by:	Ali Ebrahimi/Jay Beech	Date:	4/18/2014
Client:	Honeywell	Project:	RA-E Shoreline Offset Dredging	Project No.:	GD5453	Task No.:	03

SLOPE STABILITY ANALYSIS FOR RA-E SHORELINE DREDGING

INTRODUCTION

Three active rail lines are located immediately adjacent to a portion of the RA-E shoreline. Due to the shallow water in this area, placement of a sediment cap in this area without prior dredging would result in loss of lake surface area. A detailed geotechnical analysis was conducted to evaluate the railroad stability for the dredging and capping option which indicates that dredging along this shoreline could result in instability of the shoreline and movement of the rail lines. Therefore, an offset distance for dredging from the shoreline was developed, ranging from approximately 130 to 200 ft. The summary of geotechnical engineering analysis to calculate the offset distance is presented herein.

Specifically, the purpose of this package is to evaluate the slope stability of the RA-E shoreline along the CSX rail lines, which includes a bridge, and calculate an offset distance for the dredge and capping option in order to satisfy a minimum target factor of safety and maintain the existing stability condition of the rail lines. It should be noted that it was not the intention of the geotechnical analyses presented herein to find the actual factor of safety of the existing railroad or to evaluate whether the stability of the existing railroad is satisfactory. Instead, the analyses were performed to calculate the offset distance for dredging along the RA-E shoreline so that the calculated factor of safety with the offset option satisfies the selected target factor of safety with no reduction in the factor of safety for the existing condition of the rail lines.

SLOPE STABILITY ANALYSES

The slope stability analyses were performed using Spencer's method [Spencer, 1973], as implemented in the computer program SLIDE, version 6.025 [Rocscience, 2013]. Spencer's method, which satisfies vertical and horizontal force equilibrium and moment equilibrium, is considered to be more rigorous than other methods, such as the simplified Janbu method [Janbu, 1973] and the simplified Bishop method [Bishop, 1955].

The rotational mode (i.e., the circular slip surface mode) was considered in the analyses. The SLIDE program generated potential circular slip surfaces, calculated the factor of safety for each of these surfaces, and identified the most critical slip surface with the lowest factor of safety. Information required for the analyses included the slope geometry, the subsurface soil stratigraphy, the groundwater elevation, the external loading condition, and the properties of subsurface materials.

TARGET FACTOR OF SAFETY

The minimum required factor of safety was initially selected as 1.3 for the interim condition in accordance with the Engineering and Design Manual "*Slope Stability*" prepared by the U.S. Army Corps of Engineers [USACE, 2003]. However, for the purpose of the slope stability analyses presented in this package, the minimum required factor of safety was increased by 0.05 to account for potential seepage effect, which was evaluated by Geosyntec during the WB-B/HB barrier wall design. Therefore, the target factor of safety was considered to be 1.35 for the analyzed interim-undrained condition presented in this package.

EXTERNAL LOADING

A CSX railroad is in close proximity to the proposed dredge and capping area. Therefore, train loading was considered in the geotechnical slope stability analysis. The Coopers E80 live load model recommended by American Railway Engineering and Maintenance-of-Way Association (AREMA) [AREMA, 2009] was used to calculate the design railroad load. The configuration of the design railroad loading is shown in Figure 1.

SUBSURFACE STRATIGRAPHY

The information regarding the subsurface stratigraphy was obtained from the available soil borings, shown in Figure 2, adjacent to the selected cross sections for the slope stability analysis. The material properties used for the geotechnical slope stability analysis herein are presented in Table 1. In summary, the subsurface materials in the vicinity of the RA-E CSX shoreline consist of six strata: Existing Fill, Sediment, Railroad Foundation Soil, Marl, and Silt and Clay. The subsurface profiles at six cross sections selected for the CSX shoreline slope stability analyses are presented in Figures 3 through 9. The locations of these cross sections are shown in Figure 2.

ANALYZED CROSS SECTIONS

The six cross sections analyzed in this calculation package (i.e., Cross Sections 1 through 6) are shown in Figure 2. These cross sections were selected to represent anticipated critical conditions (from a stability viewpoint) and typical conditions (from a spatial coverage viewpoint) along the CSX shoreline. The anticipated critical cross sections were selected based on a combination of factors, including subsurface conditions, load conditions, and proximity to the railroad track.

Table1. Summary of Material Properties Used in Slope Stability Analysis

Material	Total Unit Weight (pcf)	Drained Shear Strength		Undrained Shear Strength (psf)
		c' (psf)	ϕ ' (degree)	
Existing Fill	92	N/A	N/A	145
Sediment	85	N/A	N/A	$\frac{s_u}{\sigma_v'} = 0.3$
Railroad Foundation Soil	120	0	35	N/A
Marl	97	N/A	N/A	160 for depth ≤ 10 ft 250, for 10 ft < depth < 30 ft 400, for depth ≥30 ft
Silt and Clay	110	N/A	N/A	$\frac{s_u}{\sigma_v'} = 0.3$
Marl (underneath railroad)	97	N/A	N/A	$\frac{s_u}{\sigma_v'} = 0.3$

ANALYSIS RESULTS

Slope Stability of Railroad

The slope stability of the existing railroad was first evaluated. The calculated factor of safety for the existing railroad condition at Cross Section 1 (factor of safety = 0.96) is shown in Figure 3. The soils directly under the rail lines have been surcharged by several feet of railroad ballast for many years. The soil strength gain due to the surcharge of the ballast was calculated and accounted for in the slope stability analysis. The analysis result indicates that the calculated most critical slip surface extends to the in-lake area. Dredging along the shoreline could result in reduction of the calculated factor of safety and therefore, shoreline and rail line instability, which could potentially cause movement of the rail lines.

Dredging Offset for RA-E Shoreline

Geotechnical slope stability analyses were performed for the interim-undrained condition after dredging with varying offset distances from the shoreline in order to meet the selected target factor of safety of 1.35. The dredge depth was assumed to be 3.5 ft (i.e., 3 ft of dredging plus 0.5 ft overdredge allowance) with a 5 Horizontal:1 Vertical (5H:1V) dredge slope in the analyses. Table 2 tabulates the slope stability analysis results. The selected minimum required offset distance varies between 130 ft and 200 ft depending on the surface geometry, subsurface stratigraphy, and proximity of the railroad tracks. Figures 4 through 9 show the slip surfaces and calculated factors of safety along each cross section with the selected minimum required offset distances. These offset distances were used to define the boundary of the no-dredging area shown in Figure 10.

Table 2. Summary of Slope Stability Analysis Results with Offset Dredging

Cross Section	**Offset Distance from Shoreline (ft)**	**Calculated Factor of Safety**	**Note**
E1	180	1.47	
	165	1.38	
	160	**1.34**	Results shown in Figure 4
E2	180	1.28	
	190	1.32	
	200	**1.36**	Results shown in Figure 5
E3	180	1.66	
	140	1.41	
	130	**1.36**	Results shown in Figure 6
E4	180	1.54	
	140	**1.34**	Results shown in Figure 7
E5	180	1.27	
	200	**1.35**	Results shown in Figure 8
E6	**180**	**1.34**	Results shown in Figure 9

Dredging Offset for CSX Bridge

Based on information provided by CSX Transportation (CSXT) to Honeywell, the bridge over the Onondaga Creek has 2 spans originally built in 1915. The bridge was raised in about 1922 to provide increased clearances. CSXT also provided copies of two letters dated July 9 and 16, 1923 that specially addressed track settlement and the original pile driving operation. CSXT stated in the letter that "CSXT has not experienced any settlement problems at this bridge since assuming responsibility for this rail line from the former Conrail" and requested that "Any dredging now planned in the canal or along the lake must address the possible effects of settlement on the bridge and the adjoining track structure".

The ability to dredge closer to the bridge in the vicinity of the shoreline was evaluated. The condition of the bridge is unknown and recent photographs of the bridge shows signs of deterioration. A review of the results in Table 2 shows that an offset of 180 ft from the shoreline is required to achieve an acceptable calculated factor of safety at Cross Section E6. This section is through the railroad embankment adjoining the bridge. Given the need to maintain stability and control settlement of the railroad track on either side of the bridge and the unknown condition of the bridge, it is recommended that the offset established for the railroad track along the shoreline be maintained in the vicinity of the bridge. The required offset of 180 ft from the shoreline equates to a required offset of 280 ft from the railroad embankment adjacent to the bridge foundation, as shown in Figure 10.

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Written by:	**Mustafa Erten**	Date:	**4/18/2014**	Reviewed by:	**Ali Ebrahimi/Jay Beech**	Date:	**4/18/2014**
Client:	**Honeywell**	Project:	**RA-E Shoreline Offset Dredging**	Project No.:	**GD5453**	Task No.:	**03**

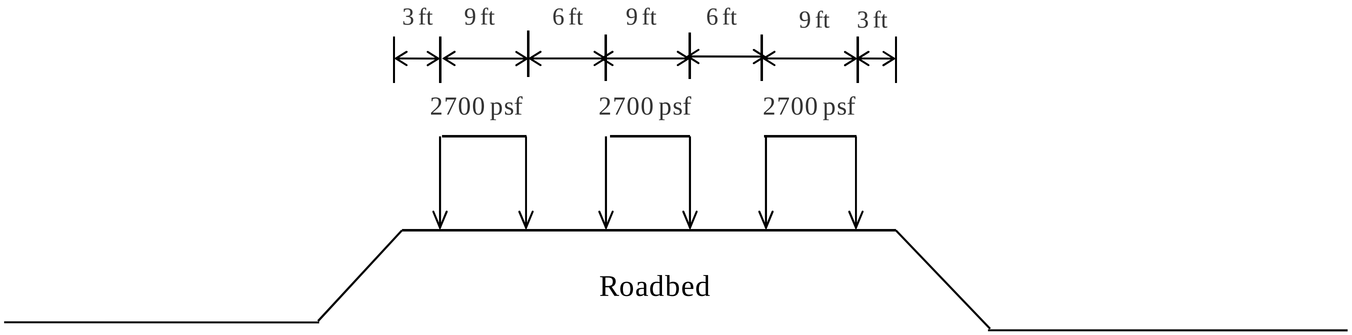

Figure 1. Configuration of Railroad Loading

Notes:

1. The Coopers E80 live load model recommended by American Railway Engineering and Maintenance-of-Way Association (AREMA) [AREMA, 2009] was used to calculate the design railroad load.
2. A dynamic load factor of 1.5 was used to account for dynamic effect caused by track and wheel irregularities.

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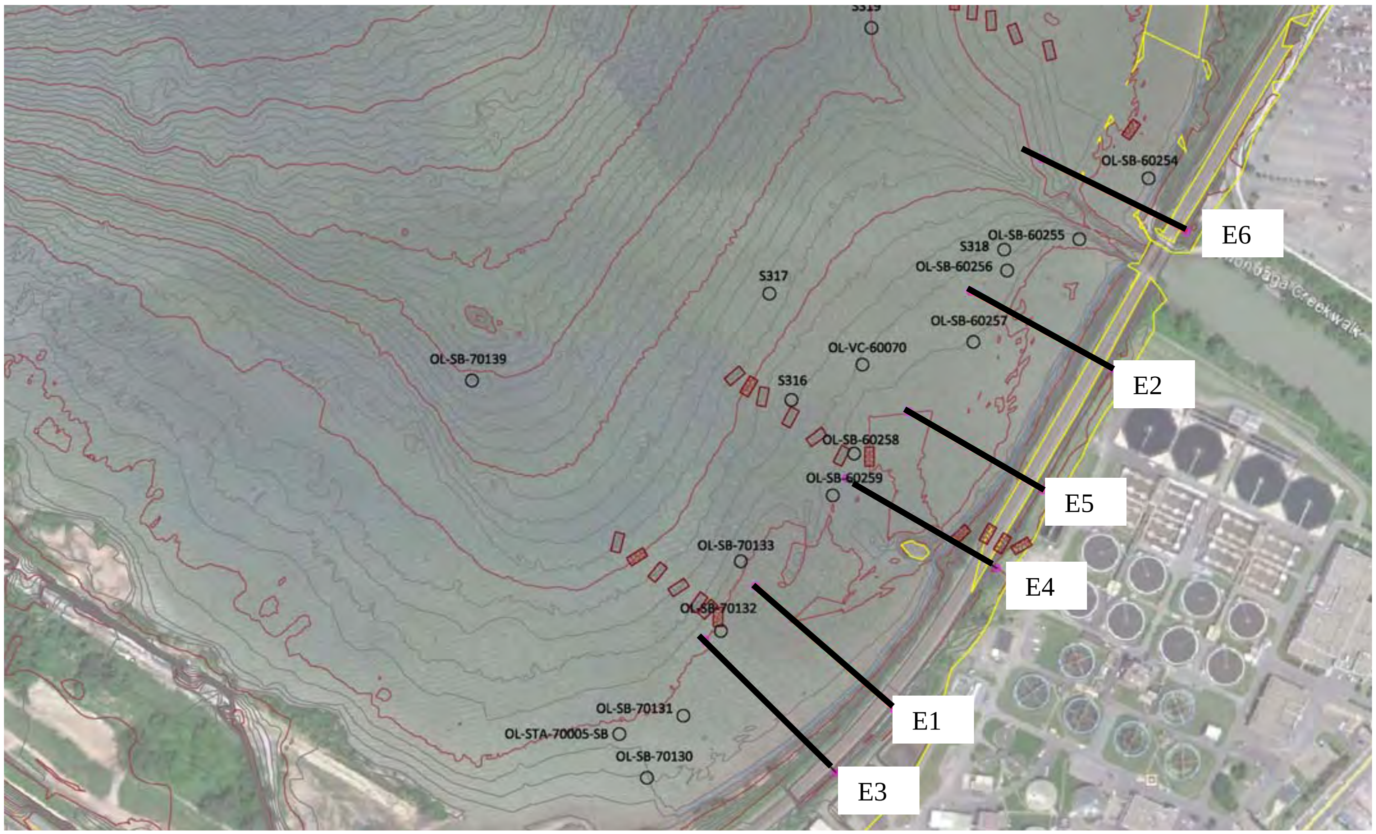

Figure 2. Locations of Selected Six Cross Sections and Available Soil Borings

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Written by:	Mustafa Erten	Date:	4/18/2014	Reviewed by:	Ali Ebrahimi/Jay Beech	Date:	4/18/2014
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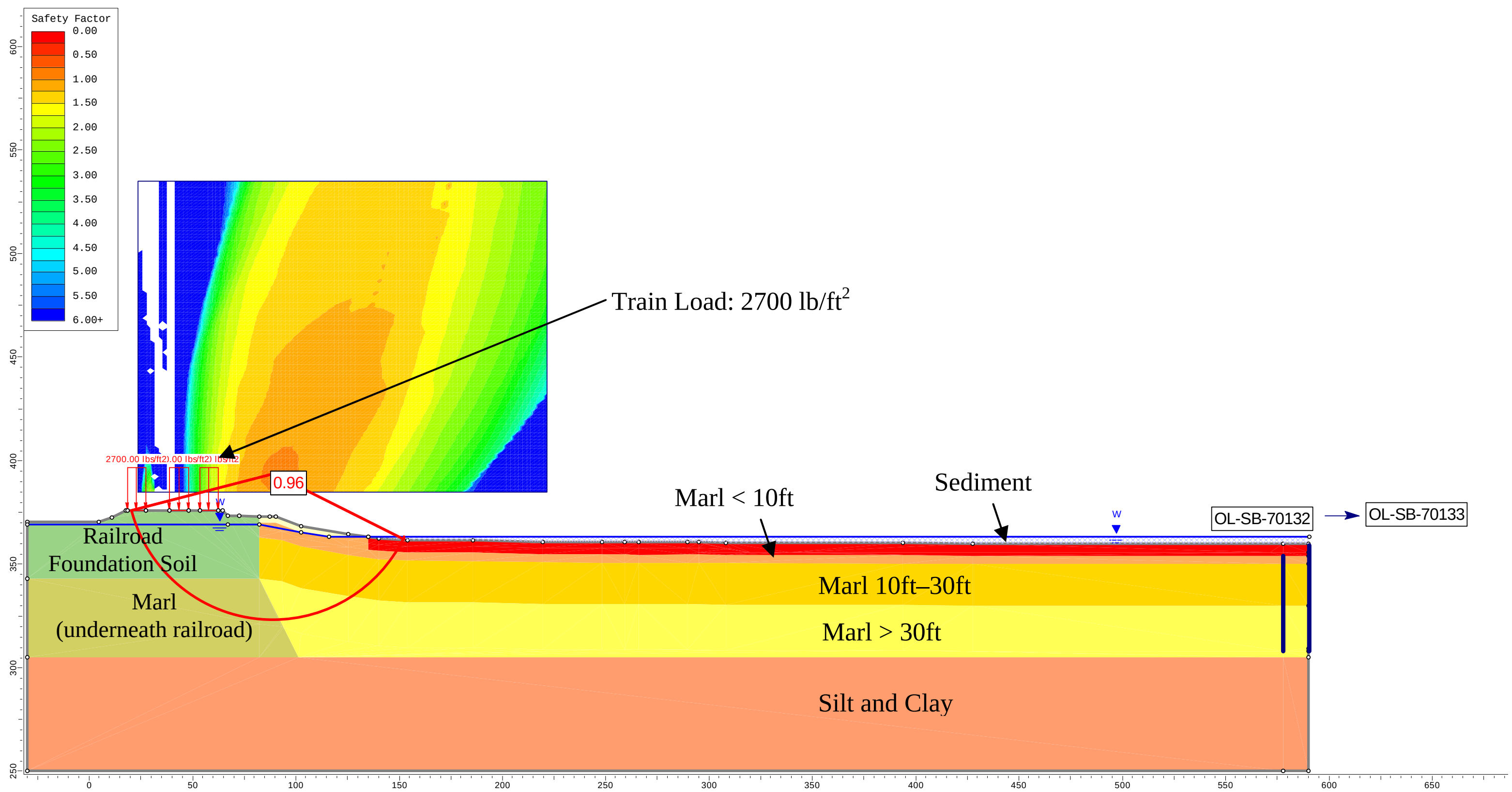

Figure 3. Slope Stability Analysis Results of Cross Section E1 for the Existing Condition

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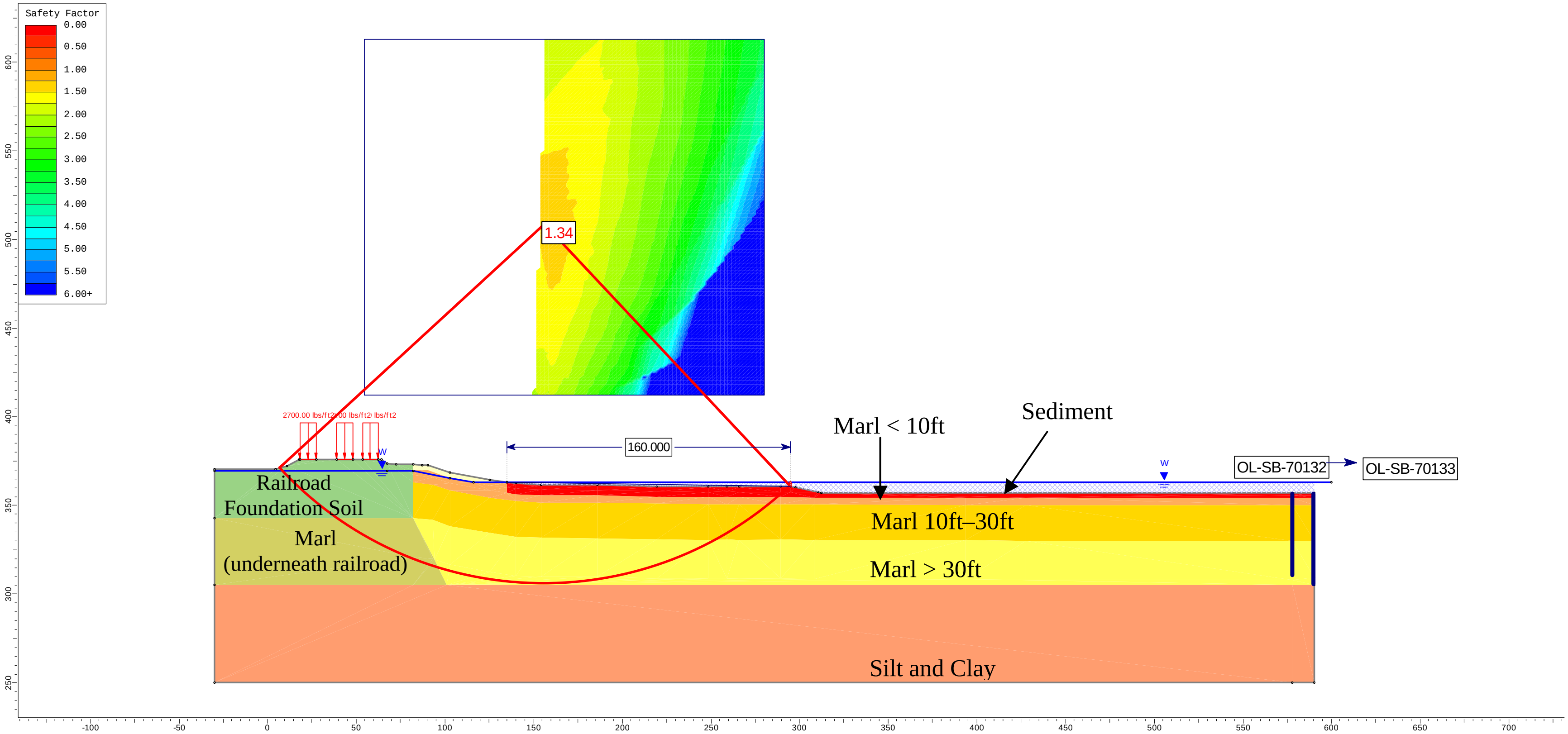

Figure 4. Geometry and Slope Stability Analysis Results of Cross Section E1 (with Offset Distance of 160 ft from the Shoreline)

Written by:	**Mustafa Erten**	Date:	**4/18/2014**	Reviewed by:	**Ali Ebrahimi/Jay Beech**	Date:	**4/18/2014**
Client:	**Honeywell**	Project:	**RA-E Shoreline Offset Dredging**	Project No.:	**GD5453**	Task No.:	**03**

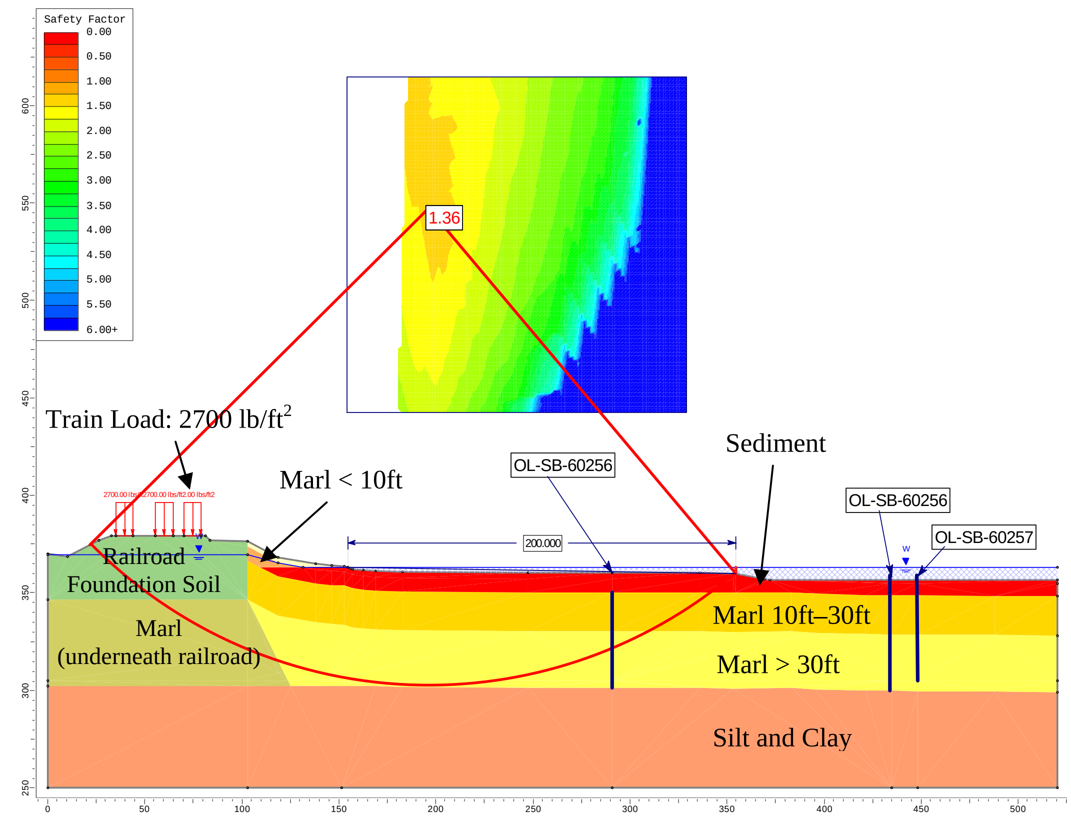

Figure 5. Geometry and Slope Stability Analysis Results of Cross Section E2 (with Offset Distance of 200 ft from the Shoreline)

Written by:	Mustafa Erten	Date:	4/18/2014	Reviewed by:	Ali Ebrahimi/Jay Beech	Date:	4/18/2014
Client:	Honeywell	Project:	RA-E Shoreline Offset Dredging	Project No.:	GD5453	Task No.:	03

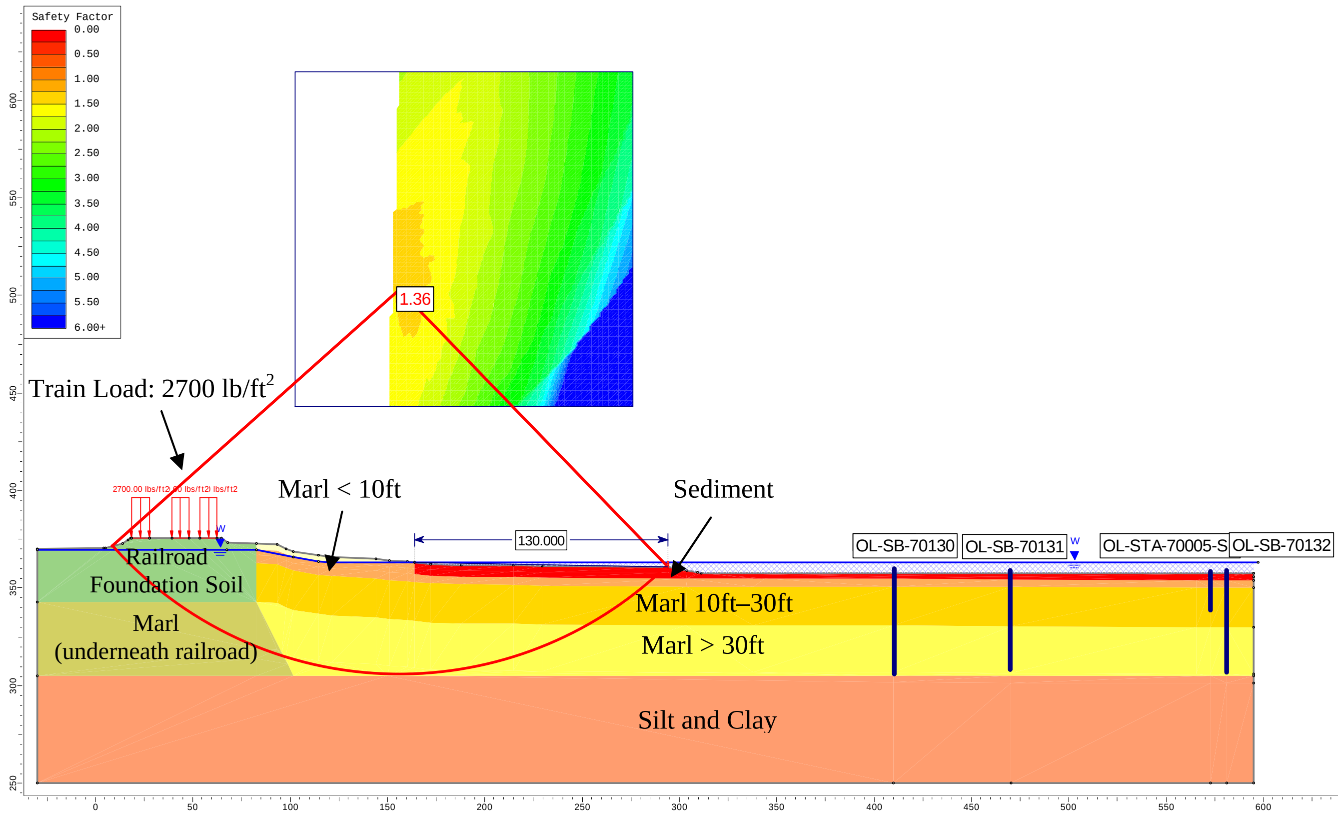

Figure 6. Geometry and Slope Stability Analysis Results of Cross Section E3 (with Offset Distance of 130 ft from the Shoreline)

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Client:	**Honeywell**	Project:	**RA-E Shoreline Offset Dredging**	Project No.:	**GD5453**	Task No.:	**03**

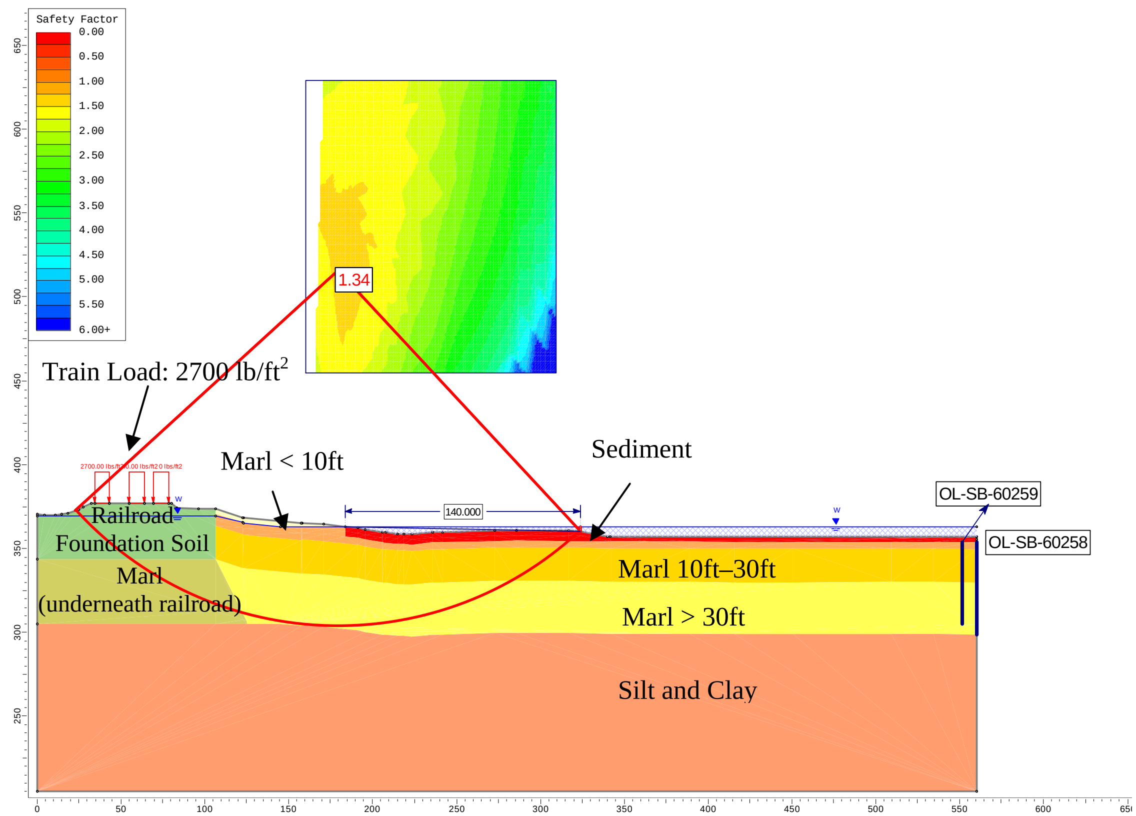

Figure 7. Geometry and Slope Stability Analysis Results of Cross Section E4 (with Offset Distance of 140 ft from the Shoreline)

Written by:	**Mustafa Erten**	Date:	**4/18/2014**	Reviewed by:	**Ali Ebrahimi/Jay Beech**	Date:	**4/18/2014**
Client:	**Honeywell**	Project:	**RA-E Shoreline Offset Dredging**	Project No.: **GD5453**		Task No.:	**03**

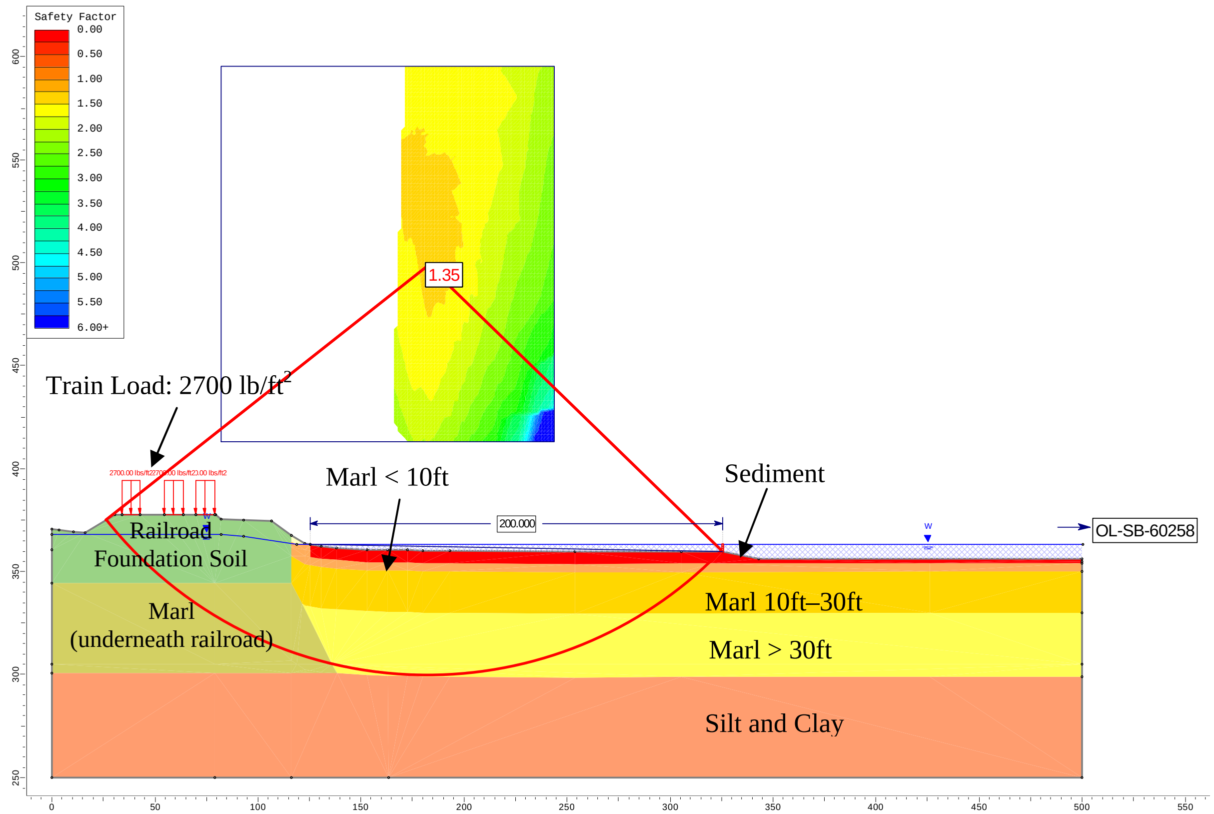

Figure 8. Geometry and Slope Stability Analysis Results of Cross Section E5 (with Offset Distance of 200 ft from the Shoreline)

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Client:	**Honeywell**	Project:	**RA-E Shoreline Offset Dredging**	Project No.:	**GD5453**	Task No.:	**03**

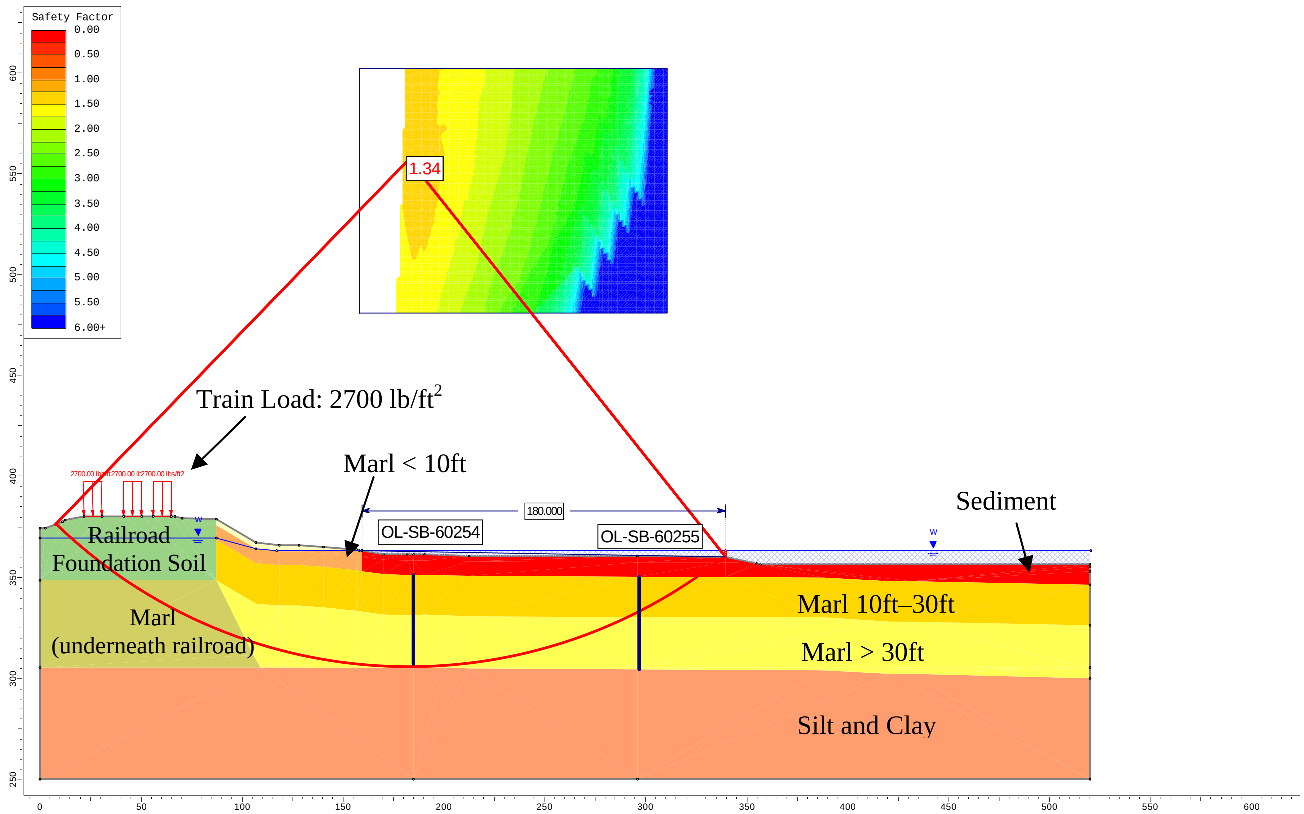

Figure 9. Geometry and Slope Stability Analysis Results of Cross Section E6 (with Offset Distance of 180 ft from the Shoreline)

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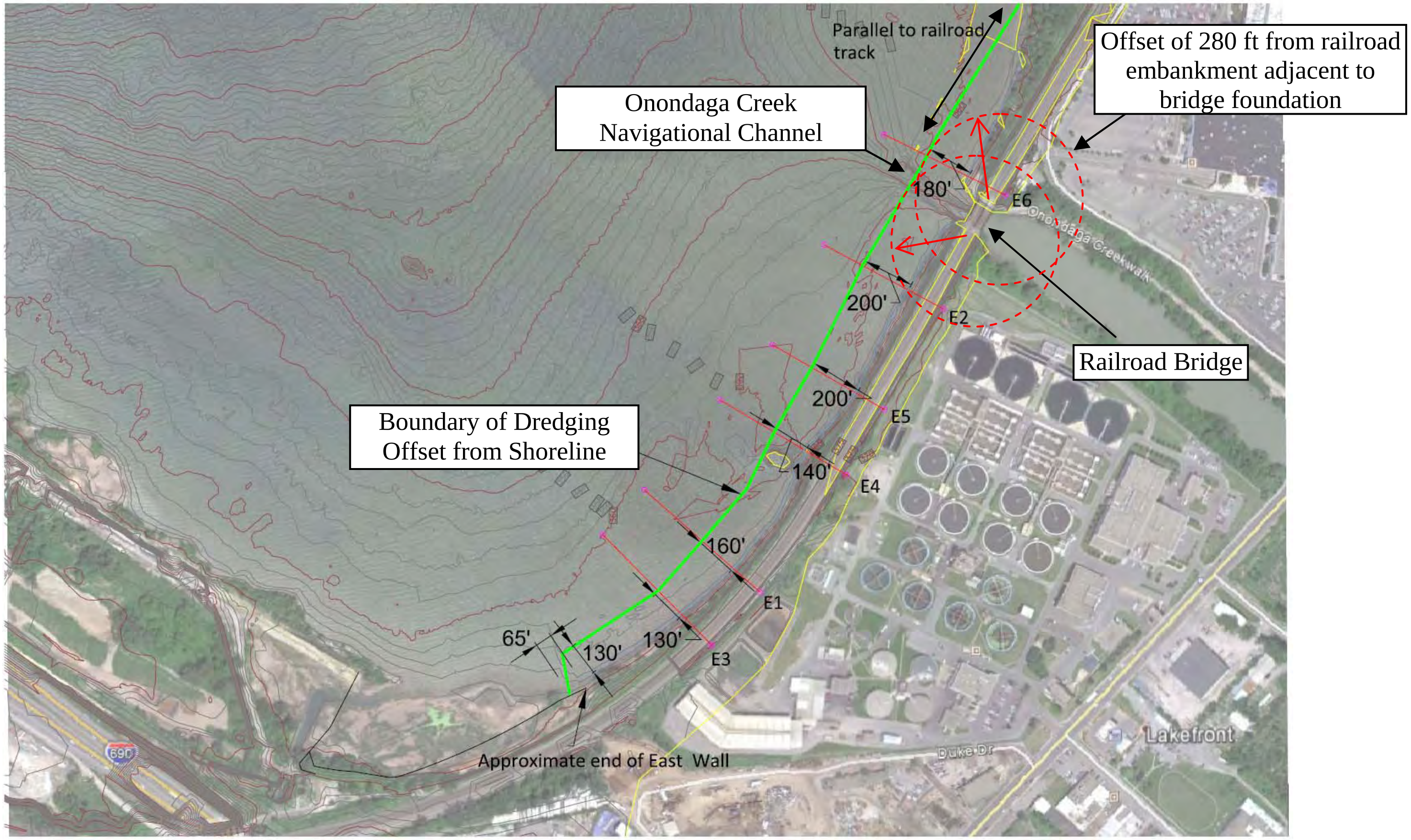

Figure 10. Minimum Required Dredge Offset Distance and Recommended Boundary of No-Dredging Area

Written by:	Mustafa Erten	Date:	5/1/2014	Reviewed by:	Ali Ebrahimi/Jay Beech	Date:	5/1/2014
Client:	Honeywell	Project:	RA-E Shoreline Sheetpile	Project No.:	GD5453	Task No.:	03

ADDENDUM TO APPENDIX A TITLED "SLOPE STABILITY ANALYSIS FOR RA-E SHORELINE DREDGING" (Sheetpile Option along RA-E Shoreline)

INTRODUCTION

The purpose of this addendum is to conduct a preliminary evaluation of using a sheetpile wall with 30 ft distance from the shoreline to improve the dredging stability along the shoreline of Remediation Area E (RA-E) as an alternative to the proposed offset dredging option presented in Appendix A titled "Slope Stability Analysis for RA-E Shoreline Dredging" (herein referred to as Appendix A). A distance of 30 feet was selected in order to avoid the debris present along the shoreline. Geosyntec performed global slope stability and deformation analyses for the sheetpile option to evaluate the railroad stability for the dredging along this alignment. The global slope stability analysis is conducted using limit equilibrium method and the deformation analysis is conducted using finite element method.

There are a number of challenges associated with constructing a sheetpile wall. A considerable portion, if not all, of the sheetpile wall will be on the CSX property and, therefore, obtaining access to construct it may be an issue. Due to the close proximity of the sheetpile wall to the railroad, dredging will induce deflections of the wall. Excessive deflections could cause movement of the railroad.

The detailed geotechnical analyses, presented herein, indicate that global stability of sheetpile can be achieved by a proper embedment depth during installation; however, the dredge-induced deformation at the top of sheetpile can progress toward the rail track, which could result in unacceptable movement of rail lines. Uncertainties related to the properties of railroad foundation prevent a reliable evaluation of railroad deformation.

SLOPE STABILITY AND DEFORMATION ANALYSES

The global slope stability analyses presented in this addendum use the same methodology and subsurface stratigraphy described in Appendix A. Discussion of groundwater elevations, steel sheetpile parameters, external loading, and analysis scenarios are the same as those presented in the package titled "Global Slope Stability Analysis" (herein referred to as Global

Stability Package), which was prepared as part of the East Wall Final Design. The deformation analysis using the finite element method is described below.

The finite element analysis (FEA) was conducted using PLAXIS® version 2012.01, referred to as Plaxis herein, a 2-dimensional (2-D) finite element program. Information required for Plaxis analyses includes the geometry, subsurface stratigraphy, water level, material strength properties, and material stiffness properties.

The Plaxis analysis was performed in two phases: (i) the Initial Condition Phase; and (ii) the Post-Dredging Phase. The Initial Condition Phase calculated the initial state of stress by applying the self-weight as loading and using steady-state porewater pressures corresponding to the specified water level. Because the calculated displacements in this phase under the self-weight loading should have already occurred, those displacements were set to zero prior to the beginning of the Post-Dredging Phase.

In the Post-Dredging Phase, undrained shear strength and stiffness parameters were assigned to the subsurface materials. In the undrained analysis, the analysis option implemented in Plaxis was used to capture the undrained behavior of the subsurface materials. This analysis option calculates excess porewater pressure with the undrained shear strength parameters and the effective stiffness parameters.

TARGET FACTOR OF SAFETY FOR LIMIT EQUILIBRIUM ANALYSIS

The minimum required factor of safety was initially selected as 1.3 for the interim condition in accordance with the Engineering and Design Manual "*Slope Stability*" prepared by the U.S. Army Corps of Engineers [USACE, 2003]. However, for the purpose of the slope stability analyses presented in this package, the minimum required factor of safety was increased by 0.05 to account for potential seepage effect, which was evaluated by Geosyntec during the WB-B/HB barrier wall design. Therefore, the target factor of safety was considered to be 1.35 for the analyzed interim-undrained condition presented in this package.

SUBSURFACE STRATIGRAPHY AND MATERIAL PARAMETERS

The information regarding the subsurface stratigraphy was obtained from the available soil borings, shown in Figure 1, adjacent to the selected cross sections for the slope stability analysis (see more details in Appendix A). The material properties used for the slope stability analysis herein are presented in Table 1 (see more details in Appendix A). In the stability analyses, a 60-ft wide work platform made of fill materials was considered along the shoreline. It is assumed that this work platform remains in-place after sheetpile wall installation.

Table 1. Summary of Material Properties Used in Slope Stability Analysis

Material	Total Unit Weight (pcf)	Drained Shear Strength		Undrained Shear Strength (psf)
		c' (psf)	ϕ' (deg)	
Fill	92	N/A	N/A	145
Sediment	85	N/A	N/A	$\frac{s_u}{\sigma_v^{'}} = 0.3$
Railroad Foundation Soil	120	0	35	N/A
Marl	97	N/A	N/A	160 for depth ≤ 10 ft 250, for 10 ft < depth < 30 ft 400, for depth ≥30 ft
Silt and Clay	110	N/A	N/A	$\frac{s_u}{\sigma_v^{'}} = 0.3$
Marl (underneath railroad)	97	N/A	N/A	$\frac{s_u}{\sigma_v^{'}} = 0.3$

Materials in the FEA were modeled by the linear-elastic, perfectly plastic model with the Mohr-Coulomb failure criteria (i.e., the Mohr-Coulomb Model), which is commonly used for geotechnical analysis. The Mohr-Coulomb Model requires a total of six material parameters: (i) Poisson's ratio (ν); (ii) Young's Modulus (E); (iii) cohesion (c); (iv) friction angle (ϕ); (v) dilatancy angle (ψ); and (vi) unit weight (γ). The linear elastic model requires three material parameters: (i) ν; (ii) E; and (iii) γ. Due to lack of geotechnical laboratory test results in the rail foundation materials, the material parameters of railroad foundation, as listed in Table 2, were estimated based on the available in-situ test results. The value of ψ is added to the residual friction angle to calculate the peak friction angle. For dense soils the value of ψ is greater than zero. For loose soils, the value could be zero. The analyses performed herein conservatively assumed a value of zero for ψ.

Table 2. Summary of Material Properties Used in the Finite Element Analysis

Material	Total Unit Weight (pcf)	Young Modulus, E' (ksf)	Poisson's Ratio, ν'	Undrained Shear Strength, s_u (psf)
Fill	92	50	0.25	145
Sediment	85	50	0.25	40[4]
Railroad Foundation Soil	120	100, 300, 3000[3]	0.30	1600
Marl, depth <10				160
Marl, 10< depth <30	97	50	0.25	250
Marl, depth >30				400
Silt and Clay	110	70	0.25	14.28d + 1035[1,2]
Silt and Clay - Lake	110	70	0.25	14.28d + 566[1,2]
Marl (underneath railroad)	97	300	0.30	640

Notes:
[1]. Undrained shear strength of Silt and Clay was calculated using the undrained shear strength ratio presented in Table 1 for the finite element analysis.
[2]. Depth (d) is measured from the top of the Silt and Clay Layer (assumed El. 305 ft)
[3]. Parametrical analysis was conducted to evaluate the sensitivity of the calculated deformation to the modulus of railroad foundation soil.
[4]. Undrained shear strength of Sediment was calculated using the undrained shear strength ratio presented in Table 1 for the finite element analysis.

ANALYZED CROSS SECTIONS

The six cross sections analyzed in this addendum (i.e., Cross Sections 1 through 6) are shown in Figure 1 (see more details in Appendix A). The global slope stability analysis considered a sheetpile wall that is installed 30 feet from the shoreline. The dredging in the lake was assumed to be 3.5 ft deep and start 30 feet from the sheetpile wall. The train loading was considered in the analysis.

Written by:	Mustafa Erten	Date:	5/1/2014	Reviewed by:	Ali Ebrahimi/Jay Beech	Date:	5/1/2014

Client:	Honeywell	Project:	RA-E Shoreline Sheetpile	Project No.:	GD5453	Task No.:	03

ANALYSIS RESULTS

Global Slope Stability Analysis of Sheetpile for Dredging

The slope stability analyses were performed for the interim-undrained condition after dredging with varying sheetpile length along the shoreline in order to meet the selected target factor of safety of 1.35. Table 3 tabulates the slope stability analysis results and calculated minimum embedment depth for the sheetpile located 30 ft from the shoreline. The calculated minimum required sheetpile length varies between approximately 56.5 ft and 77.5 ft and is dependent on the surface geometry, subsurface stratigraphy, and proximity of the railroad tracks. Figure 2 to Figure 7 show the slip surfaces and calculated factors of safety along each cross section with the calculated minimum required sheetpile lengths.

Table 3. Summary of Analysis Results of Dredging Stability with Sheetpile Located 30 ft from the Shoreline

Cross Section	Calculated Minimum Required Sheetpile Length (ft)	Approximate Distance between Railroad and Sheetpile (ft) [1]	Calculated FS	Note
E1	66.0	100.4	1.35	Figure 2
E2	75.5	103.5	1.35	Figure 3
E3	56.5	129.2	1.36	Figure 4
E4	59.5	133.7	1.36	Figure 5
E5	77.5	76.0	1.36	Figure 6
E6	70.0	122.7	1.36	Figure 7

Note:
[1]. Approximate distance between sheetpile wall and Onondaga Lake shoreline is 30 ft.

Deformation Analysis of Sheetpile for Dredging

The FEA was performed for the interim-undrained condition after dredging to estimate the horizontal and vertical deformation of the railroad due to the dredging in front of sheetpiles. The finite element analysis was conducted for Cross Section E5 as a representative cross section to evaluate the dredge-induced railroad deformation. The geometry and subsurface layers of Cross

Section E5 used in the FEA is shown in Figure 8. The analyses were performed for the sheetpile located 30 ft from the shoreline in the lake.

The calculated deformation contours for Cross Section E5 after dredging with the assumed railroad foundation modulus of 1000 ksf is shown in Figure 9. The deformation contours show that maximum deformation at top of the sheetpile propagates toward the railroad tracks and potentially can cause unacceptable deformation of the railroad. Because of uncertainties in the properties of railroad foundation soils, sensitivity analysis was conducted to evaluate the railroad deformation by varying the modulus of railroad foundation from 1000 ksf to 300 and 50 ksf. Figure 10 shows that the calculated horizontal deformation of rail tracks increases from about 0.1 in. to about 0.5 in. when the modulus of railroad foundation decreases from 1000 to 50 ksf. The results indicate that the risk of unacceptable railroad deformation associated with the uncertainty in railroad foundation modulus.

The calculated vertical and horizontal deformation profiles at Cross Section E5 with the railroad foundation modulus of 50 ksf and 1000 ksf are shown in Figure 11. At the location of rail tracks, a calculated vertical deformation of about 0.2 in. and horizontal deformation of about 0.1 in. for E=1000 ksf were estimated. For E=50 ksf, a calculated vertical deformation of about 0.6 in. and horizontal deformation of about 0.5 in. was estimated at the location of rail tracks. The estimated vertical and horizontal deformation of the railroad tracks after dredging in front of the sheetpile, using the assumed higher modulus (E=1000 ksi) for railroad foundation, is considered marginally acceptable based on the maximum tolerable deformation of 0.6 in., specified by Davis and Chrismer (2007). In the case of a lower modulus for the railroad foundation, the calculated deformation increases significantly, as presented in Figure 9 and Figure 11.

SUMMARY

Geotechnical engineering analyses to evaluate the global stability of sheetpile and deformation of the railroad are presented in this addendum. The analysis results indicate that the global stability of dredging can be achieved by selecting a minimum required sheetpile length ranging from between approximately 56.5 ft and 77.5 ft at the selected cross sections; however, the dredge-induced deformation at the top of sheetpile can potentially progress toward the rail tracks and result in unacceptable movement of the rail tracks. Therefore, the installation of a sheetpile wall located 30 ft from the shoreline in the lake was not considered an acceptable approach.

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Davis, D. and Chrismer, S. (2007), "Track Differential Settlement Model," Proc. Of JRCICE 2007, Colorado, Pueblo.

U.S. Army Corps of Engineers (USACE) (2003), "*Engineering and Design – Slope Stability*", Engineering Manual EM 1110-2-1902, October 2003.

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Client:	**Honeywell**	Project:	**RA-E Shoreline Sheet Pile**	Project No.:	**GD5453**	Task No.:	**03**

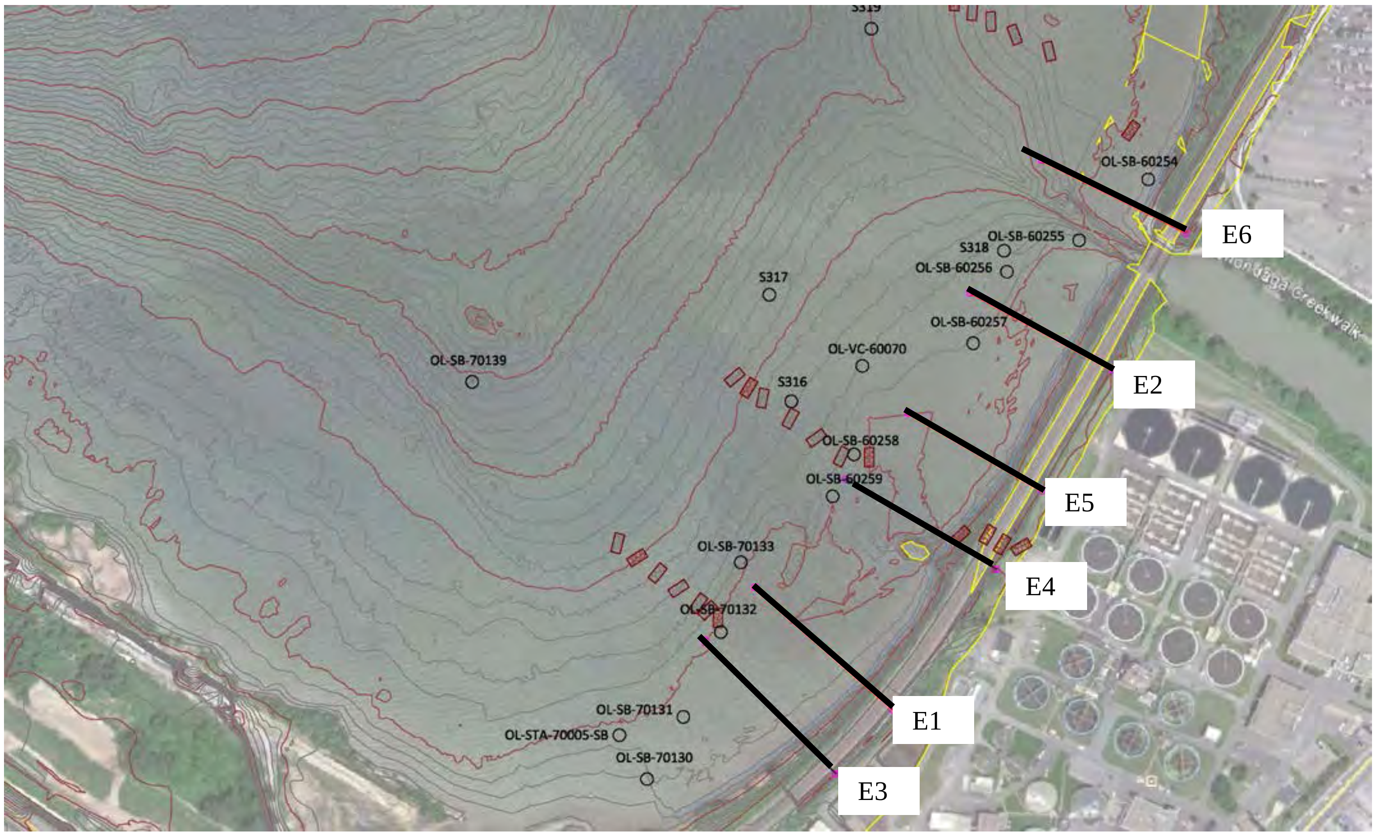

Figure 1. Locations of Selected Six Cross Sections and Available Soil Borings

Written by:	Mustafa Erten	Date:	5/1/2014	Reviewed by:	Ali Ebrahimi/Jay Beech	Date:	5/1/2014

Client: **Honeywell** Project: **RA-E Shoreline Sheet Pile** Project No.: **GD5453** Task No.: **03**

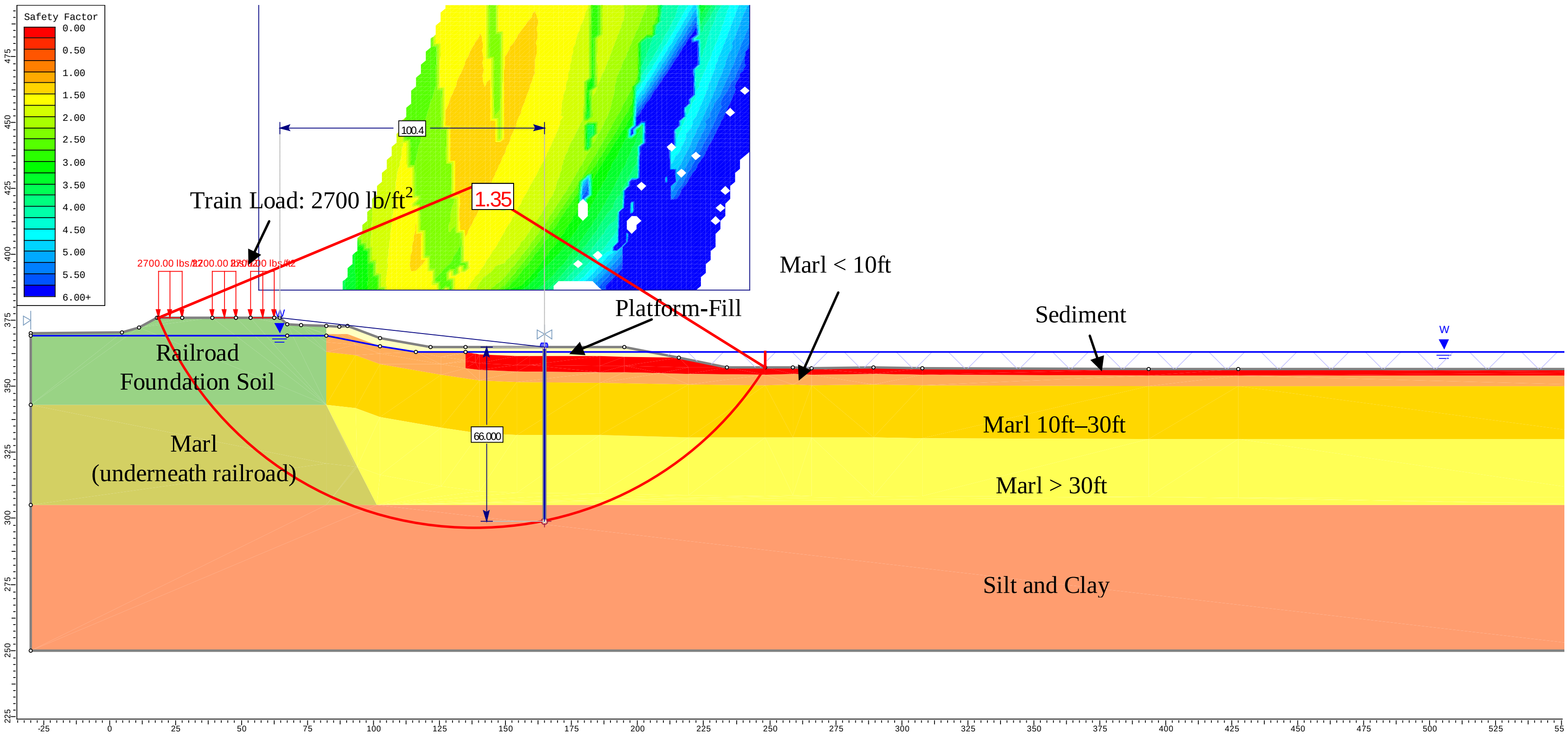

Figure 2. Geometry and Slope Stability Analysis Results of Cross Section E1 where sheetpile is 30 ft away from the shoreline

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Client:	**Honeywell**	Project:	**RA-E Shoreline Sheet Pile**	Project No.:	**GD5453**	Task No.:	**03**

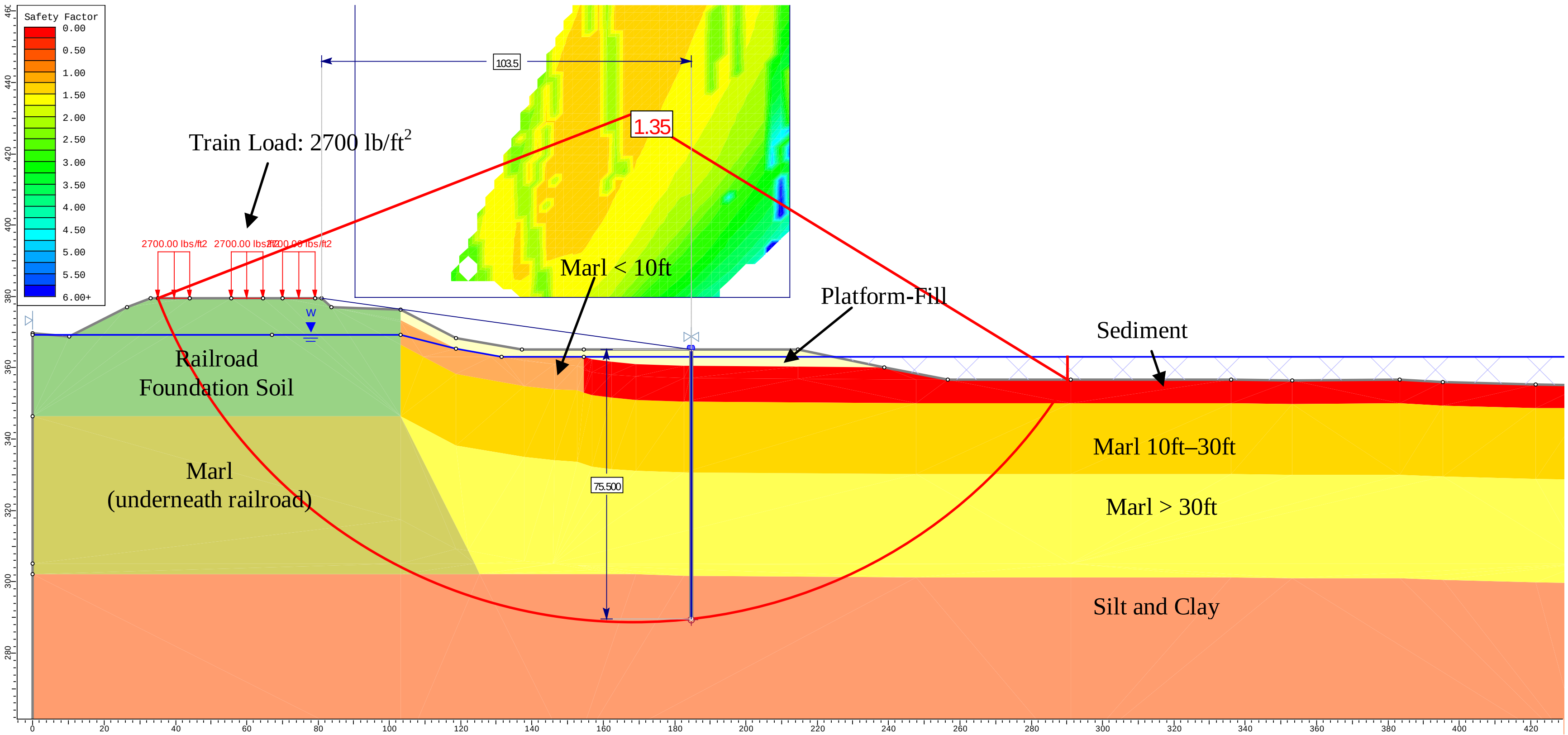

Figure 3. Geometry and Slope Stability Analysis Results of Cross Section E2 where sheetpile is 30 ft away from the shoreline

Written by: **Mustafa Erten** Date: **5/1/2014** Reviewed by: **Ali Ebrahimi/Jay Beech** Date: **5/1/2014**

Client: **Honeywell** Project: **RA-E Shoreline Sheet Pile** Project No.: **GD5453** Task No.: **03**

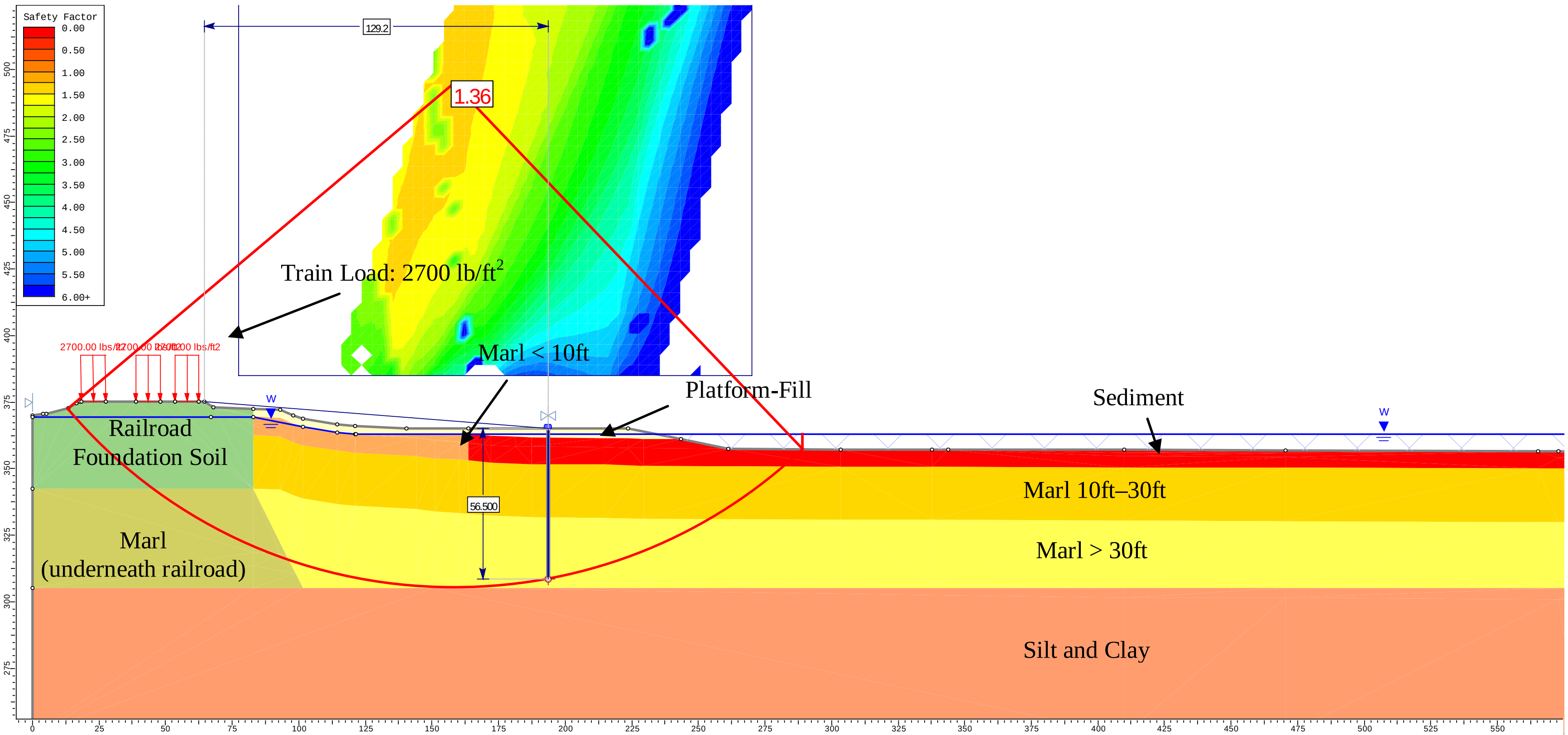

Figure 4. Geometry and Slope Stability Analysis Results of Cross Section E3 where sheetpile is 30 ft away from the shoreline

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Client:	**Honeywell**	Project:	**RA-E Shoreline Sheet Pile**	Project No.:	**GD5453**	Task No.:	**03**

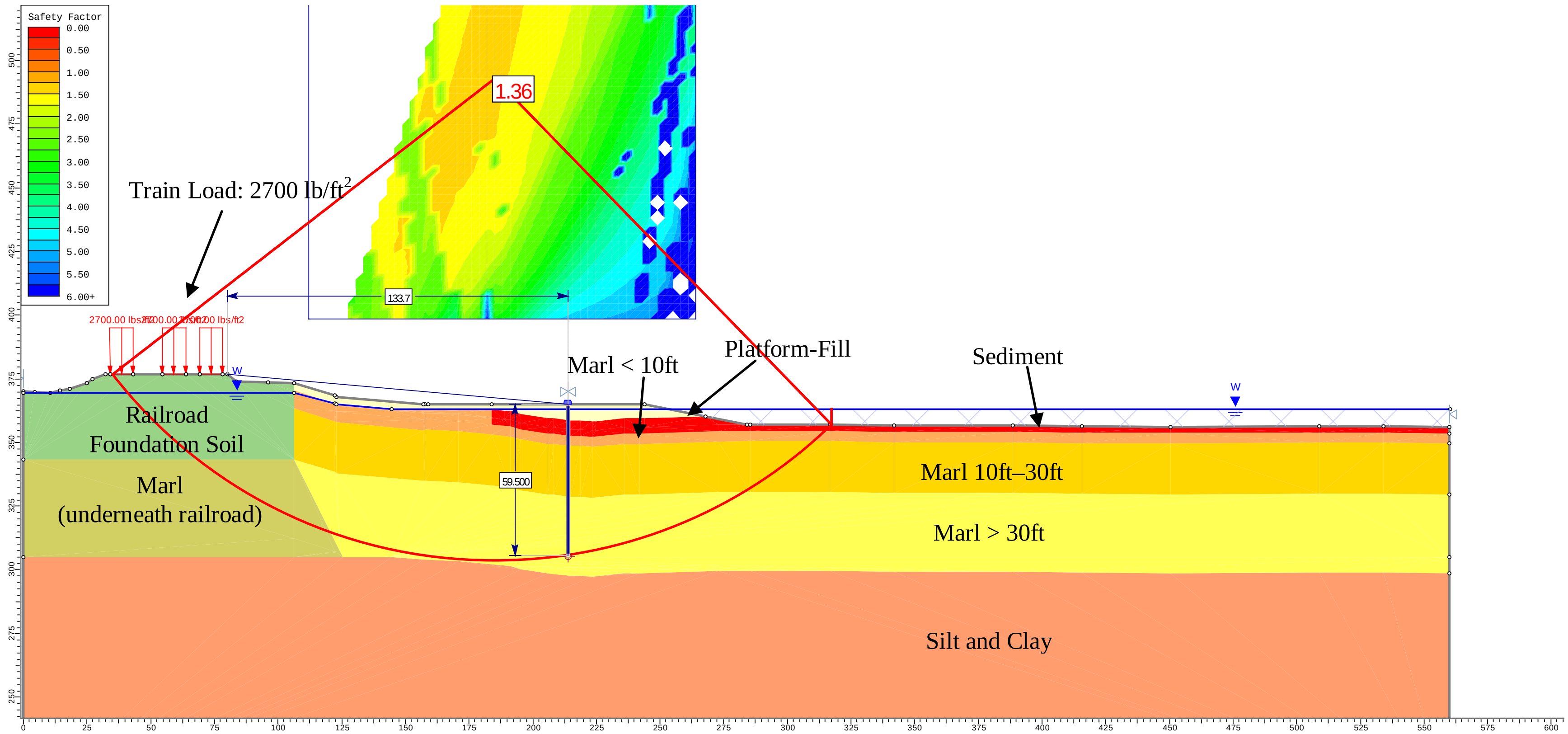

Figure 5. Geometry and Slope Stability Analysis Results of Cross Section E4 where sheetpile is 30 ft away from the shoreline

Written by:	**Mustafa Erten**	Date:	**5/1/2014**	Reviewed by:	**Ali Ebrahimi/Jay Beech**	Date:	**5/1/2014**

Client: **Honeywell** Project: **RA-E Shoreline Sheet Pile** Project No.: **GD5453** Task No.: **03**

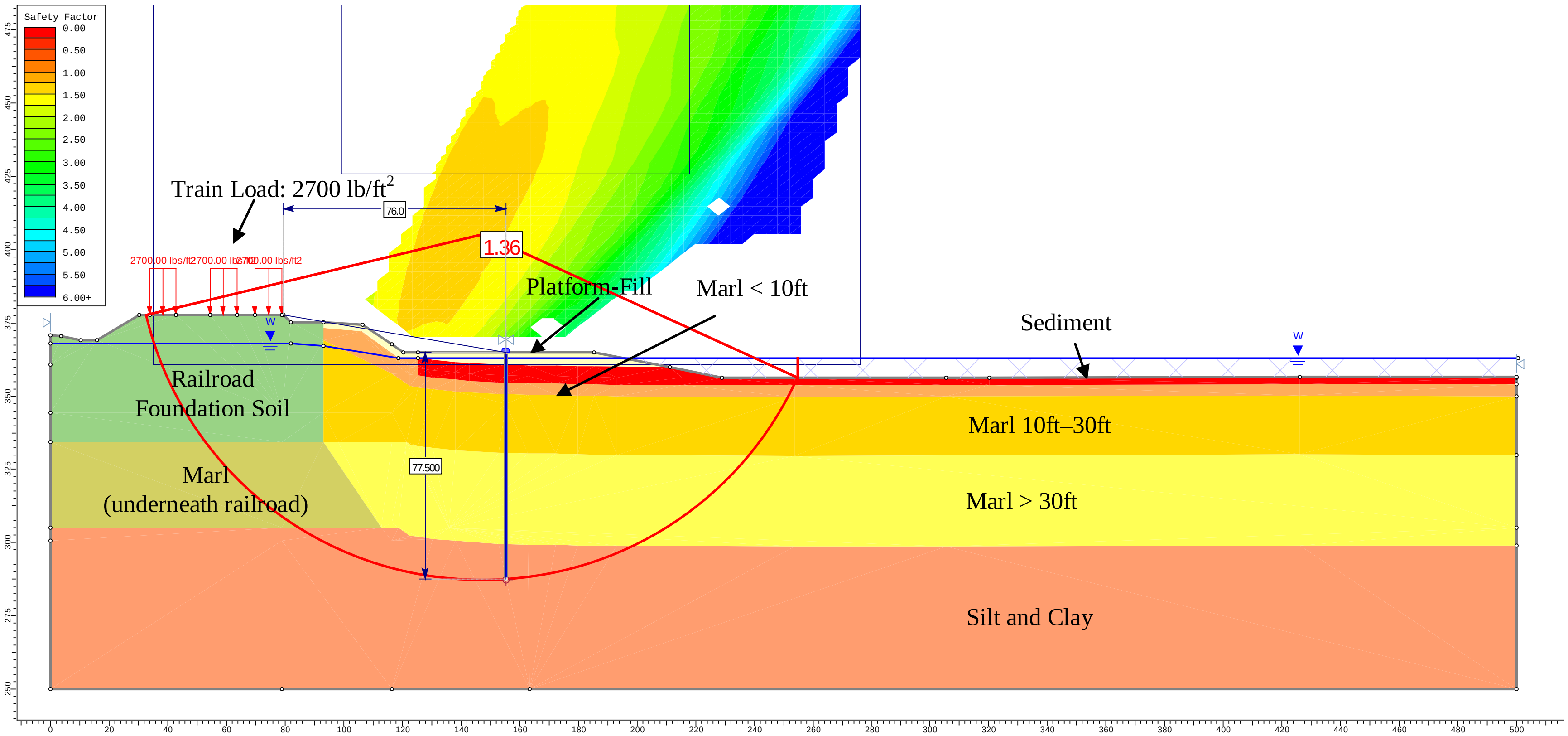

Figure 6. Geometry and Slope Stability Analysis Results of Cross Section E5 where sheetpile is 30 ft away from the shoreline

Written by: **Mustafa Erten** Date: **5/1/2014** Reviewed by: **Ali Ebrahimi/Jay Beech** Date: **5/1/2014**

Client: **Honeywell** Project: **RA-E Shoreline Sheet Pile** Project No.: **GD5453** Task No.: **03**

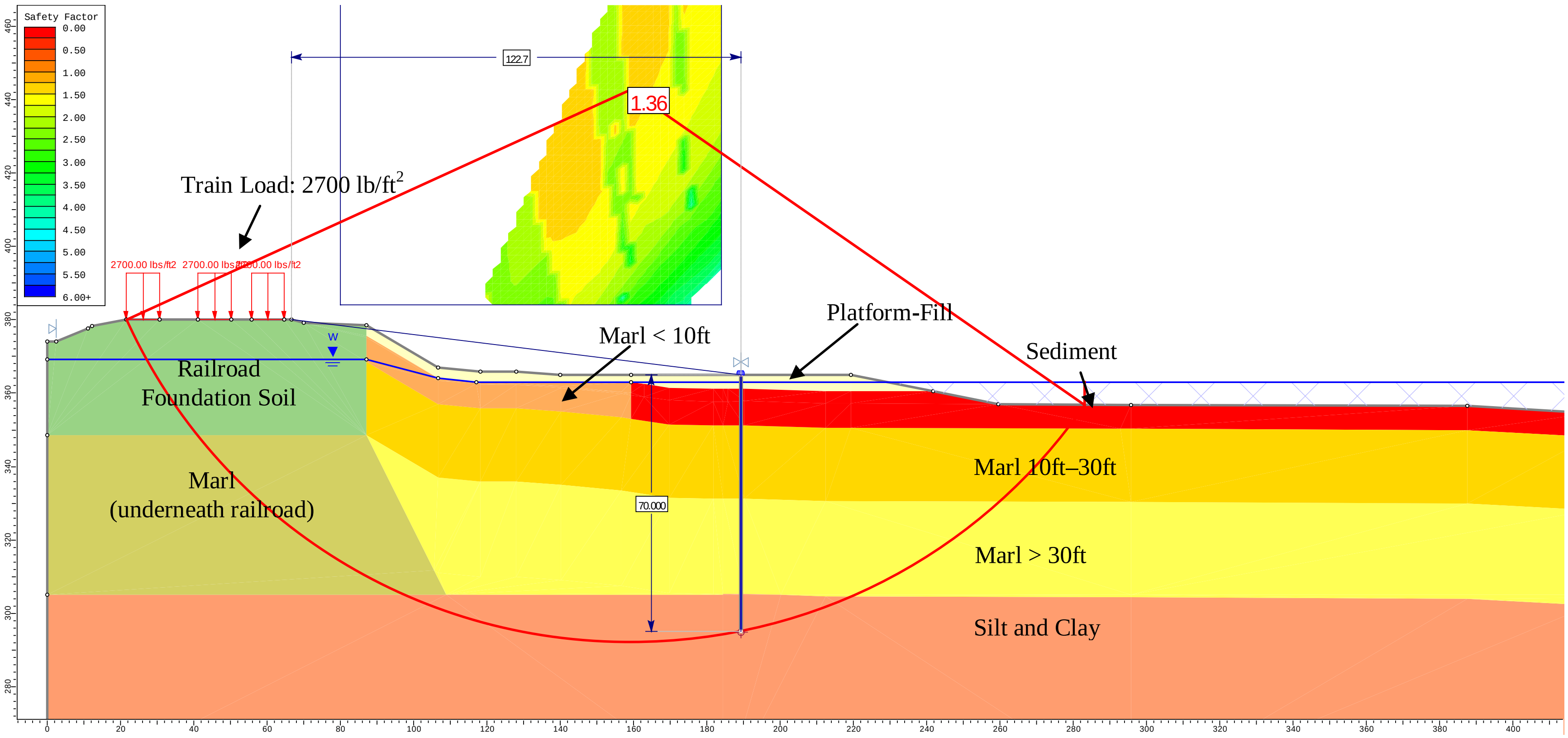

Figure 7. Geometry and Slope Stability Analysis Results of Cross Section E6 where sheetpile is 30 ft away from the shoreline

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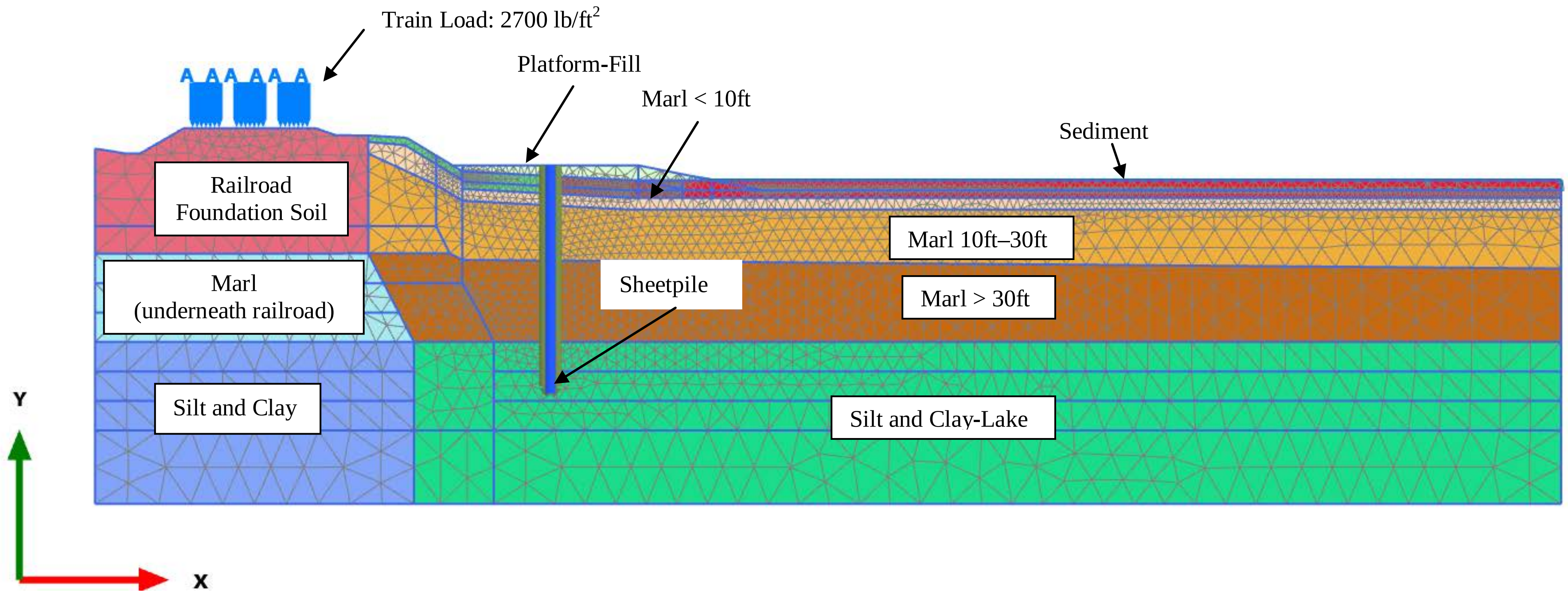

Figure 8. Geometry and Subsurface Layers Used in Finite Element Analysis of Cross Section E5 where sheetpile is 30 ft away from the shoreline

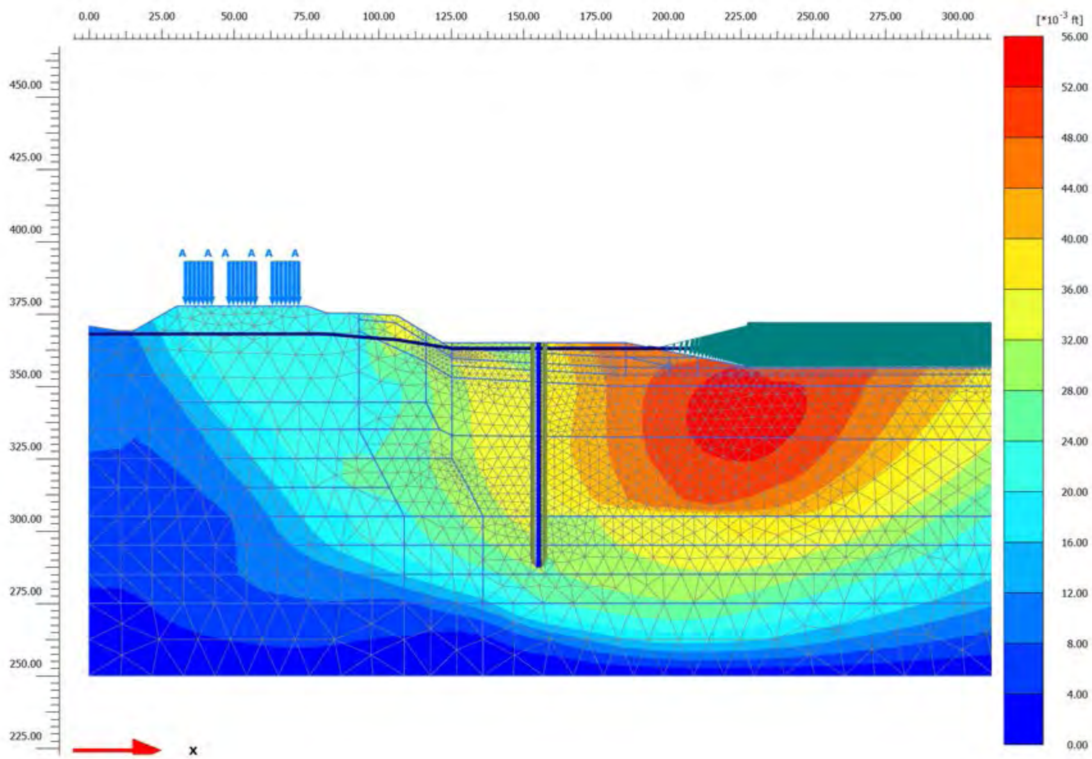

Figure 9. Calculated Deformation Contours after Dredging in front of Sheetpile at Cross Section E5 (Railroad Foundation Soil Modulus, E = 1000 ksf)

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Client:	Honeywell	Project:	RA-E Shoreline Sheet Pile	Project No.:	GD5453	Task No.:	03

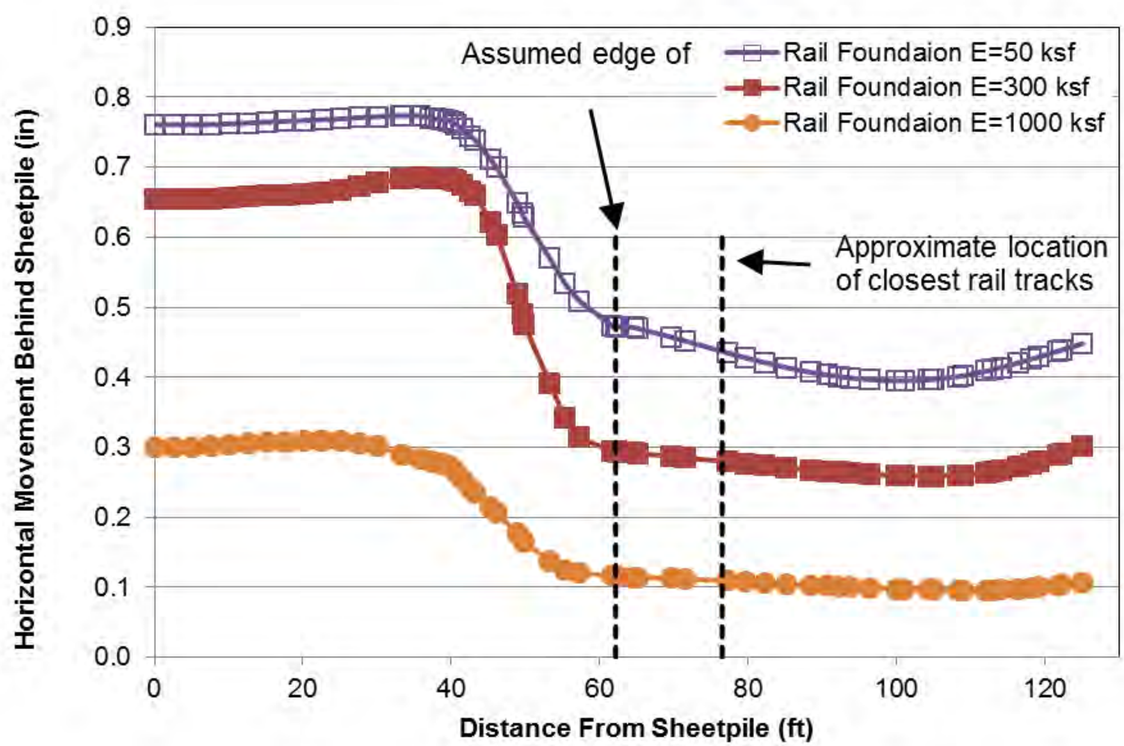

Figure 10. Sensitivity of Calculated Horizontal Deformation to Modulus of Railroad Foundation at Cross Section E5 where sheetpile is located 30 ft away from the shoreline (Railroad Foundation Soil Modulus, E = 50, 300, and 1000 ksf)

Written by:	Mustafa Erten	Date:	5/1/2014	Reviewed by:	Ali Ebrahimi/Jay Beech	Date:	5/1/2014
Client:	Honeywell	Project:	RA-E Shoreline Sheet Pile	Project No.:	GD5453	Task No.:	03

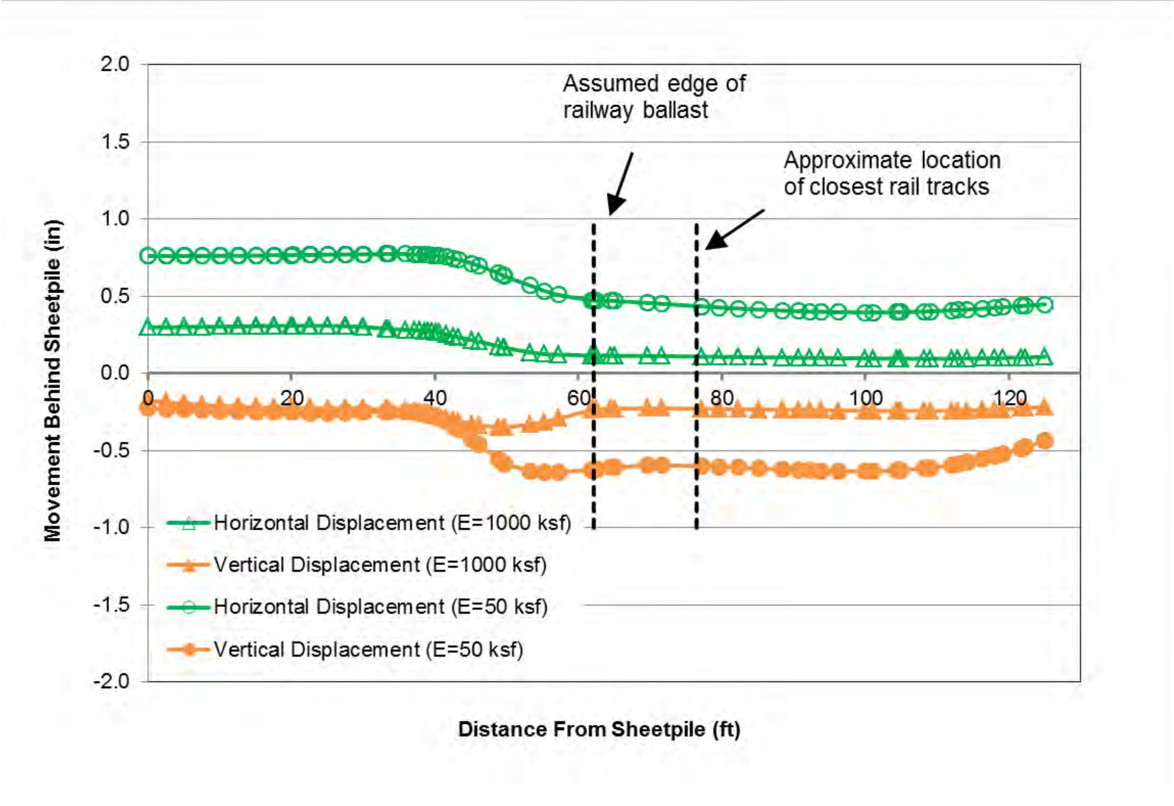

Figure 11. Calculated Vertical and Horizontal Deformation Profiles at Cross Section E5 where sheetpile is located 30 ft away from the shoreline (Railroad Foundation Soil Modulus, E = 50 and 1000 ksf)

Written by:	**Joseph Sura**	Date:	**4/29/2014**	Reviewed by:	**Ming Zhu/Jay Beech**	Date:	**4/29/2014**
Client:	**Honeywell**	Project:	**RA-E Shoreline Surcharge**	Project No.:	**GD5453**	Task No.:	**03**

ADDENDUM TO APPENDIX A TITLED "SLOPE STABILITY ANALYSIS FOR RA-E SHORELINE DREDGING" (Temporary Surcharge Option along RA-E Shoreline)

INTRODUCTION

The purpose of this addendum is to present the evaluation of using temporary surcharge (i.e., a soil berm) to improve the dredging stability and place the cap along the shoreline of Remediation Area E (RA-E) as an alternative to the proposed offset dredging option presented in Appendix A titled "Slope Stability Analysis for RA-E Shoreline Dredging" (herein referred to as Appendix A). Geotechnical analyses were performed to evaluate the required temporary surcharge to be placed within the Onondaga Lake in order to be able to install the lake cap and provide buttress for the lake dredging. The analyses were performed to evaluate the impact of the proposed surcharge loading on the rail lines.

Theoretically, the surcharge would improve the stability by serving as a buttress against slip and would also cause sufficient settlement of the subsurface soils to allow placement of the cap without dredging. As shown in Figures 1A through 1E, the surcharge would be a five-step process: (i) Stage 1 – construction of the soil berm (including the cap at the bottom) at Year 0; (ii) Stage 2 – dredging in front of the soil berm at Year 2; (iii) Stage 3 – construction of the cap in front of the soil berm (the lake side) at Year 2; (iv) Stage 4 – placement of additional fill in front of the soil berm at Year 2; and (v) Stage 5 – removal of the soil berm at the end of Year 5 to leave the cap in place after sufficient settlement has occurred.

METHODOLOGY

Slope stability analyses were performed using the same methodology described in detail in Appendix A. The minimum required factor of safety (FS) was considered to be 1.35 as discussed in Appendix A. The calculations were performed for Stage 2 of the surcharge phasing (i.e., after dredging of the lake). It is noted that Stages 3 and 4 involve placement of cap material and additional fill, respectively, which will serve as a buttress for the railroad and are expected to increase the calculated FS for slip surfaces passing through the railroad bed and the dredge area. It is noted that for purposes of this addendum, slip surfaces which did not pass through the railroad bed (e.g., local slip of the berm itself) and bearing capacity of the sediment underlying the surcharge were not considered. If the surcharge is selected, additional slope stability and bearing capacity analyses should be performed to refine and finalize the analyses for the recommended approach.

Settlement analyses were performed using the computer program Settle3D [Rocscience, 2013]. Settle3D is a 3-dimensional program for analysis of settlement and consolidation, which computes settlements, stresses and porewater pressures throughout the 3-dimensional volume. Settle3D was selected for use due to the program's ability to model load dissipation (e.g., using Bousinesq stress dissipation methods) and time-dependent consolidation. Input parameters required for Settle3D include loading (i.e., surcharge geometry and weight), subsurface stratigraphy, consolidation parameters, and unit weights. It is noted that due to the assumed 5-year timespan of the surcharge, only the primary consolidation was considered in the analysis; and the secondary consolidation (creep) was not considered.

INPUT PARAMETERS

Surcharge Geometry

As shown in Figures 1A through 1E, the surcharge installed in Stage 1 is considered to have a top width of 30 ft and a height of 18.5 ft. The side slopes of the soil berm was assumed to be 2.5 horizontal to 1 vertical (2.5H:1V). It is noted that the 18.5 ft height includes the assumed 5-ft thick cap at the bottom of the surcharge. As shown in Figure 1D, an additional 50-ft wide fill will be placed in Stage 4. The additional fill will have the same top elevation and side slope geometries.

Analyses were also performed to evaluate the potential for using a smaller surcharge with a cap thickness of 3 ft (referred to as the 10.5-ft surcharge). The 10.5-ft surcharge is considered to have a top width of 30 ft, a height of 10.5 ft (including the assumed 3-ft thick cap) and side slopes of 2.5H:1V. The 10.5-ft surcharge follows the same staging pattern as the 18.5 ft surcharge (e.g., construction of the soil berm at Year 0, dredging at Year 2, etc.).

Slope Stability Parameters

Six cross sections were selected in Appendix A to represent the anticipated critical conditions (from a stability viewpoint) and typical conditions (from a spatial coverage viewpoint) along the RA-E shoreline, as shown in Figure 2. The material properties, railroad loading, cross section geometry, and subsurface stratigraphy for the slope stability analyses have been previously described in detail in Appendix A. A summary of the selected material properties is included in Table 1 for reference.

It is noted that the placement of the surcharge is expected to increase the shear strength of the subsurface layers due to consolidation. Therefore, the shear strength of the Marl underneath the flat top of the surcharge was increased appropriately to model the effect of consolidation, as shown in Table 1. Due to stress dissipation effects, while the subsurface Marl underneath the side slopes of the surcharge is expected to experience some strength gains, the increased strength may be limited. Therefore, the increased shear strength is conservatively only applied underneath the flat top of the surcharge.

Settlement Parameters

Settlement parameters were selected as part of the WB-B/HB barrier wall design for the existing fill, sediment, Marl, and Silt and Clay layers, as shown in Table 2. For purposes of the settlement analyses, one representative cross-section (i.e., Section E1) was evaluated. In order to evaluate the potential variability in settlement in the field, a sensitivity analysis was performed by varying the consolidation parameters using average and average plus and minus one standard deviation as shown in Table 2. Each case of the sensitivity analysis calculated: (i) the settlement after 5 years of surcharge placement; (ii) the theoretical full primary consolidation settlement after a long time (e.g., 50 years or more); and (iii) the time in years necessary to estimate 3-ft or 5-ft settlement underneath the flat top of the surcharge (i.e., how long the surcharge must remain in place to cause sufficient settlement for cap placement).

ANALYSIS RESULTS

Slope Stability

18.5-ft Surcharge

For the 18.5-ft surcharge, the calculated FS values for Stage 2 (post-dredging) were found to vary between 0.92 and 1.69, as shown in Table 3. Based on the slip surfaces shown in Figures 3 through 8, the calculated FS seems to be significantly affected by the distance between the railroad and the shoreline. For Section E5, where the shoreline is very close to the railroad, the side slope of the surcharge loading is located on top of the railroad roadbed and the calculated FS value of 0.92 indicates that slip surface is potentially unstable.

10.5-ft Surcharge

For the 10.5-ft surcharge, analyses were performed for a typical cross-section (E1) and the most critical section from the 18.5 ft surcharge stability analyses, E5. The calculated FS values for Stage 2 were found to be 1.25 and 1.18, for E1 and E5, respectively, as shown in Table 3 and Figures 9 and 10.

Settlement Analyses

18.5-ft Surcharge

The settlement analyses for the 18.5-ft surcharge are summarized in Table 4. Figure 11 illustrates the calculated settlement using average consolidation parameters underneath the surcharge as a function of distance from the lake toe of the surcharge (e.g., distance of 0 ft indicates the toe of the surcharge within the lake) for several consolidation times after surcharge construction (i.e., 1 year, 2 year, 5 year and 50 years). Based on these results, the calculated settlement under the surcharge is approximately 5.1 ft after 5 years, with a calculated settlement after 50 years (i.e., full consolidation) of approximately 5.5 ft. It is noted that Figure 11 indicates that long-term, settlements of 0.4 ft may be observed 30 ft or more away from the surcharge. In Section E5, the distance between the limit of the surcharge loading to the nearest railroad track is only 27 ft and therefore, the surcharge-induced settlement may impact the existing railroad.

As summarized in Table 4, the time required to reach 5-ft of calculated settlement varies significantly based on the assumed consolidation parameters. For average coefficient of consolidation (Cv), 5 ft of settlement will be achieved after 1.3 years when the mean plus one standard deviation values were considered for Cce (modified compression index) and Cre (modified recompression index). However, when the mean minus one standard deviation values were considered for Cce and Cre, the calculated full primary consolidation settlement is only 4.0 ft and therefore, the target 5 ft of settlement could not be achieved.

10.5-ft Surcharge

The settlement analyses for the 10.5-ft surcharge are summarized in Table 5. Figure 12 illustrates the calculated settlement using average consolidation parameters underneath the surcharge as a function of distance from the lake toe of the surcharge (e.g., distance of 0 ft indicates the toe of the surcharge within the lake) for several consolidation times after surcharge construction (i.e., 1 year, 2 year, 5 year and 50 years). Based on these results, the calculated settlement under the surcharge is approximately 3.0 ft after 5 years, with a calculated settlement after 50 years (i.e., full consolidation) of approximately 3.2 ft. It is noted that Figure 12 indicates that long-term, settlements of 0.2 ft may be observed 30 ft or more away from the surcharge. In

Section E5, the distance between the limit of the 10.5-ft surcharge loading to the nearest railroad track is only 35 ft and therefore, the surcharge-induced settlement may impact the existing railroad.

As summarized in Table 5, the time required to reach 3-ft of calculated settlement varies significantly based on the assumed consolidation parameters. For average coefficient of consolidation (Cv), 5 ft of settlement will be achieved after 1.5 years when the mean plus one standard deviation values were considered for Cce (modified compression index) and Cre (modified recompression index). However, when the mean minus one standard deviation values were considered for Cce and Cre, the calculated full primary consolidation settlement is only 2.6 ft and therefore, the target 3 ft of settlement could not be achieved.

SUMMARY

Based on the settlement analysis results presented in this package, the surcharge load may cause unacceptable settlement to the existing railroad tracks. It is also noted that the required time for the surcharge to reach a sufficient settlement may be unacceptably long (i.e., up to approximately 9 years for the 18.5-ft surcharge and 5-ft cap or up to approximately 17 years for the 10.5-ft surcharge and 3-ft cap) and in the worst case scenario, satisfactory settlement may never be achieved in certain locations due to potential variability in consolidation properties of the subsurface materials. In addition, based on the slope stability analysis results, not all of the calculated FS values met the target FS. Therefore, placement of a temporary surcharge was not considered to be an acceptable approach.

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Client: **Honeywell** Project: **RA-E Shoreline Surcharge** Project No.: **GD5453** Task No.: **03**

REFERENCES

Rocscience (2013), "*Settle3D*" Rocscience Software, Inc., Toronto, Ontario, Canada, November 2013.

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Client: **Honeywell** Project: **RA-E Shoreline Surcharge** Project No.: **GD5453** Task No.: **03**

Table 1. Summary of Material Properties Used in Slope Stability Analysis

Material	Total Unit Weight (pcf)	Drained Shear Strength		Undrained Shear Strength (psf)
		c' (psf)	ϕ ' (degree)	
Existing Fill	92	N/A	N/A	145
Sediment	85	N/A	N/A	$\frac{s_u}{\sigma_v'} = 0.3$
Railroad Foundation Soil	120	0	35	N/A
Marl	97	N/A	N/A	160 for depth ≤ 10 ft 250, for 10 ft < depth < 30 ft 400, for depth ≥30 ft
Marl (consolidated) 18.5-ft surcharge[2]	97	N/A	N/A	758 for depth ≤ 10 ft 811, for 10 ft < depth < 30 ft 845, for depth ≥30 ft
Marl (consolidated) 10.5-ft surcharge[2]	97	N/A	N/A	488 for depth ≤ 10 ft 558, for 10 ft < depth < 30 ft 645, for depth ≥30 ft
Silt and Clay	110	N/A	N/A	$\frac{s_u}{\sigma_v'} = 0.3$
Marl (underneath railroad)	97	N/A	N/A	$\frac{s_u}{\sigma_v'} = 0.3$
Surcharge Fill	120	0	35	N/A

Notes:

1. The material properties listed above are the same as those in Appendix A.
2. After 2 years of consolidation under surcharge loading (i.e., prior to dredging in the lake), the undrained shear strengths are expected to increase as shown. The increase in shear strength was calculated based on the percent consolidation after 2 years calculated for each layer in the settlement analyses.

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Table 2. Summary of Material Properties Used in Settlement Analysis

Material	Layer Thickness (ft)	Unit Weight (pcf)	Consolidation Coefficient (C_v)			OCR		Modified Compression Index ($C_{c\varepsilon}$)			Modified Recompression Index ($C_{r\varepsilon}$)		
			Average	St.Dev	Selected	Average	Selected	Average	St.Dev	Selected	Average	St.Dev	Selected
Soil Berm	22	120	-	-	-	-	-	-	-	-	-	-	-
Sediment	3	92	-	-	4.5	-	2.0	-	-	0.061	-	-	0.006
Marl	52	97	1.382	1.399	1.382	1.4	1.2	0.170	0.052	0.170	0.012	0.007	0.012
Silt and Clay	45	110	0.187	0.113	0.187	0.9	1.0	0.189	0.048	0.189	0.020	0.007	0.020

Notes:

1. Secondary compression has not been included for the settlement analyses due to the assumed timespan of the surcharge (i.e., 5 years or less).
2. For purposes of the settlement analyses, the existing fill at the shoreline and the sediment within the lake have been considered to be the same “Sediment” material. Properties of the Sediment layer are assumed to be the same as the Outboard Area of the East Wall. It is noted that since the 3-ft thickness of this layer is small relative to the thicknesses of the underlying Marl and Silt and Clay strata (52 and 45 ft, respectively), the parameters of the Sediment are expected to have a minimal impact on the calculation results.
3. The thickness of Marl in RA-E typically varies from 50-55 ft bgs, based on borings OL-SB-70130, OL-SB-70131, OL-SB-70132, and OL-SB-70133.
4. The thickness of Silt and Clay is assumed to extend to 100 ft bgs. Due to stress dissipation effects, it is expected that the thickness of Silt and Clay beyond 100 ft bgs will not have a significant impact on the calculated settlement.
5. St. Dev = Standard Deviation

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Table 3. Summary of Slope Stability Analysis Results

Cross Section	Surcharge Height (ft)	Case Analyzed	Calculated FS	Target FS	Meets Target?	Figure #
E1	18.5	Step 2 – Post-Dredging	1.32	1.35	No[1]	3
E2	18.5	Step 2 – Post-Dredging	1.23	1.35	No[1]	4
E3	18.5	Step 2 – Post-Dredging	1.69	1.35	Yes	5
E4	18.5	Step 2 – Post-Dredging	1.57	1.35	Yes	6
E5[2]	18.5	Step 2 – Post-Dredging	0.92	1.35	No[1]	7
E6	18.5	Step 2 – Post-Dredging	1.57	1.35	Yes	8
E1	10.5	Step 2 – Post-Dredging	1.25	1.35	No[1]	9
E5[2]	10.5	Step 2 – Post-Dredging	1.18	1.35	No[1]	10

Notes:

1. In several cases, the calculated FS value does not satisfy the target FS value of 1.35. If the surcharge is selected as the remediation option, additional analyses will be required to evaluate ways to increase the calculated FS values (e.g., increased surcharge width, in-situ soil densification, and staged construction).
2. For Section E5, the shoreline is close to the railroad and therefore, the surcharge needs to be placed on part of the railroad.
3. As discussed in the text, local stability of the surcharge and bearing capacity were not considered. Due to the height of the surcharge, for some sections, these local stability mechanisms may be more critical than slip through the railroad. If the surcharge is selected as the remediation option, additional analyses will need to be performed to evaluate these potential slip modes.

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Table 4. Summary of Settlement Analysis Results: 18.5-ft Surcharge

Settle3D File	Case Description[1]	Marl			Silt/Clay			5-year Settlement (ft)[3]	Full Consol. Settlement (ft)[4]	5-ft Settlement Time (yr)[5]
		Ccε	Crε	Cv (ft^2/day)	Ccε	Crε	Cv (ft^2/day)			
Section E1	Average values[6]	0.170	0.012	1.382	0.189	0.020	0.187	5.1	5.5	4.3
Section E1_M+SD	Mean plus one standard deviation	0.222	0.019	1.382	0.238	0.028	0.187	6.5	7.0	1.3
Section E1_M-SD	Mean minus one standard deviation	0.117	0.005	1.382	0.141	0.013	0.187	3.7	4.0	[7]
Section E1_HighCV	Higher Cv value (i.e., better drainage)	0.170	0.012	2.074	0.189	0.020	0.280	5.3	5.5	2.9
Section E1_LowCV	Lower Cv value (i.e., worse drainage)	0.170	0.012	0.691	0.189	0.020	0.093	4.6	5.5	8.6

Notes:

1. These cases represent a sensitivity analysis to evaluate the effect of variability in the consolidation parameters.
2. Because the sediment layer is relatively thin, the sensitivity of the sediment layer was not evaluated.
3. This column represents the total settlement calculated underneath the surcharge after 5 years.
4. This column represents the calculated full consolidation settlement (i.e., 100% primary settlement). It is noted that the time to reach full consolidation settlement may require many years (e.g., up to 50 years in some cases). Therefore, this settlement is not likely to be observed in the expected projected timeline.
5. This column represents the approximate time required to estimate a total settlement of 5 ft underneath the surcharge.
6. This is the base case settlement analysis using average consolidation parameters, as shown graphically in Figure 9.
7. The calculated full consolidation settlement is approximately 4 ft. The calculation indicates that a total primary settlement of 5 ft cannot be reached.

Table 5. Summary of Settlement Analysis Results: 10.5-ft Surcharge

Settle3D File	Case Description[1]	Marl			Silt/Clay			5-year Settlement (ft)[3]	Full Consol. Settlement (ft)[4]	3-ft Settlement Time (yr)[5]
		Ccε	Crε	Cv (ft^2/day)	Ccε	Crε	Cv (ft^2/day)			
Section E1	Average values[6]	0.170	0.012	1.382	0.189	0.020	0.187	3.0	3.2	5.0
Section E1_M+SD	Mean plus one standard deviation	0.222	0.019	1.382	0.238	0.028	0.187	3.7	4.0	1.5
Section E1_M-SD	Mean minus one standard deviation	0.117	0.005	1.382	0.141	0.013	0.187	2.4	2.6	Infinite[7]
Section E1_HighCV	Higher Cv value (i.e., better drainage)	0.170	0.012	2.074	0.189	0.020	0.280	3.1	3.2	4.0
Section E1_LowCV	Lower Cv value (i.e., worse drainage)	0.170	0.012	0.691	0.189	0.020	0.093	2.6	3.2	17.0

Notes:

1. These cases represent a sensitivity analysis to evaluate the effect of variability in the consolidation parameters.
2. Because the sediment layer is relatively thin, the sensitivity of the sediment layer was not evaluated.
3. This column represents the total settlement calculated underneath the surcharge after 5 years.
4. This column represents the calculated full consolidation settlement (i.e., 100% primary settlement). It is noted that the time to reach full consolidation settlement may require many years (e.g., up to 50 years in some cases). Therefore, this settlement is not likely to be observed in the expected projected timeline.
5. This column represents the approximate time required to estimate a total settlement of 3 ft underneath the surcharge.
6. This is the base case settlement analysis using average consolidation parameters, as shown graphically in Figure 9.
7. The calculated full consolidation settlement is approximately 2.6 ft. The calculation indicates that a total primary settlement of 3 ft cannot be reached.

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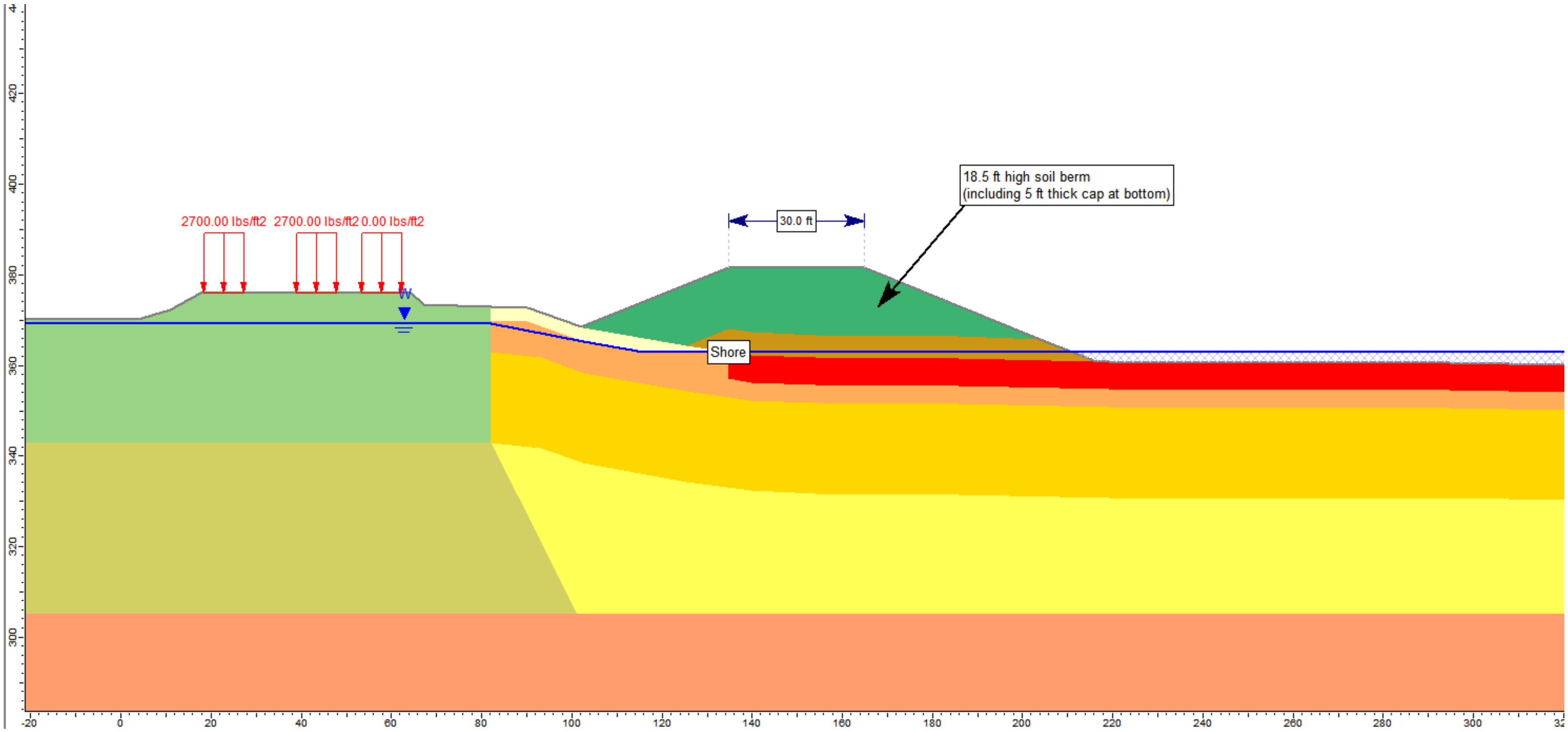

Figure 1A. Stage 1 of Surcharge Placement Sequence: Construction of Soil Berm at Year 0

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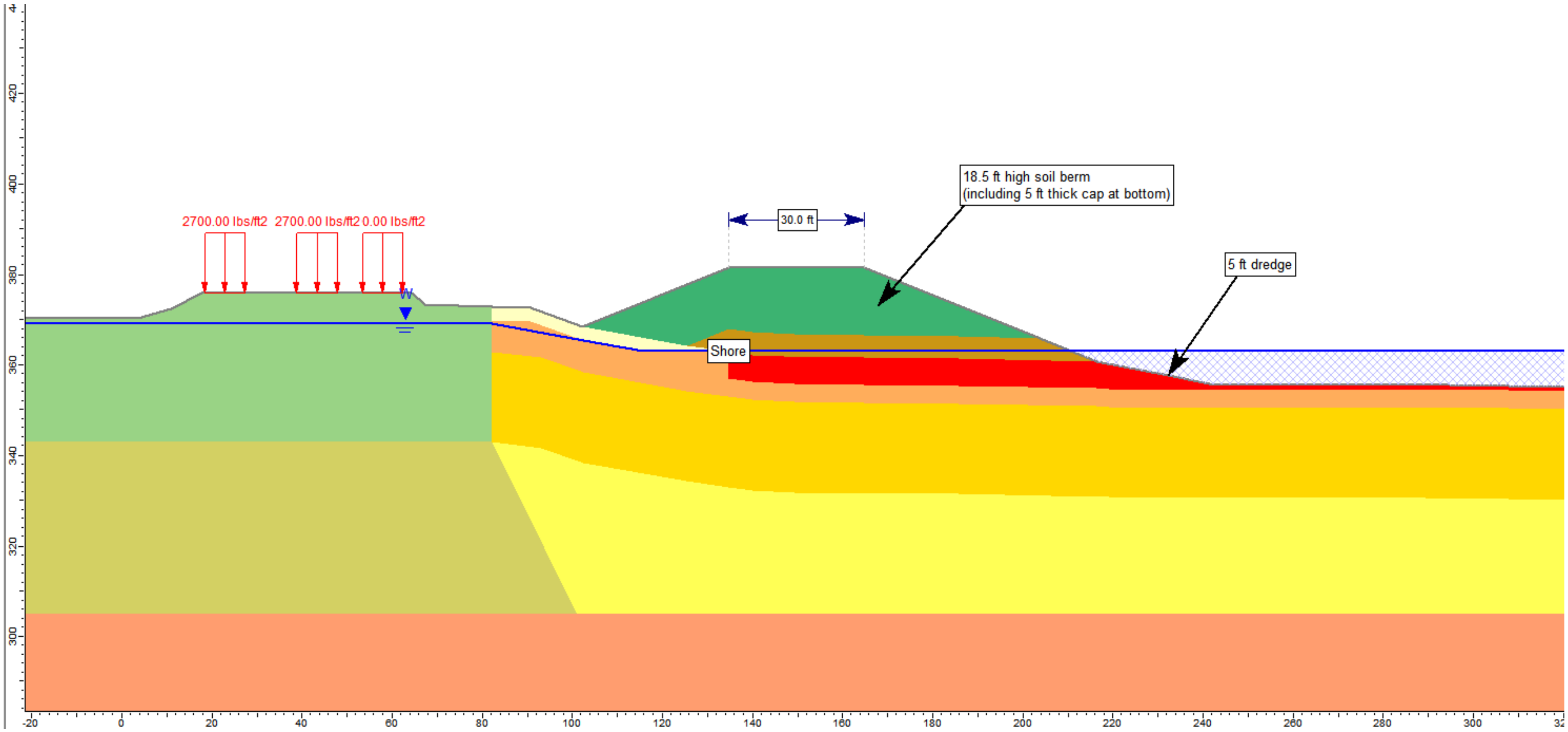

Figure 1B. Stage 2 of Surcharge Placement Sequence: Dredging in front of Soil Berm at Year 2
Note: Settlement that will occur under the soil berm is not illustrated in the figure.

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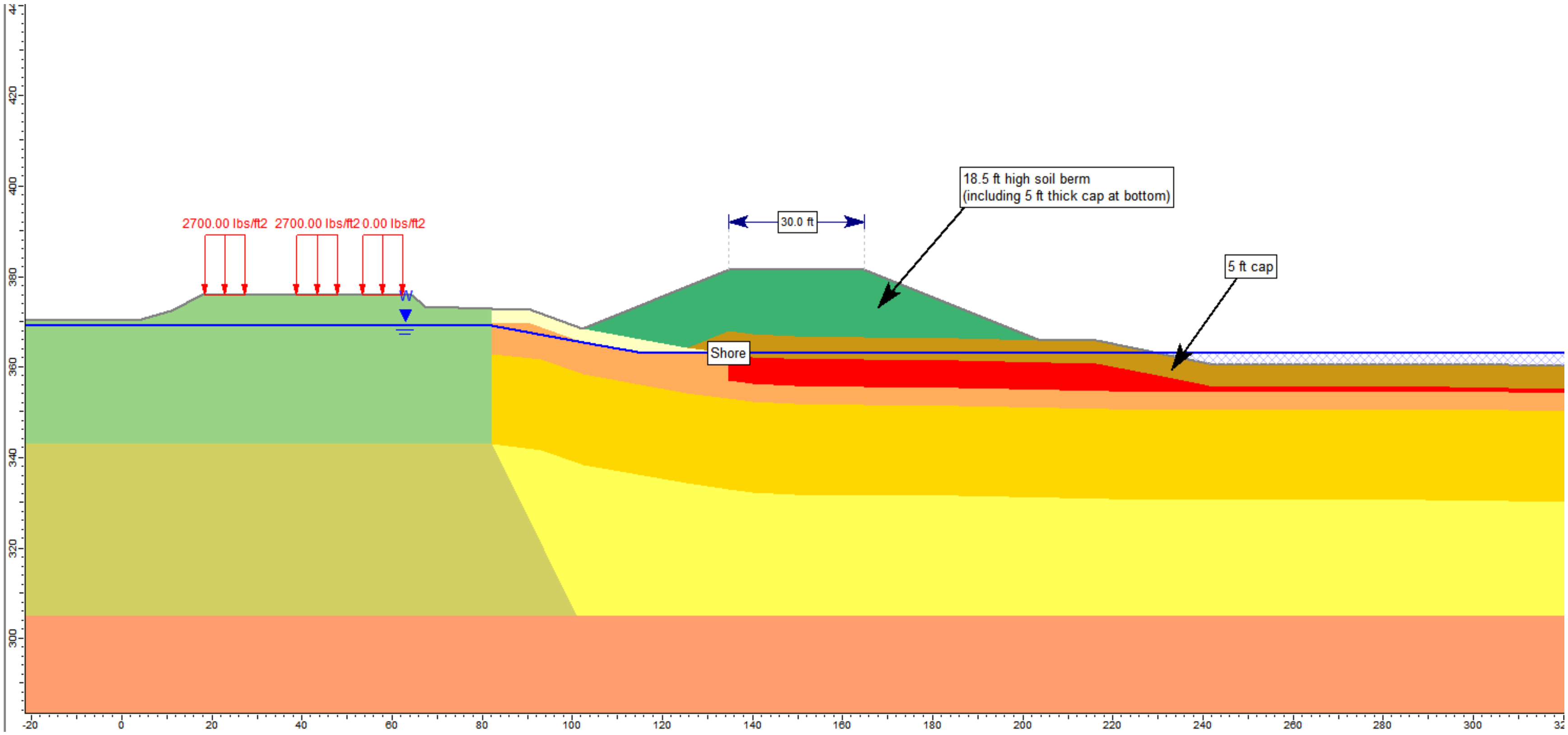

Figure 1C. Stage 3 of Surcharge Placement Sequence: Construction of Cap in front of Soil Berm at Year 2

Note: Settlement that will occur under the soil berm is not illustrated in the figure.

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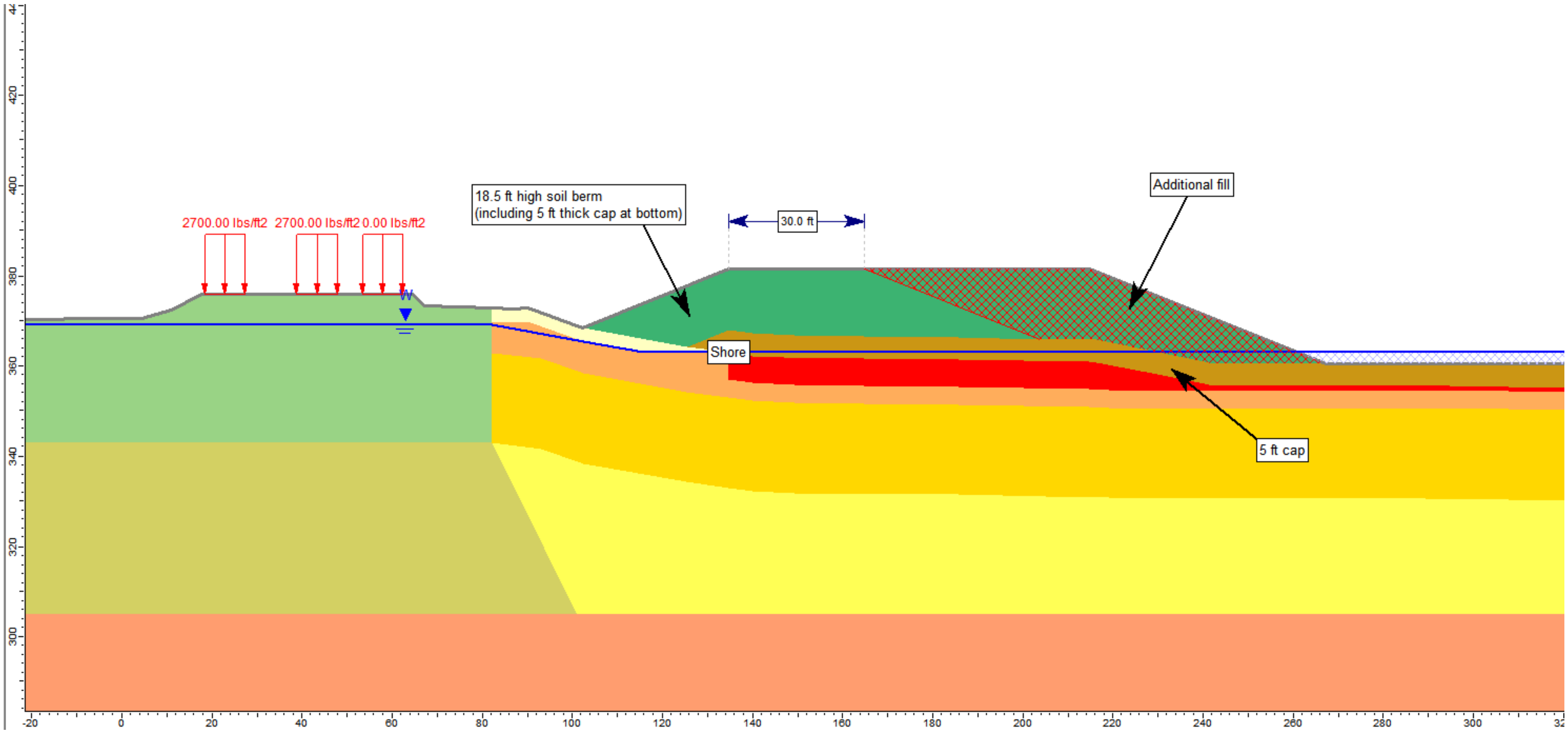

Figure 1D. Stage 4 of Surcharge Placement Sequence: Placement of Additional Fill in front of Soil Berm at Year 2

Note: Settlement that will occur under the soil berm is not illustrated in the figure.

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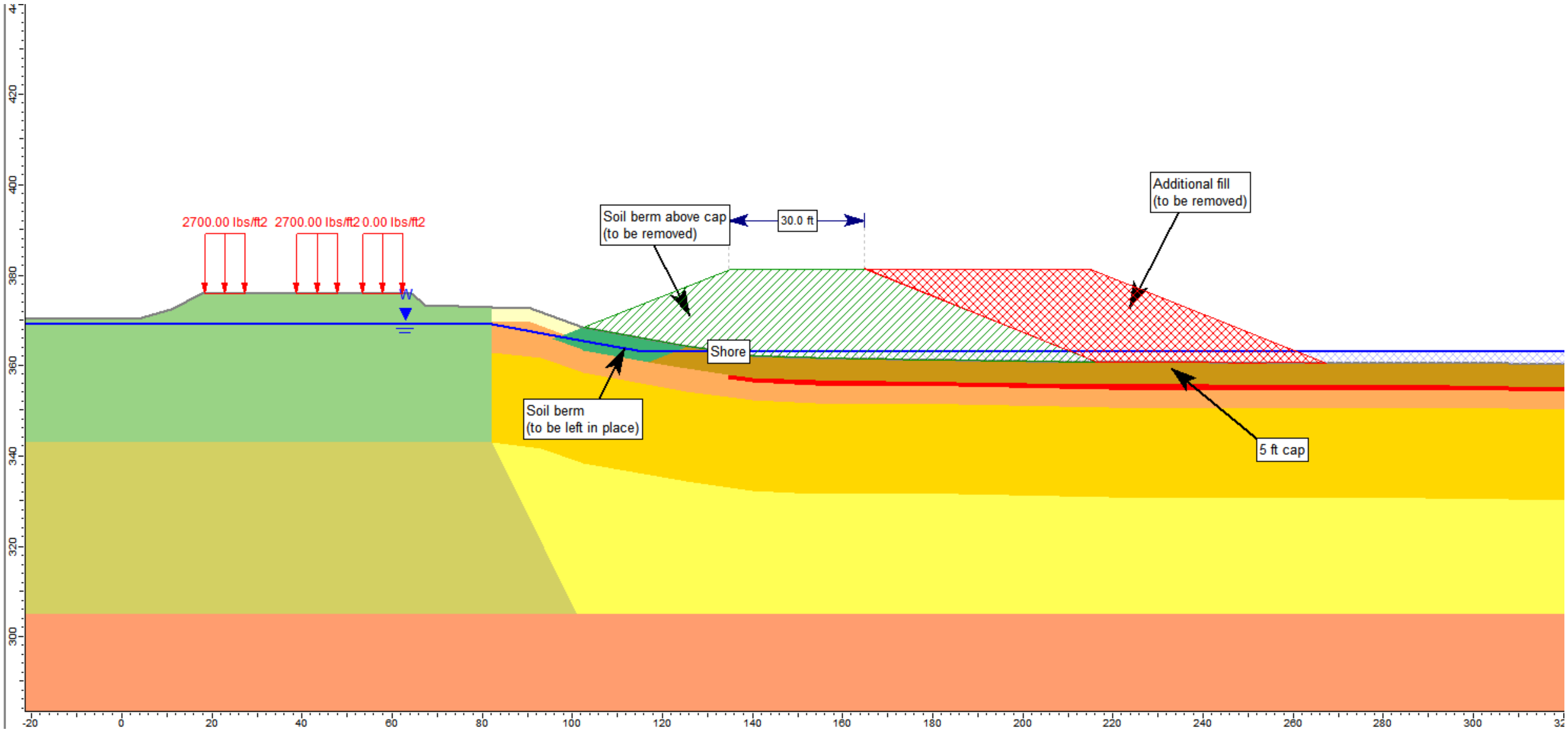

Figure 1E. Stage 5 of Surcharge Placement Sequence: Removal of Soil Berm at End of Year 5

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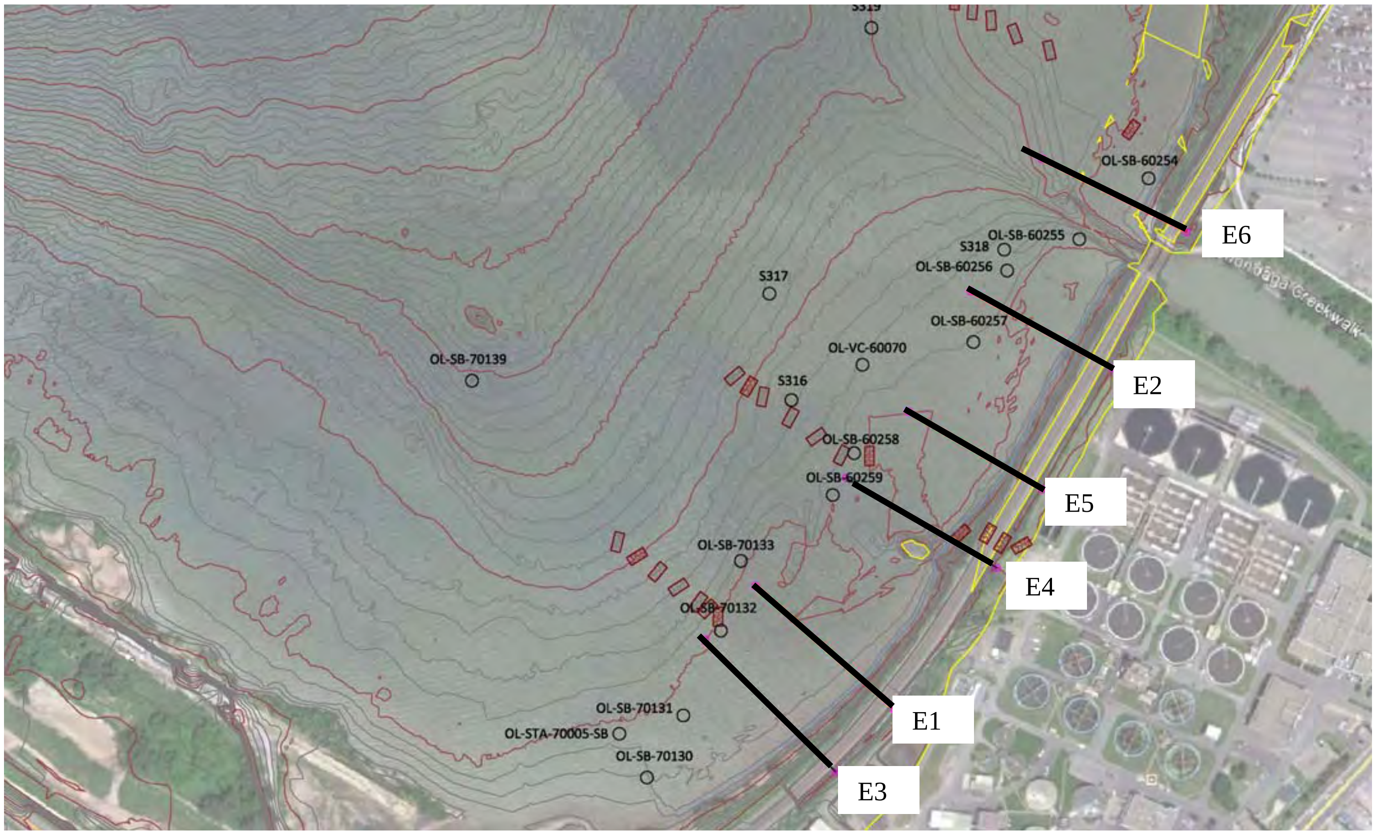

Figure 2. Locations of Selected Six Cross Sections for Stability and Available Soil Borings

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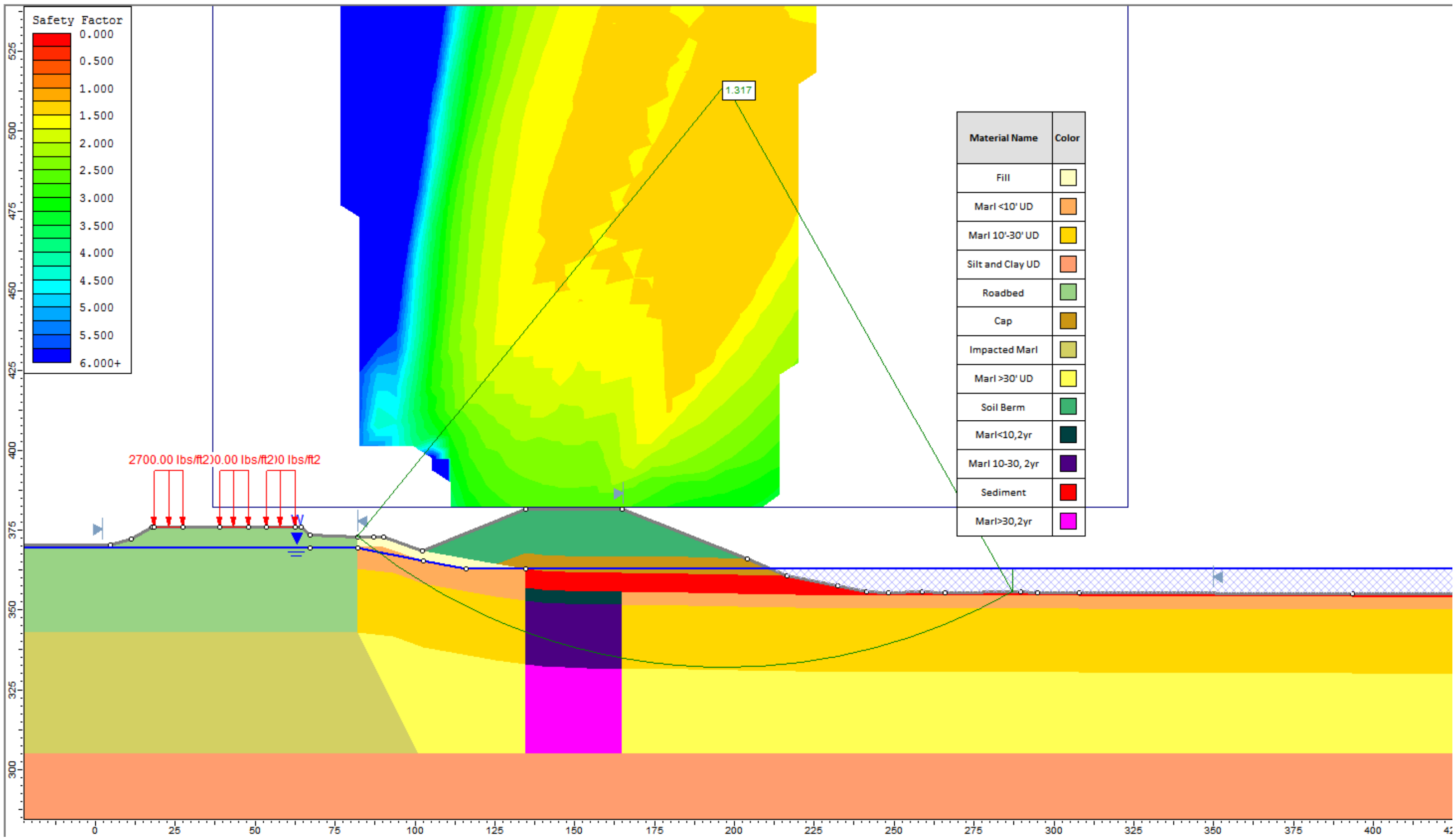

Figure 3. Slope Stability Analysis Results – 18.5-ft Surcharge, Section E1, Stage 2

Written by: **Joseph Sura** Date: **4/29/2014** Reviewed by: **Ming Zhu/Jay Beech** Date: **4/29/2014**

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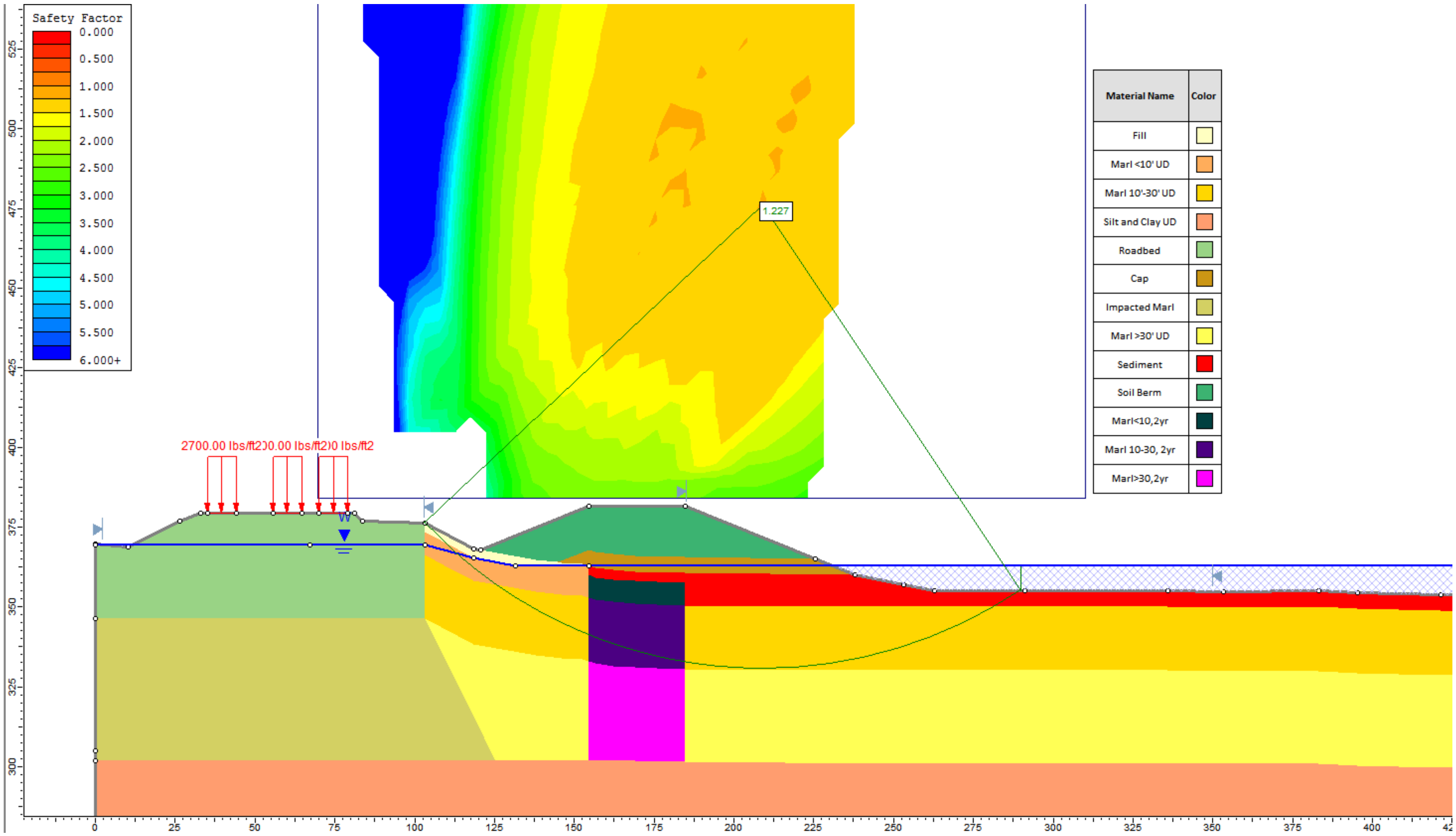

Figure 4. Slope Stability Analysis Results – 18.5-ft Surcharge, Section E2, Stage 2

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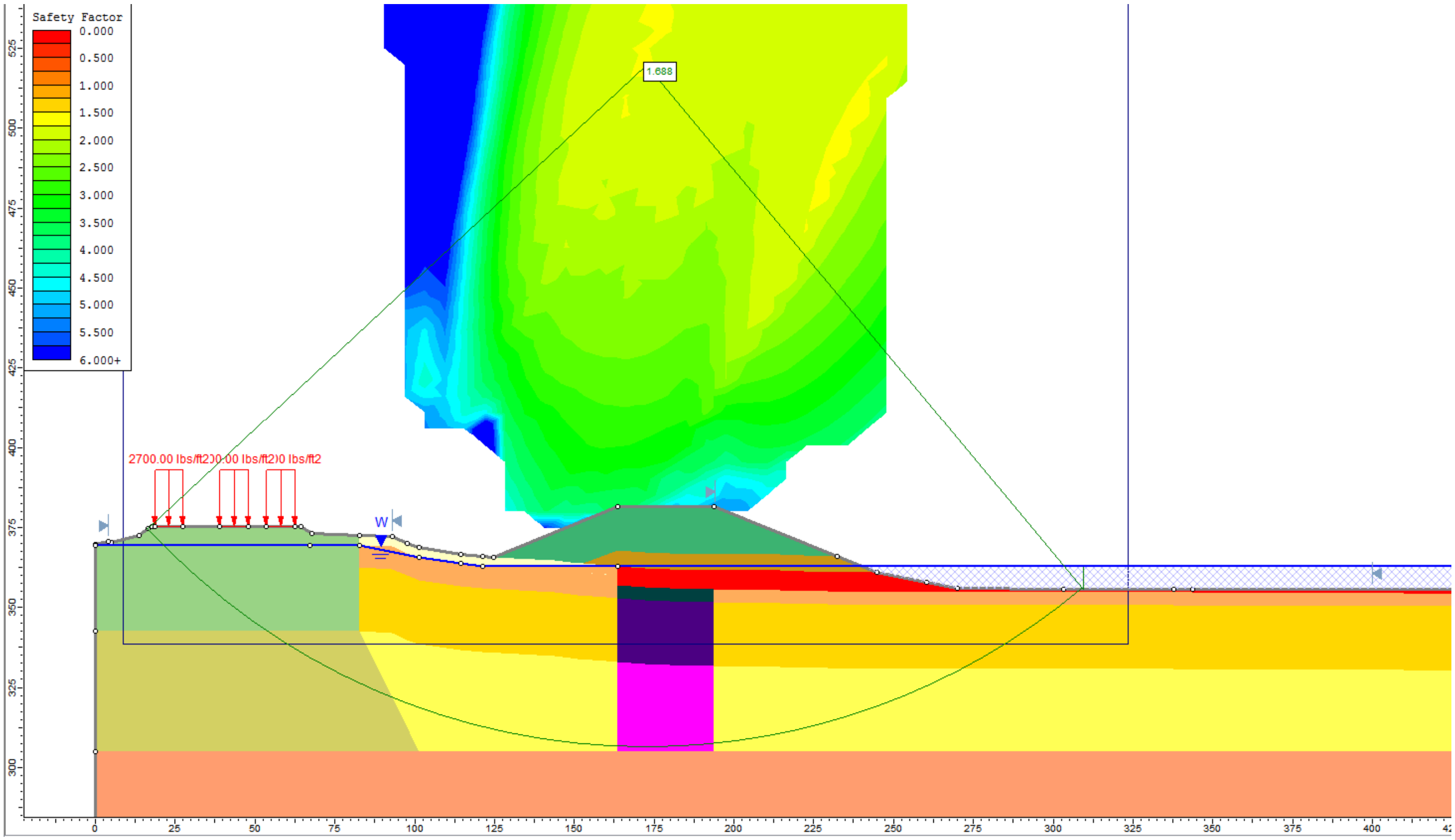

Figure 5. Slope Stability Analysis Results – 18.5-ft Surcharge, Section E3, Stage 2

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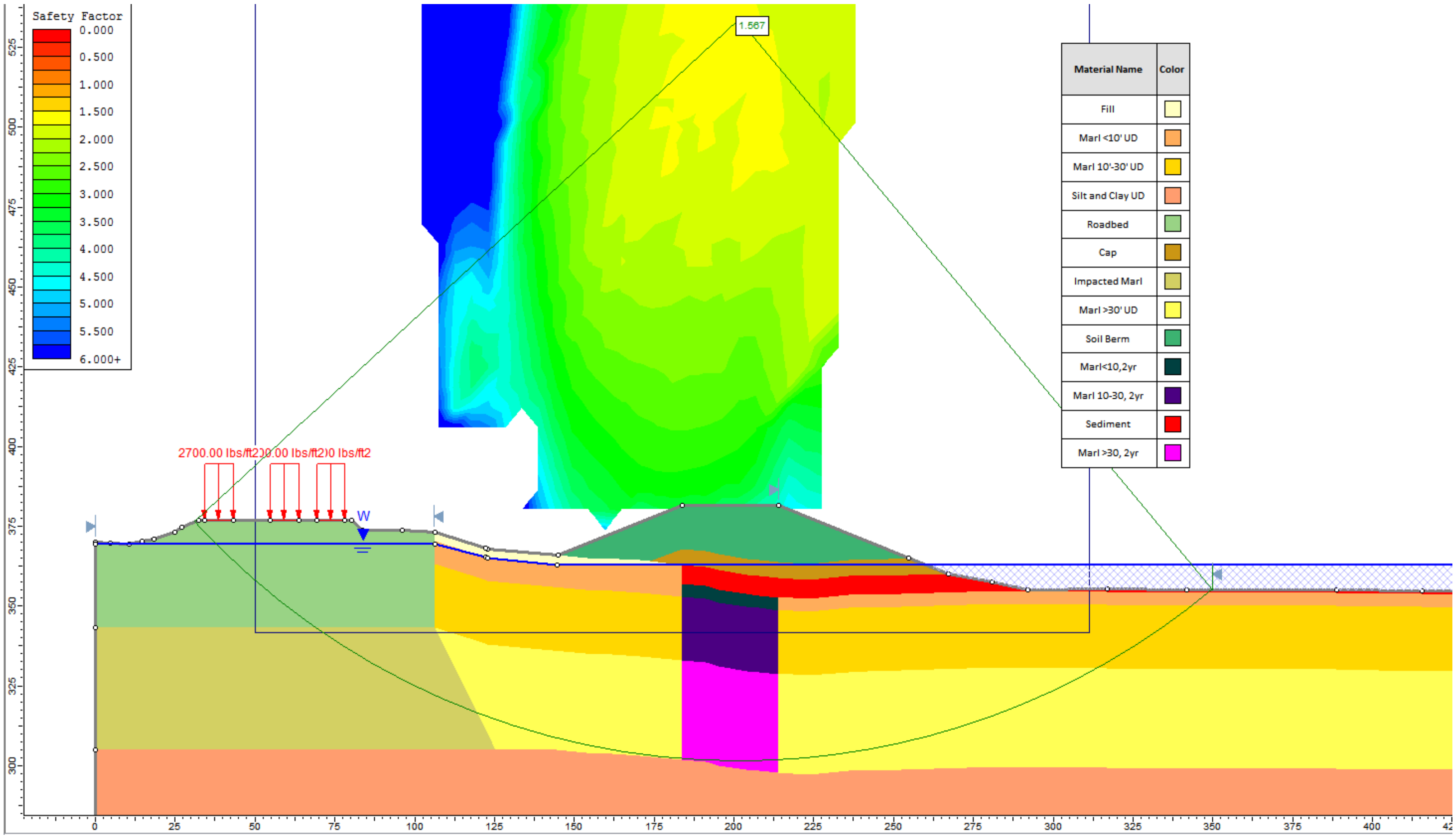

Figure 6. Slope Stability Analysis Results – 18.5-ft Surcharge, Section E4, Stage 2

Written by: **Joseph Sura** Date: **4/29/2014** Reviewed by: **Ming Zhu/Jay Beech** Date: **4/29/2014**

Client: **Honeywell** Project: **RA-E Shoreline Surcharge** Project No.: **GD5453** Task No.: **03**

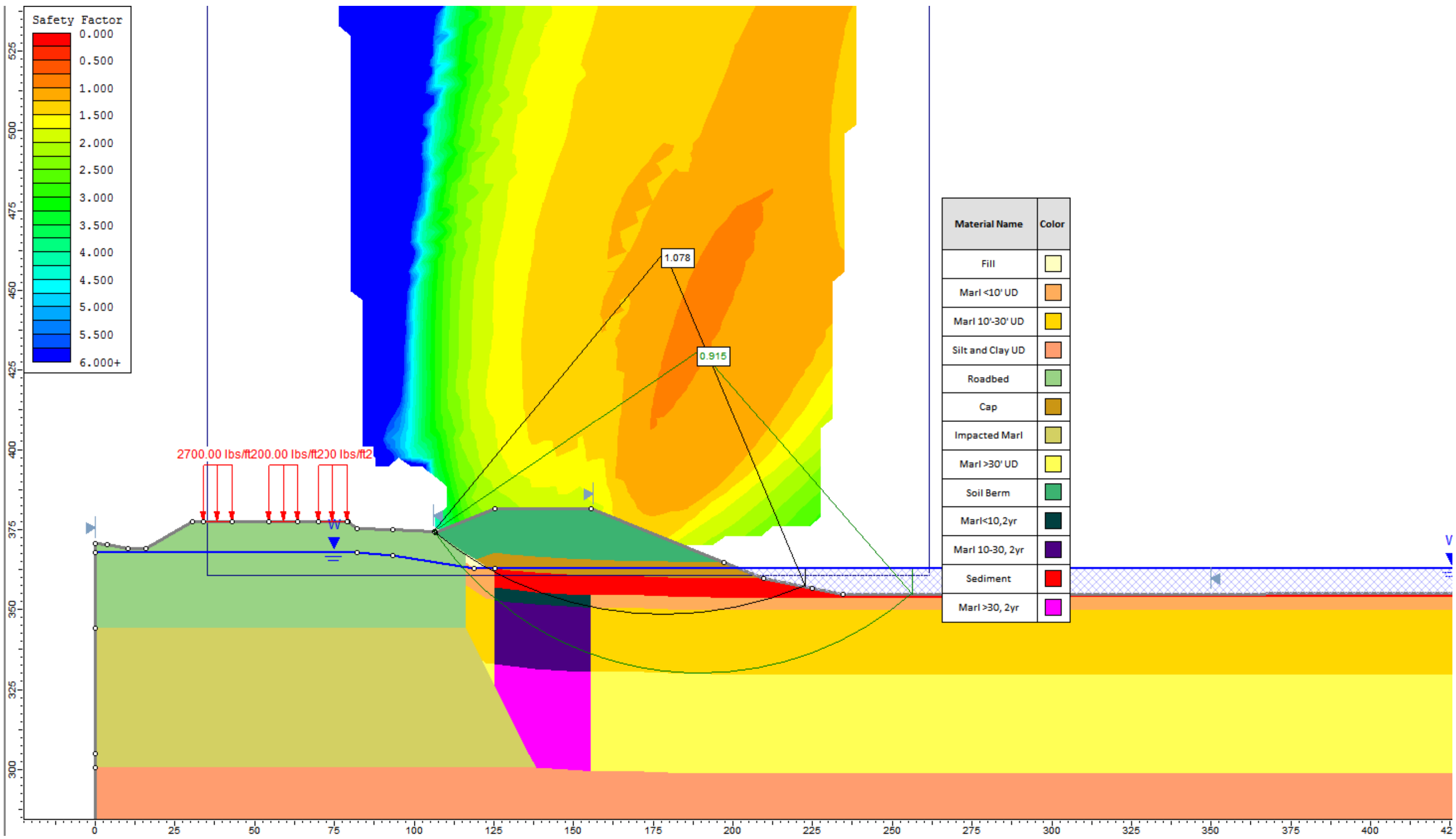

Figure 7. Slope Stability Analysis Results – 18.5-ft Surcharge, Section E5, Stage 2

Written by: **Joseph Sura** Date: **4/29/2014** Reviewed by: **Ming Zhu/Jay Beech** Date: **4/29/2014**

Client: **Honeywell** Project: **RA-E Shoreline Surcharge** Project No.: **GD5453** Task No.: **03**

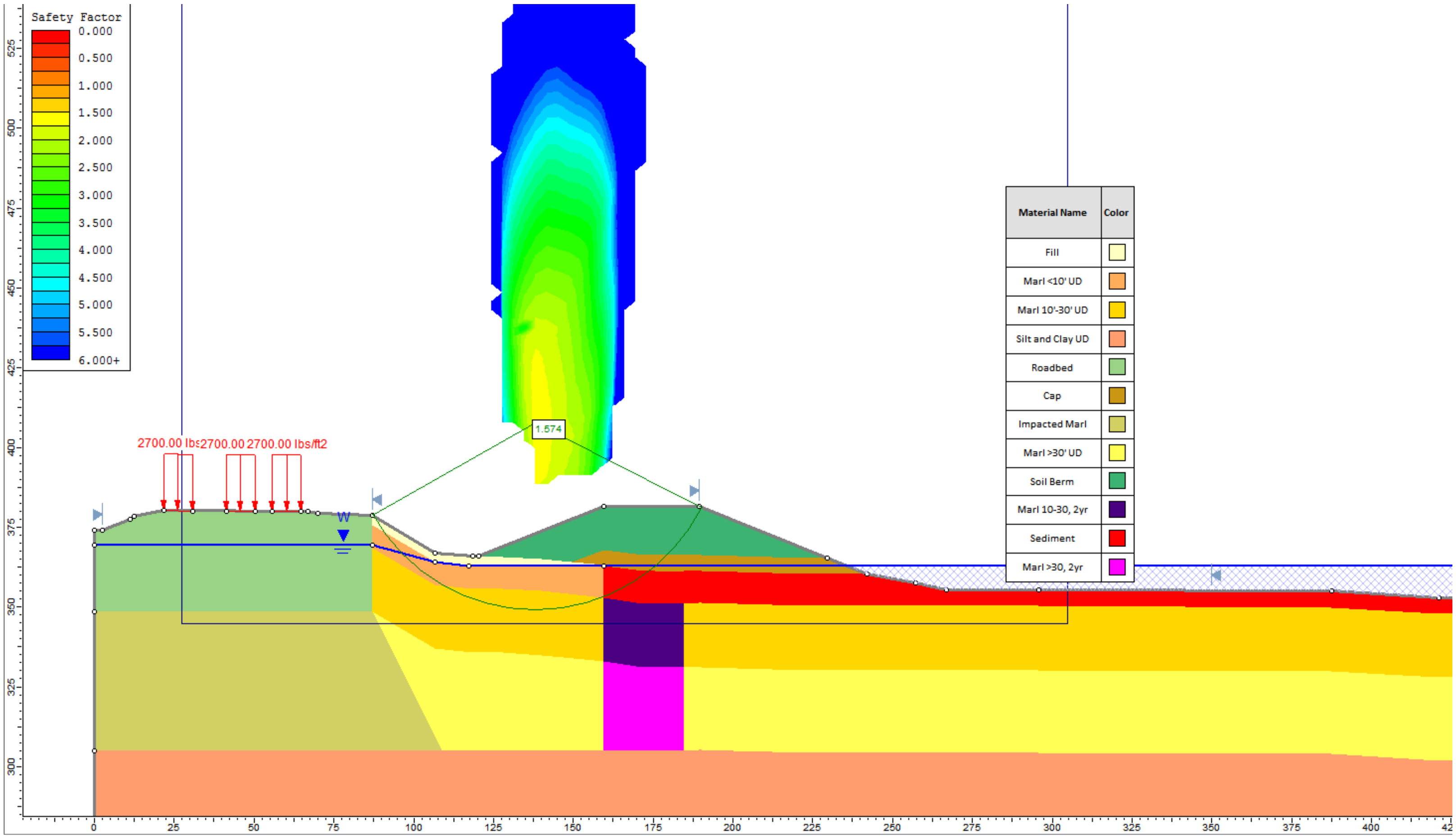

Figure 8. Slope Stability Analysis Results – 18.5-ft Surcharge, Section E6, Stage 2

Written by: **Joseph Sura** Date: **4/29/2014** Reviewed by: **Ming Zhu/Jay Beech** Date: **4/29/2014**

Client: **Honeywell** Project: **RA-E Shoreline Surcharge** Project No.: **GD5453** Task No.: **03**

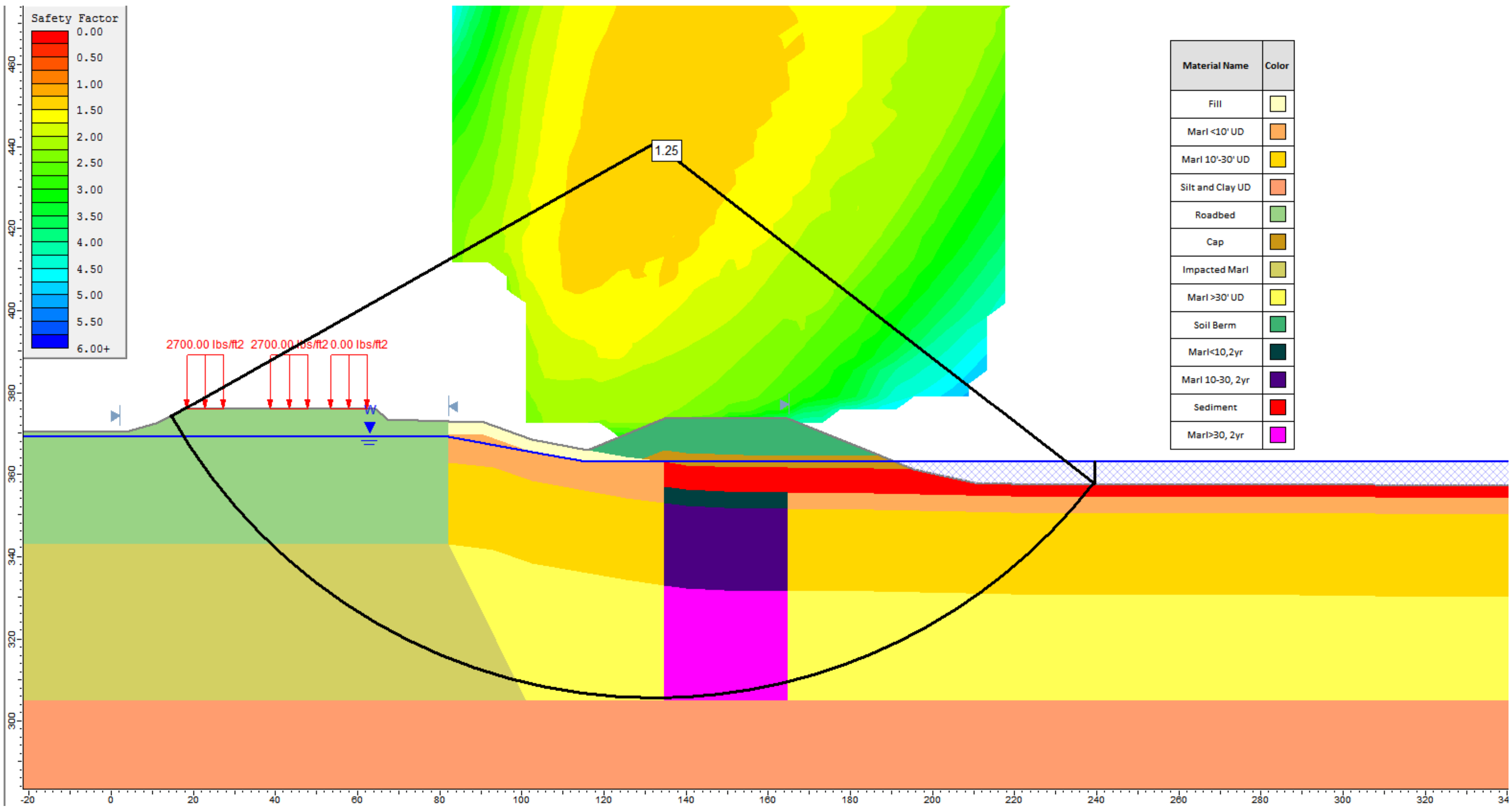

Figure 9. Slope Stability Analysis Results – 10.5-ft Surcharge, Section E1, Stage 2

Written by: **Joseph Sura** Date: **4/29/2014** Reviewed by: **Ming Zhu/Jay Beech** Date: **4/29/2014**

Client: **Honeywell** Project: **RA-E Shoreline Surcharge** Project No.: **GD5453** Task No.: **03**

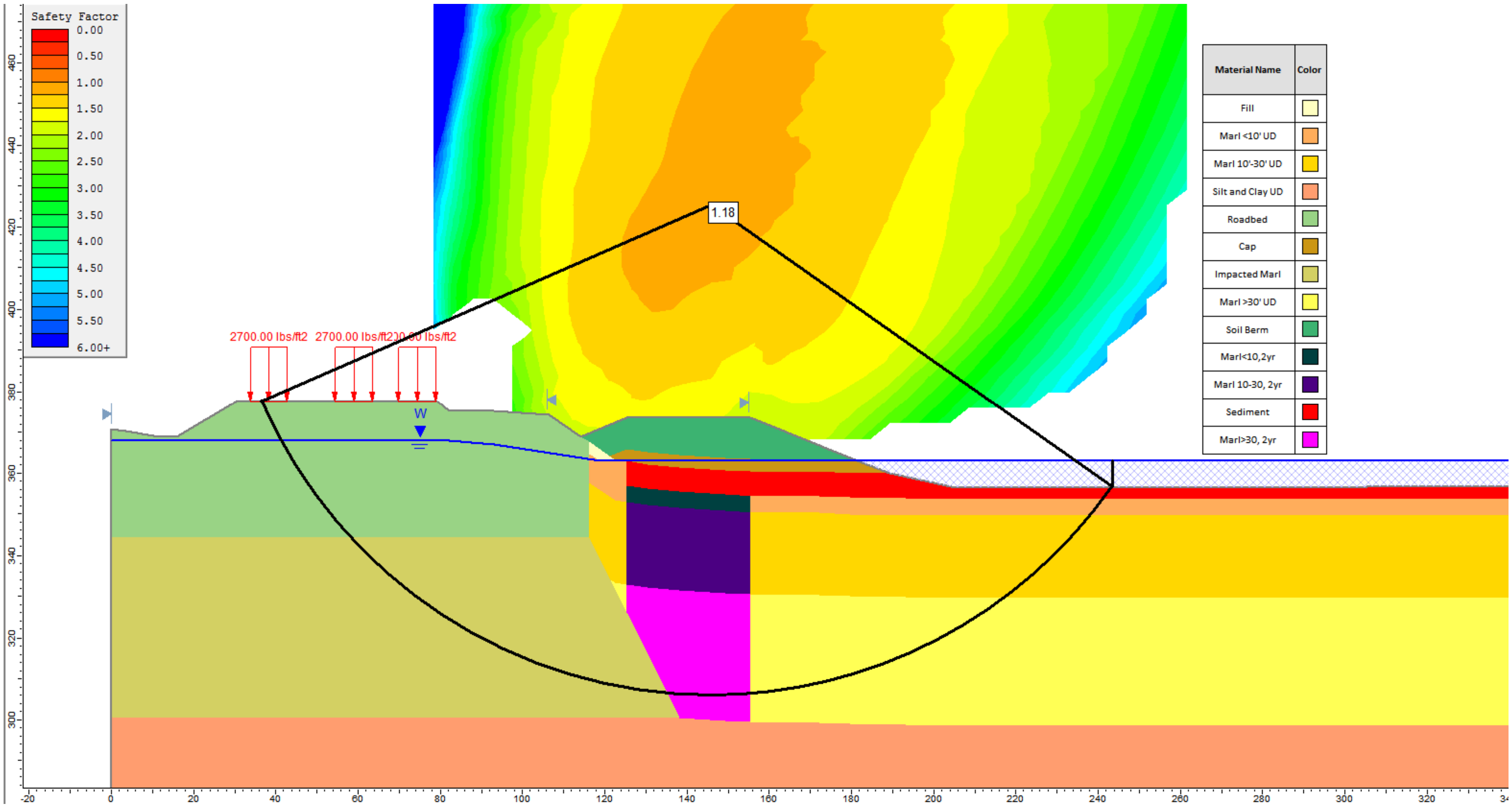

Figure 10. Slope Stability Analysis Results – 10.5-ft Surcharge, Section E5, Stage 2

Written by: Joseph Sura Date: 4/29/2014 Reviewed by: Ming Zhu/Jay Beech Date: 4/29/2014

Client: Honeywell Project: RA-E Shoreline Surcharge Project No.: GD5453 Task No.: 03

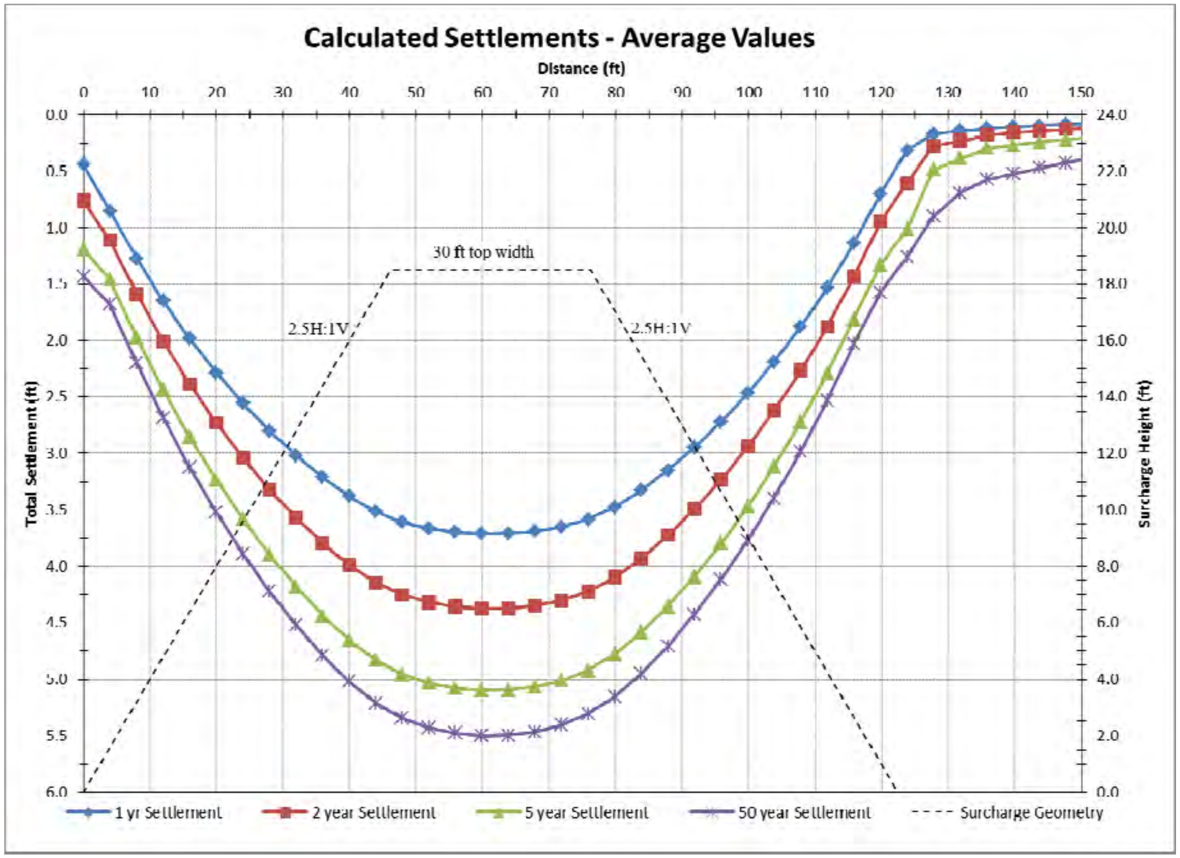

Figure 11. Calculated Primary Consolidation Settlement at Different Times using Average Consolidation Parameters: 18.5- ft Surcharge
Note: The distance is measured from the toe of the surcharge inside the lake (i.e., 0 ft indicates the toe of the surcharge in the lake and 122.5 ft indicates the toe of the surcharge on land).

Written by: **Joseph Sura** Date: **4/29/2014** Reviewed by: **Ming Zhu/Jay Beech** Date: **4/29/2014**

Client: **Honeywell** Project: **RA-E Shoreline Surcharge** Project No.: **GD5453** Task No.: **03**

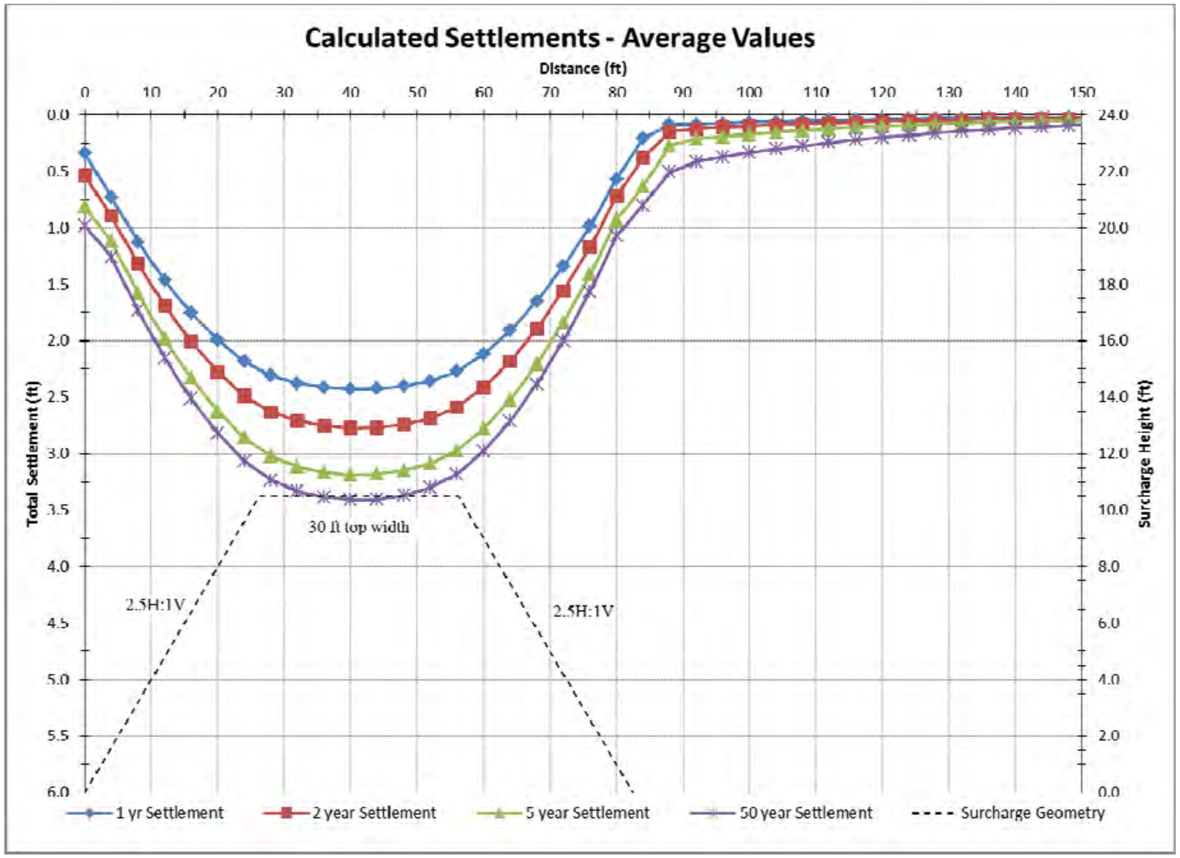

Figure 12. Calculated Primary Consolidation Settlement at Different Times using Average Consolidation Parameters: 10.5- ft Surcharge
Note: The distance is measured from the toe of the surcharge inside the lake (i.e., 0 ft indicates the toe of the surcharge in the lake and 82.5 ft indicates the toe of the surcharge on land).