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## STABILITY ANALYSIS OF THE SLOPE ALONG US-2 BETWEEN EPOUFETTE BAY AND THE CUT RIVER BRIDGE

Stephanie Watts-Garcia  
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**STABILITY ANALYSIS OF THE SLOPE ALONG US-2  
BETWEEN EPOUFETTE BAY AND THE CUT RIVER  
BRIDGE**

By  
Stephanie Watts-Garcia

A REPORT

*Submitted in partial fulfillment of the requirements for the degree of*

**MASTER OF SCIENCE**

*In Civil Engineering*

MICHIGAN TECHNOLOGICAL UNIVERSITY

2014

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This report has been approved in partial fulfillment of the requirements for the Degree of MASTER OF SCIENCE in Civil Engineering.

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## Acknowledgements

First, I would like to thank my advisor, Dr. Stan Vitton for the guidance and support throughout this research, this project would not be possible otherwise.

I am grateful for my committee, Dr. Evan Kane and Dr. Thomas Oommen for giving me helpful feedback and advice to make a more thorough analysis.

I give many thanks to many of my graduate colleagues especially, Christopher King and Bonnie Zwissler whom helped me collect samples, conduct testing as well as critiquing my work—this research would be at a dead end without all their help.

I also appreciate the previous testing and data collected through MDOT.

Lastly, I want to thank my friends and family for all the moral support throughout this graduate research.

Thank you all for your valued assistance, it is much appreciated and I could not have done this without your help.

## **Abstract**

Recently, water was observed flowing from a section of steep slope along US-2 near St. Ignace, Michigan in addition to soil sloughing in the area where the water is flowing from the slope. An inspection of the area also showed the presence of sinkholes. The original construction drawing for US-2 also indicated that sinkholes were present in this area prior to road construction in 1948. An investigation was conducted to determine the overall stability of the slope. The slope consists primarily of aeolian sand deposits. Laboratory testing determined the shear strength of the slope material to have a friction angle around 30°, which is also the slope angle. Thus, the slope is at its maximum angle for stability—however, the slope is also heavily wooded which provides additional support to the slope. Although the area surrounding the water flow has been sloughing, the remaining slope remains intact.

## **1.0 Introduction**

In late March 2012 a routine traffic stop was made along US-2 near Epoufette Bay, Michigan about a mile west of the Cut River Bridge. This site is also located about thirty miles west of Saint Ignace, MI (see

Figure 1). At this location the highway is located on a steep bluff overlooking Lake Michigan, about 100 feet below the level of the highway. The police officer noticed a high rate of water shooting out of the slope about midway down and reported it to the regional Michigan Department of Transportation (MDOT) office shortly afterwards. MDOT officials examined the site but did not find the reported high flow water condition as noted by the police officer but did find a relatively large slough [Slough May 2012] about midway down where the water was coming out of the slope. MDOT personnel also noted the presence of sinkholes [Sinkhole] on top of the bluff along both sides of the highway in the vicinity of the slough. MDOT then contacted Michigan Tech and requested that a site visit be made to make an initial site investigation. This site investigation was made by Dr. Stan Vitton in May 2012 who also noted a large slough with water emanating from the slough as well as number of sinkholes in the immediate vicinity of the highway in the location of the slough.

The highway was constructed in 1948 and consists of a concrete pavement with asphalt overlays. A review of the construction drawing for this section of highway above the slough also indicated the presence of sinkholes. A section of the 1948 construction drawing is shown in Figure 2. A total of 13 sinkholes can be seen on this section of highway just about the area where the water was observed coming out of the slough area. During the summer of 2012 MDOT conducted field operations that included site drilling, falling weight deflectometer, ground penetrating radar and surveying to investigate the area.

While some of the sinkholes (noted in the 1948 drawings) along the sides of US-2 are still present, none of the drilling or GPR indicated the presence of sinkholes under the pavement. All of the data collected by MDOT at the site was sent to Michigan Tech for further analysis.

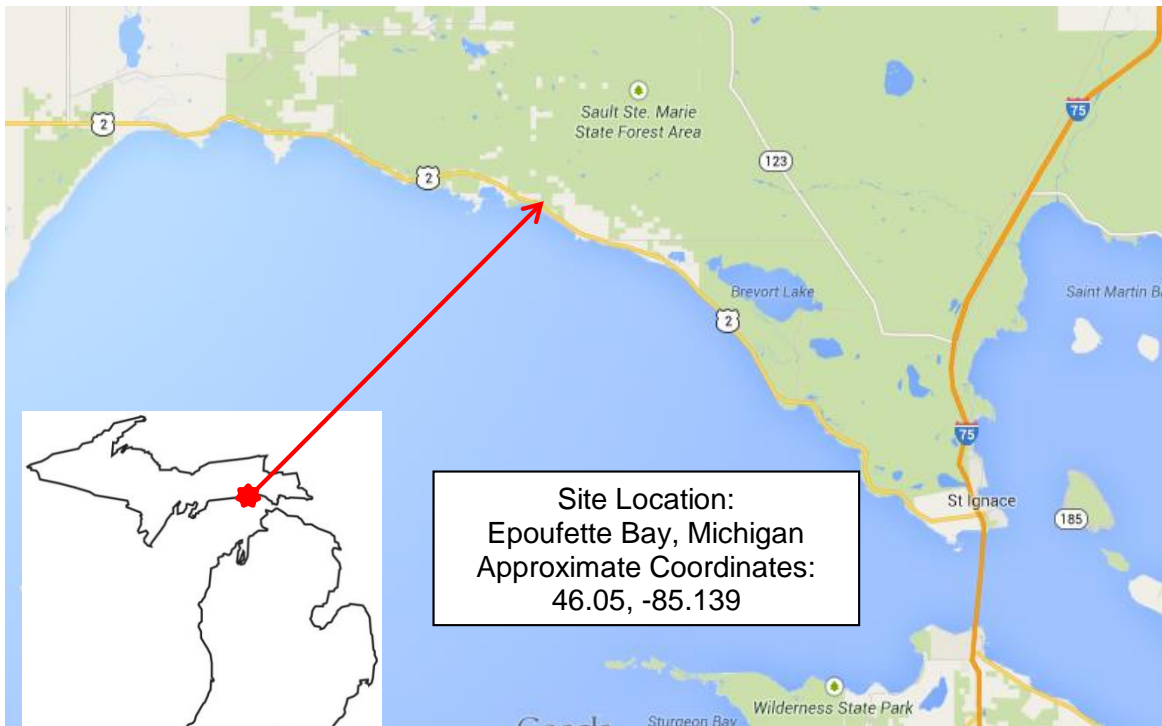


Figure 1 Site Location Epoufette Bay, Michigan.

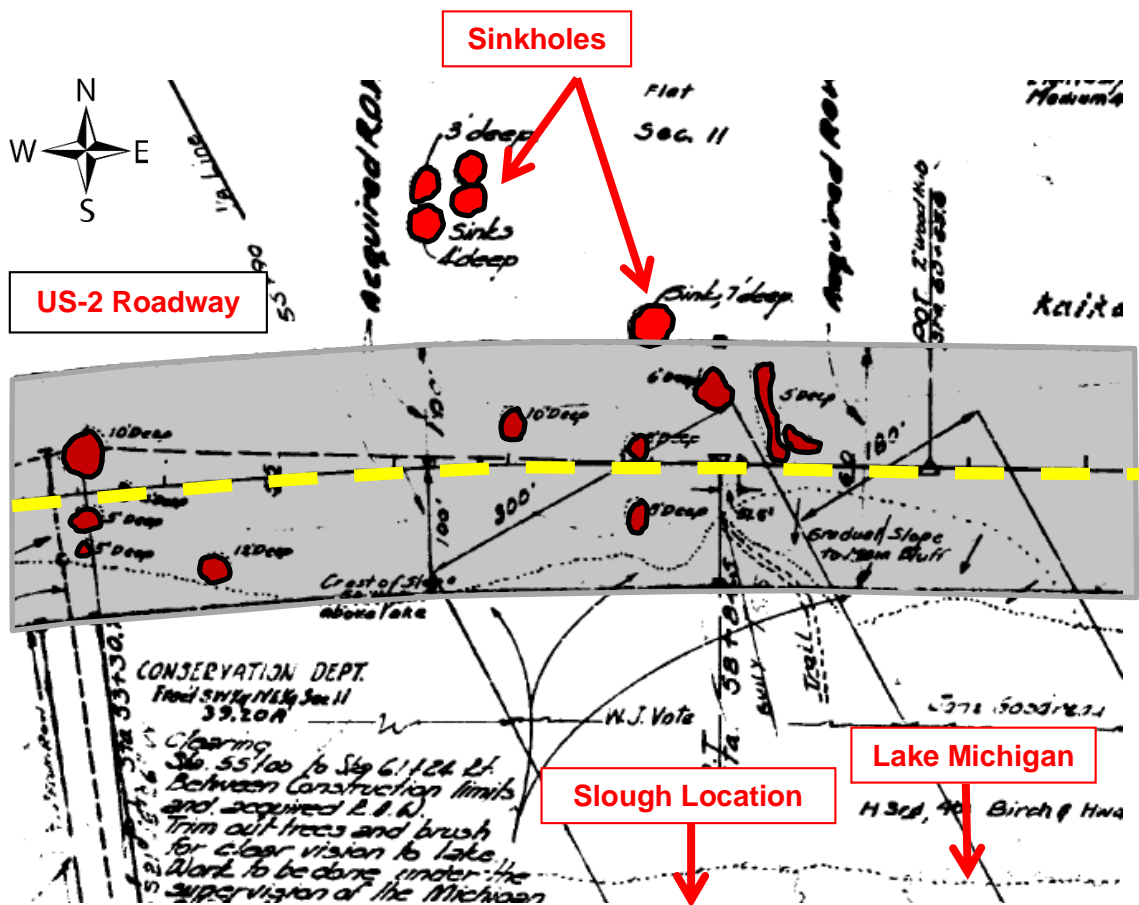


Figure 2 Construction drawing for the section of US-2 above the slough; sinkholes are indicated in red.

## **1.1 Project Scope**

The aim of this project was to provide a preliminary assessment of the overall stability of the slope. To accomplish this, a site visit was made in the summer of 2013. All of the data collected from the MDOT investigations was also obtained. This data includes nine drill holes, Ground Penetrating Radar (GPR), Falling Weight Deflectometer (FWD) and survey transections from the highway down to the base of the slope. Nine boreholes were drilled at the site to a depth of 21.5 feet. Since that lacks the depth to understand the natural soils, one drill hole was drilled to a depth of 96.5 feet [Truth Boring Data]. The main objective of the drilling operations was to assess if any sinkholes were present under the highway pavement structure.

## **1.2 Investigation Outline**

The site assessment included an investigation of the site's geology, which included both the bedrock geology as well as the glacial history of the site. The primary intent of the geological investigation was to establish the crucial stratigraphy of the site. This way an analysis of the slope stability as well as an explanation for the formation of the sinkholes became possible. An additional component of the investigation was an analysis if adjacent areas with similar geology also might have had landslides and sinkhole development.

The main tools that were integrated into this analysis consisted of two Rocscience software programs. The initial model was created in Slide (version 6.0), a limit-equilibrium slope stability analysis program which was then exported into Phase2 (version 8.0), a 2D elasto-plastic finite element stress program that can be used to assess slope stability. The primary emphasis of the Rocscience analytical study was concentrated on the stability of the slope.

The investigation of the sinkholes was limited to an assessment of the glacial and bedrock geology. While no analytical methods were used to assess the development of sinkholes, observations made concerning the geology of the site were proposed.

## **2.0 Site Assessment**

In order to assess the site, the area of interest (AOI) was investigated in the vicinity of the sinkholes and slough along US-2, east of Epoufette Bay. Since the site had already been documented as an area prone to sinkholes, further research was performed to help understand the issue that is occurring along the roadway. This section of the report will start with the field inspection followed by the glacial geology and will finish with the mechanics behind sinkhole formation.

### **2.1 MDOT Site Analysis**

To investigate the possibility of unstable ground underneath this section of US-2, MDOT conducted GPR and FWD. Based on these tests, MDOT then conducted nine boreholes in areas that were suspect from the GPR and the FWD results and surveyed six cross-sections perpendicular to the highway and slope. The site consists mostly of fine sands, which vary from very loose to dense sands. Borehole #2 had a depth of nearly 97 feet. None of the nine MDOT boreholes encountered bedrock nor groundwater. Based on water well data in the local area bedrock was estimated to be at approximately the elevation of Lake Michigan at an elevation of 577 feet above Mean Sea Level (MSL). The six cross-sections of the area were started at the center of the main depression (zero point), 75', 50' and 25' west of the zero point, 25' east of the zero point, as well as at the center of the artesian well. Truth boring data and ground surveyed cross-sections can be found in Appendix B: Materials from MDOT.

### **2.2 Field Inspection**

As previously mentioned, over a year ago a concern (regarding roadway stability) occurred upon spotting water flowing out of the south slope along US-2. Later site inspections noticed water flowing from the slope. However, it is possible that artesian conditions could occur at this point in the slope, during the spring time. Artesian Conditions are common along the base of the slope between Epoufette Bay to the west and the Cut River Bridge one mile to the east of the AOI. Two examples were noted, on the following page. The first example occurs at the Cut River Bridge where an artesian well is located on the west side of the Cut River between the base of the slope and Lake Michigan. This artesian well is shown in

Figure 3. A second example occur very near to the AOI and is shown in Figure 4.



Figure 3: Artesian well at the Cut River Bridge



Figure 4: Artesian well and creek at the base of the AOI

Upon the site to the AOI, more sinkholes were discovered. A sinkhole approximately 5' in diameter and 1.5'-2' deep has formed on both the North and South side of the roadway [Sinkhole]. Although sinkholes are much more numerous south of the road, it is possible that the sinkhole on the north side was initiated through similar means. As previously implied, the sinkholes off both sides of the roadway are relatively circular. This could mean one of two things: either it is a subsidence sinkhole or it is a sinkhole formed from the process called "rat-holing". Both of these types of sinkholes have somewhat similar mechanisms of development, which will be clarified in section 2.4 Sinkhole Formation.

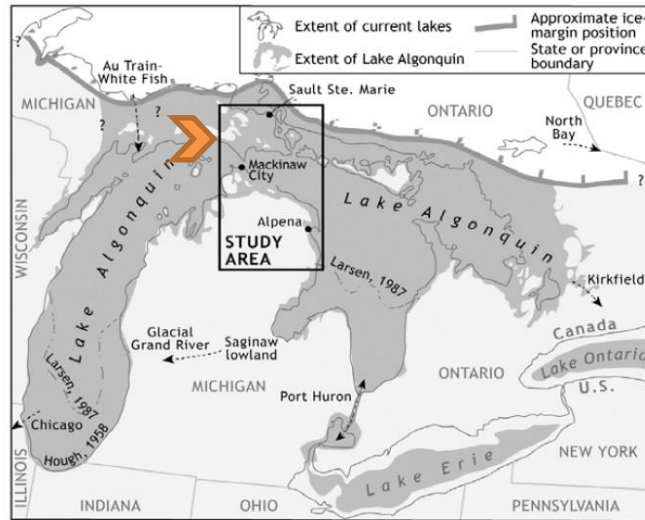
Aside from the sinkhole concern another important issue is the development of the landslide potential. The slope between the highway and Lake Michigan has a very steep gradient, which was measured to be about 31 ° [Ground Survey]. The slough area is about three-quarters of the way down the bluff, starting at an elevation of nearly 670 feet above MSL. After reaching the very top edge of the slough, the soil directly below the slough's escarpment had a slow settlement of sand particles falling from the weight of the observation group. More sand fell in greater amounts as the group moved around the top edge of the slough [Slough June 2013]. It was more than obvious that the land is experiencing some stabilization issues around this area; whether or not this slough is directly related to the sinkholes is unknown. Fortunately, while looking north (towards the road) from the Lake Michigan shoreline; there was no indication of any additional slough or sinkhole formation. The only suggestion of on-going weathering or potential stabilization issues was a small meandering stream that leads to Lake Michigan from the artesian well, located approximately at elevation 634 MSL.

A small water flow was observed coming out of the slough. The water flow was about 6 inches wide by about 3-4 inches deep and can be seen in Appendix D: Visual Walkthrough, [Small Stream 2012, 2013 respectively (Looking South), Stream April 2014].

## **2.3 Glacial Geology**

Since the AOI was once covered by glaciers, it is important to understand the area's glacial geology.

About 11,000 years ago, during a period of the glacial retreat, a proglacial lake formed, known as Lake Algonquin. Lake Algonquin was a large lake that encompassed current day Green Bay, Lake Michigan, Lake Huron, North Channel and the eastern half of Michigan's Upper Peninsula. Figure 5 is taken from Schaetzl et al. (2002) and is shown to illustrate the location of the ancient shoreline. Although this is only a hypothesized



**Figure 5: Extent of Lake Algonquin. The AOI is approximately where the orange arrow points (after Schaetzl et al. 2002).**

coverage of the ancient lake, Larson et al. (2001), Drzyzga et al. (2002) and Sage (2006) all concluded that the latest proglacial lake was formed at a consistent elevation of roughly 630 feet above MSL. From observing the picture above, it is seen that parts of the U.P. were submerged, especially on the eastern end of the Upper Peninsula and only a small portion of the land was above Lake Algonquin, thus forming a number of ancient shorelines. The formation of these shorelines was a direct result of isostatic rebound of the land surface as the glaciers retreated and outlet control as the water either moved south through various rivers or east to the current St. Lawrence Seaway.

Schaetzl et al. (2002) confirms the existence of four definite Algonquin shorelines, as the lake phases changed over time. Schaetzl et al. recommended that the four shorelines be referred to as Main phase, Ardtrea phase, Wyebridge phase and Payette phase—from highest shoreline in elevation to the lowest. The research located a couple of data points near the Cut River Bridge, identifying the shoreline as being from the Payette phase at an elevation of approximately 630 feet above MSL.

Research conducted by Sterrett and Edil (1982) investigated artesian conditions in glacial slopes along Lake Michigan in Wisconsin and their stability. One of the first issues noted was the fact that a glacial sand unit was formed under artesian conditions about 15' from the base of the bluff. Sterrett and Edil state that "wave action at the base

of the bluff is the most important cause of bluff top retreat”—since the US-2 roadway AOI has decent bluff top erosion occurring, the base wave action should be considered. However, the article expands on this theory and concludes that the top of the bluff is not directly influenced by wave action but is related to the transmission of water through jointed till. This fact could explain a portion, if not all, of the mechanism behind the artesian well within AOI. Although their area of study is not near the US-2 roadway AOI, the area involves the same ancient shoreline of Lake Algonquin.

## 2.4 Sinkhole Formation

Sinkholes are generally associated with rocks that form as chemical precipitates. The more soluble the rock (e.g. rocksalt) the more the rock will likely dissolve. Unlike rocksalt, dolostone and limestone are less soluble and will tend to dissolve only through fissures and fractures, where water flows--slowly dissolving the rock from the discontinuity and eventually forming cavities in the rock. These rock types are referred to as karst.

There are two major classifications of sinkholes; one type is a subsidence sinkhole and the other is a dropout sinkhole. A subsidence sinkhole can be formed from any type of soil but are most commonly found in till, especially where Pleistocene glaciers coated sediment over a limestone surface. The key component of a subsidence sinkhole is suffusion—the transport or settlement of the top soil

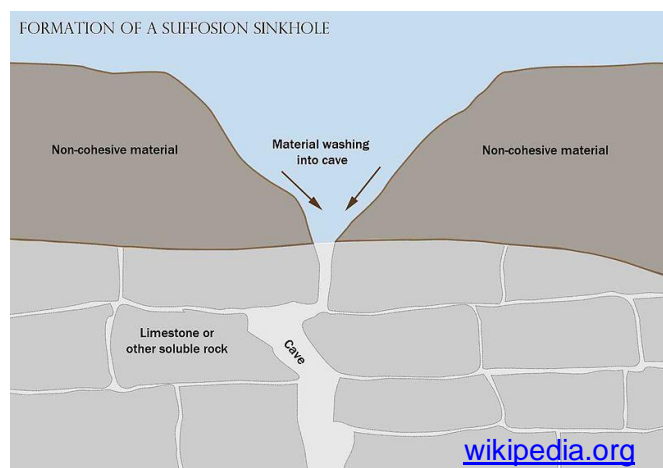


Figure 6: Formation of a Suffusion Sinkhole

layer into a pre-existing fissure of the underlying bedrock. Figure 6 illustrates suffusion. Since both glacial till and sand are soils, a subsidence sinkhole is more common as they would have limited strength to hold the soil in place if a void were to form below, in the bedrock. Differing from a subsidence sinkhole is a dropout sinkhole, distinguished by their rather rapid (or seemingly instantaneous) development. A dropout sinkhole can easily be considered a dramatic failure since it deals with a

sudden failure of soil over top a void in the bedrock. The reason it is such a sudden failure (many times without warning) is because a cohesive soil (e.g. clay) would be able to maintain the weight of the overlying soil until the void becomes too large for the cohesive soil to continue giving support. Unlike a subsidence sinkhole that is formed from sand slowly entering fissures in the bedrock, a clay layer would prevent any sort of suffosion of the topsoil but would continue to allow any possible water movement to dissolve the bedrock. A continuation of this process would ensure an increase in size of the fissures and ultimately increase the likelihood of a dropout sinkhole.

As previously mentioned, a dropout sinkhole can only be formed from a cohesive soil layer suddenly losing strength and a subsidence sinkhole is formed from suffosion of cohesionless soils. It should be understood that sinkhole formation mechanism may be different than expected. For example, if the soil from the area of interest consists of cohesionless soils it would seem that the sinkhole would form from suffosion and be considered a subsidence sinkhole. While this may be true, the possibility that an existing cohesive soil layer located beneath the cohesionless soil cannot be overlooked. If a cohesive soil layer exists, it is entirely possible that the sinkhole will start to form as a subsidence sinkhole, due to a crack in the cohesive soil layer, which eventually fails. Naturally, the cohesive layer will only fail after the strength of the cohesive soil is exceeded.

### **3.0 Assessment Site Soils**

The samples obtained for this research were collected using a hand auger with a diameter size of 2". Because these were taken from a sloughing portion of a slope, stability for the one gathering the soil was rather difficult—meaning the soil was collected at a very shallow depth, 1-3' below the surface. Once the samples were collected they were taken back to the lab for testing. This section will cover all of the data gathered from testing.

#### **3.1 Visual Classification**

All of the soil samples obtained at the location of the slough consisted of sand, with the main difference being the sand's density and color. The sands varied between

light, medium and dark brown sand, which seemed to be related to their depth below the roadway.

The roadway was at an elevation of about 742 feet above MSL. At an approximate elevation of about 690' the first soil sample was obtained at the very top of the slough. This sample indicated that the uppermost portion of this area consists of dark brown, medium grained sand that was in a relatively loose-compacted state. This sample was definitely the darkest sample out of the five but is just as uniform as the remaining samples.

Continuing down to the middle of the slough (elevation: 665'), another sample was taken at the surface. The middle of the slough consisted of a medium brown sand that was slightly more coarse-grained than the previously discussed sample. Another specimen taken using an extension to the auger was collected at 3' below the surface. Although these samples were taken at different depths, the visual classification of each of these two samples was identical.

Upon reaching where the water was flowing out of the slough at an elevation of nearly 635', a specimen was collected for testing. This sample was much lighter in color, a very light brown with a slight pink tint. The specimen is a medium-grain, light sand with distinctive traces of pink quartz.

The final specimen obtained was collected right from the very end of the creek, just before it runs into Lake Michigan (approximate elevation: 577'). The soil at the end of the creek consists of medium brown, fine-grained sand. This specimen seemed to be heavily composed of fine sand, much more so than any of the previous samples.

Although there was a total collection of five different soil samples, only the three that were taken directly from the slough will be discussed in detail throughout this report. The samples that will be discussed are the only ones that would have a direct effect on the stability of the slope.

### 3.2 Soil Morphology

The three samples that were taken from the slough were viewed under a microscope to observe the morphology of the particles. This section will illustrate what was discovered through means of a microscope.

The first specimen was taken from the top of the slough and shown in Figure 7 (left picture), which is magnified to a setting of 1X with a field of view (FoV) equal to 1 mm across. On the right side is the same sample magnified to 2.7X and has a FoV of 0.4 mm.

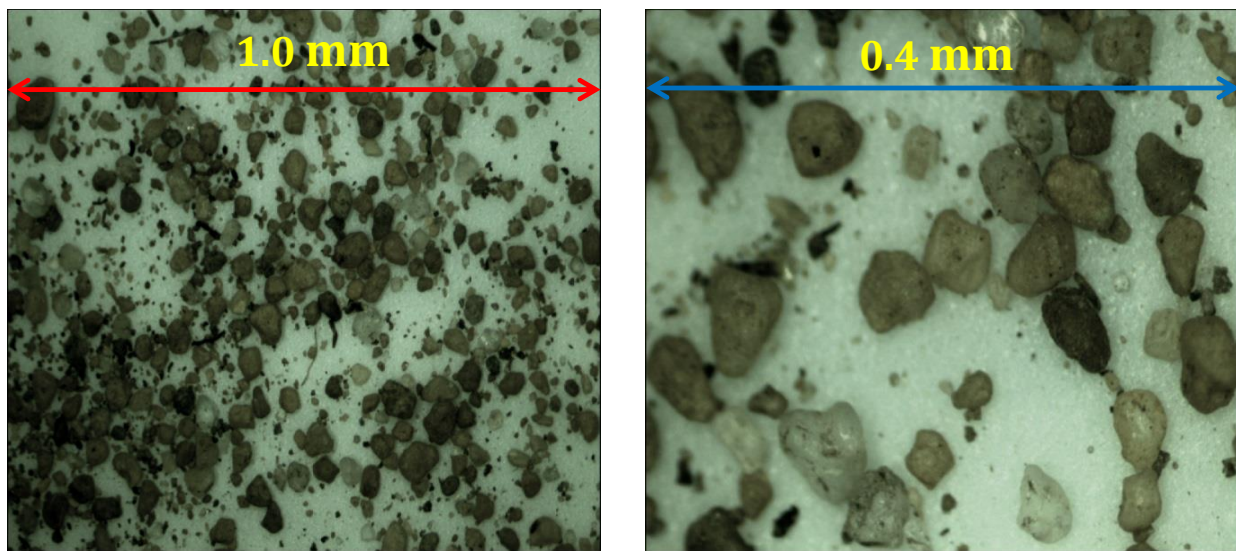


Figure 7: Top Slough Surface Soil Morphology

As mentioned in section 3.1, the sand from the top portion of the slough was considerably darker than the other samples taken at the site; the particles seem to have a consistent soil morphology that mostly resembles sub-rounded grains. From Figure 7, left, it is clear that the majority of the particles are composed of quartz (the grains near the bottom); it also seems to have traces of basalt (the very dark grains scattered about).

Moving vertically down the slope, the next two pictures below are of the surface of the mid slough (Figure 8 and Figure 9). The illustrations below are of magnification 1X and 2.7X, respectively.

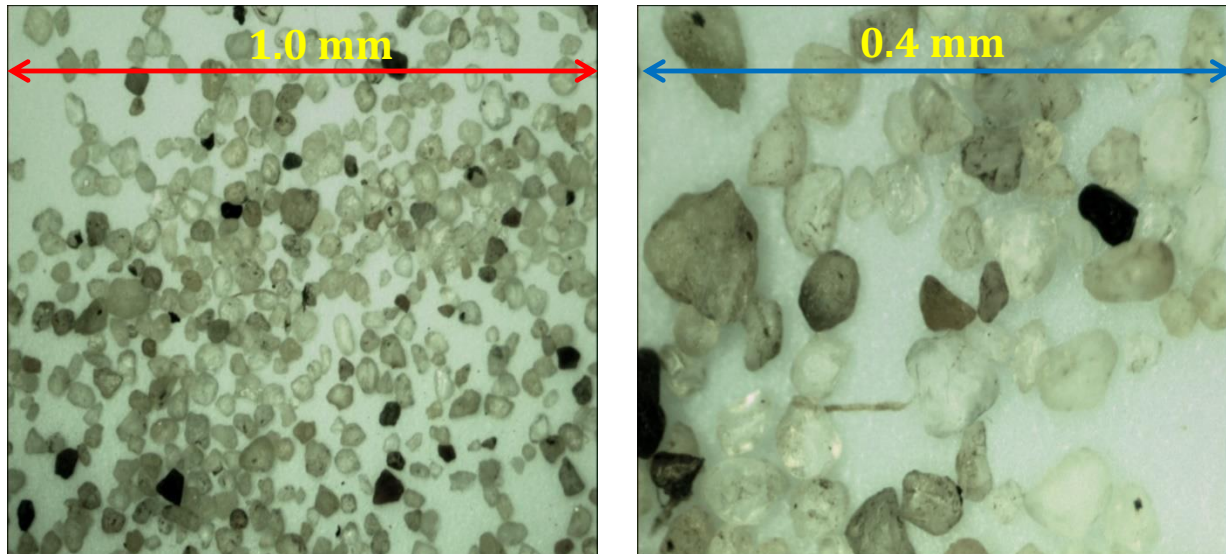


Figure 8: Middle of Slough Surface Soil Morphology

From Figure 8 it becomes very obvious that there is a heavy content of quartz in the make-up of this specimen.

Viewing Figure 9, it is seen that the soil particles look similar to the particles in Figure 8.

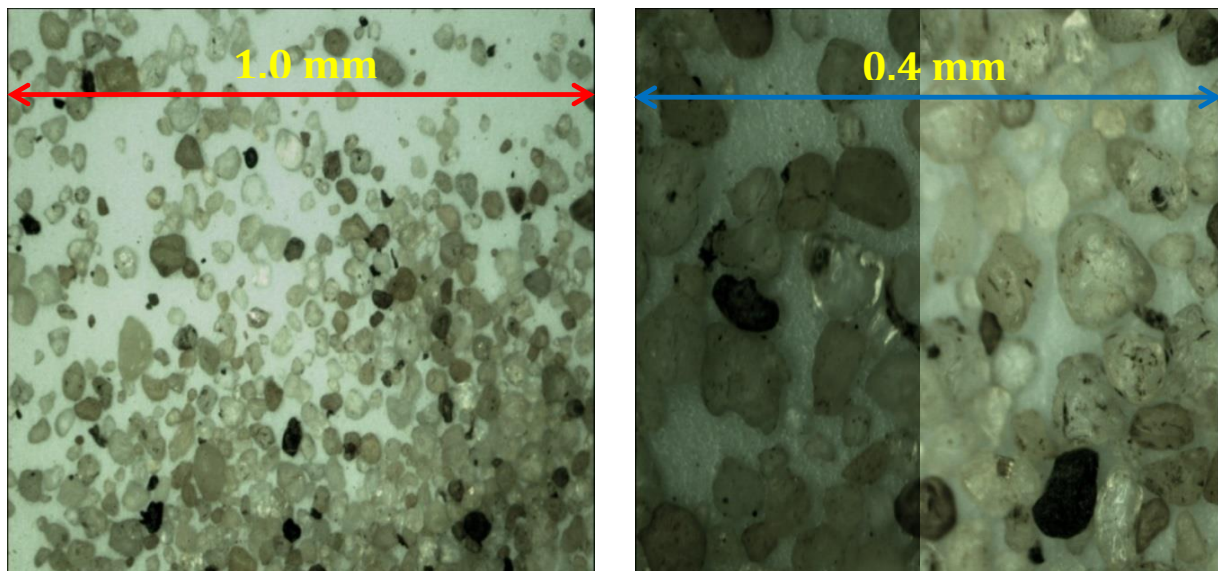


Figure 9: 3' Below Surface of Middle Slough Soil Morphology

### 3.3 Grain Size Analysis and Natural Moisture Content

As previously discussed, the sands in this area are fairly similar in texture and color and this section will expand upon the results of the grain size analysis of these sands as well as the natural moisture content.

Samples were collected on Thursday, June 7<sup>th</sup> 2013. The weather was dry with a temperature around 65°F. The samples were collected and brought back to the labs but were not tested until Monday, June 10<sup>th</sup> 2013. The soil samples were placed in an oven at 110 C° to dry for 24 hours prior to measuring their final dry weight. The moisture data is reported in Table 1.

Table 1: Natural Moisture Content

Top Slough [g]		Surface of Mid Slough [g]		3' Below Mid Slough [g]	
Wet Soil Weight	82.1	Wet Soil Weight	64.8	Wet Soil Weight	50.2
Dry Soil + Dish	118.4	Dry Soil + Dish	87.3	Dry Soil + Dish	74
Dish	38.2	Dish	24.7	Dish	25
Dry Soil	80.2	Dry Soil	62.6	Dry Soil	49
Water	1.9	Water	2.2	Water	1.2
Moisture Content [%]		Moisture Content [%]		Moisture Content [%]	
2.37		3.51		2.45	

The grain size distributions for the three samples are shown in Figure 10 through Figure 11. All three grain-size curves show a very uniform sand.

### Particle Distribution along the Top Slough

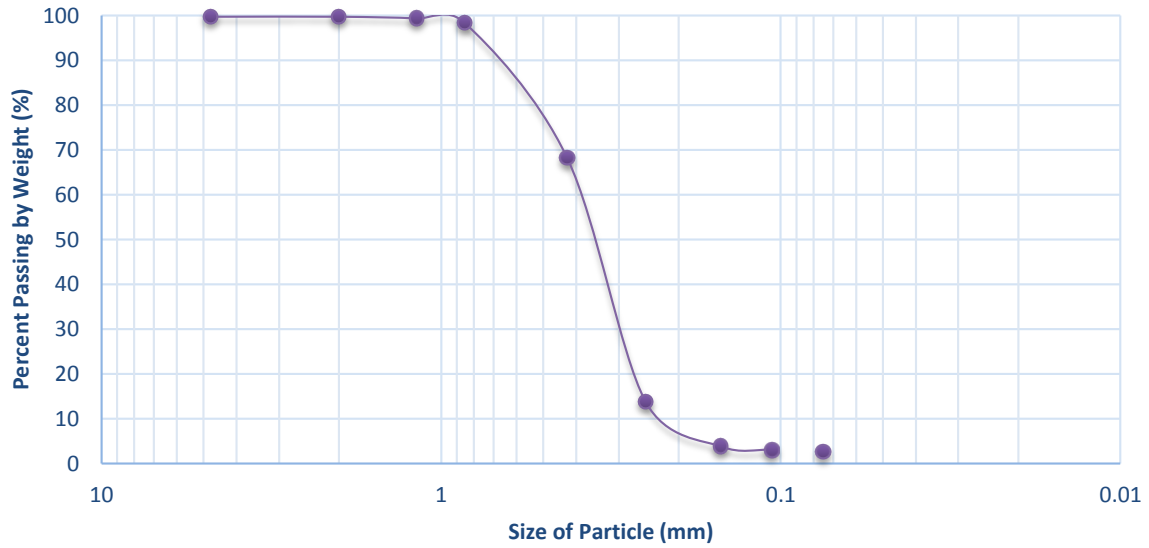


Figure 10: Top Slough Particle Distribution

### Particle Distribution 3' Below Surface of Mid Slough

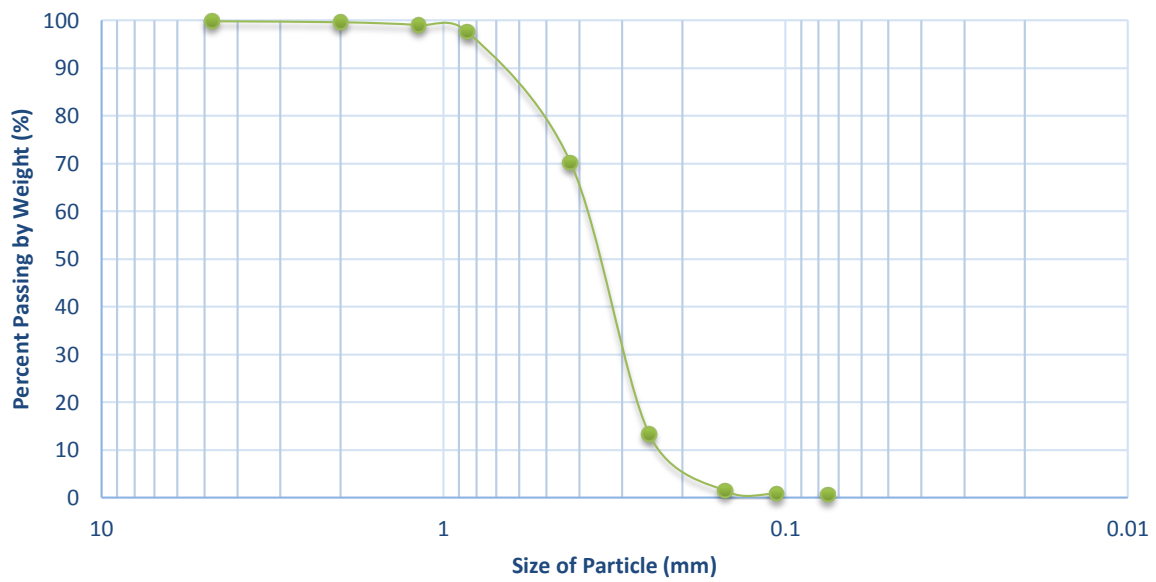


Figure 11: 3' below Surface of Mid Slough Particle Distribution

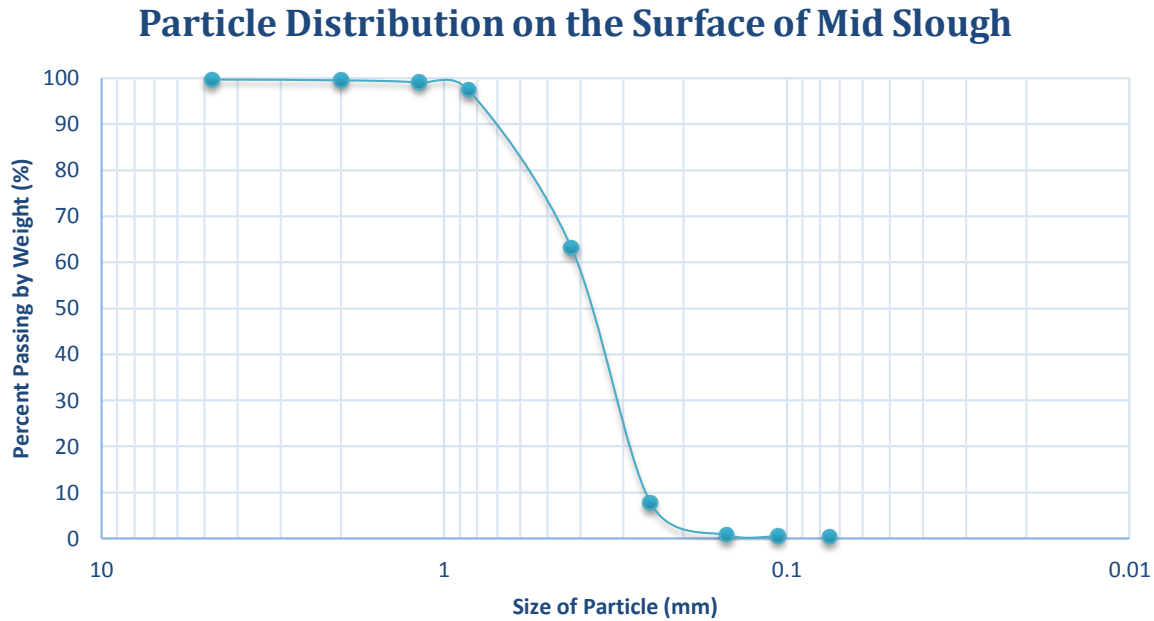


Figure 12: Surface of Mid Slough Particle Distribution

Please note—since the fines content in the samples were all less than 5% it was deemed unnecessary (by engineering standards) to perform a hydrometer analysis. Also, please reference Appendix A: Grain Size Distribution Tables & Comparison Chart for the complete material from this section.

### 3.4 Shear Strength Analysis

Determining the sand's shear strength will be used to assess the slope stability of the slough area. A standard Direct Shear Test (DST) was used to assess the strength of the sands. The tests were run using an electrical direct shear test machine with proving ring 15209, which was connected to DasyLab—a program that records all data points into a file than can easily be transferred to Excel. One statement worth mentioning before continuing with this section, is that only the “Top Slough” and “3’ Below Mid Slough Surface” will be reported. This was decided because of the similarity of the sands via visual classification, soil morphology and grain size analysis.

A number of DST tests were conducted. Since the DST were conducted with different compaction levels and various weights to get specific details regarding the soil's strength with respect to density. More specifically, both soil samples were tested a

total of six times; three dense samples and three loose. For each set of three, the normal forces used were: 26.4lbs, 50.3lbs and 75.5lbs.

For clarification, a ‘densely’ compacted specimen in this analysis implies that the soil sample consisted of three lifts and each lift was tamped 20 times versus a third of that compaction level (two lifts at 10 tamps each) for ‘loosely’ compacted samples..

On the following pages Figure 13 and Figure 14 display the shear strength vs horizontal displacement of the two samples. It was noticed that the loosely compacted and densely compacted samples gave very similar outcomes. Because the results show similarities between the dense and loose compaction stages, it confirms the grain-size analysis of uniform sand as well as the mostly rounded shape seen in the soil morphology—which is very similar to the results of Ottawa 20-30 sand—a sand that also has troubles being compacted.

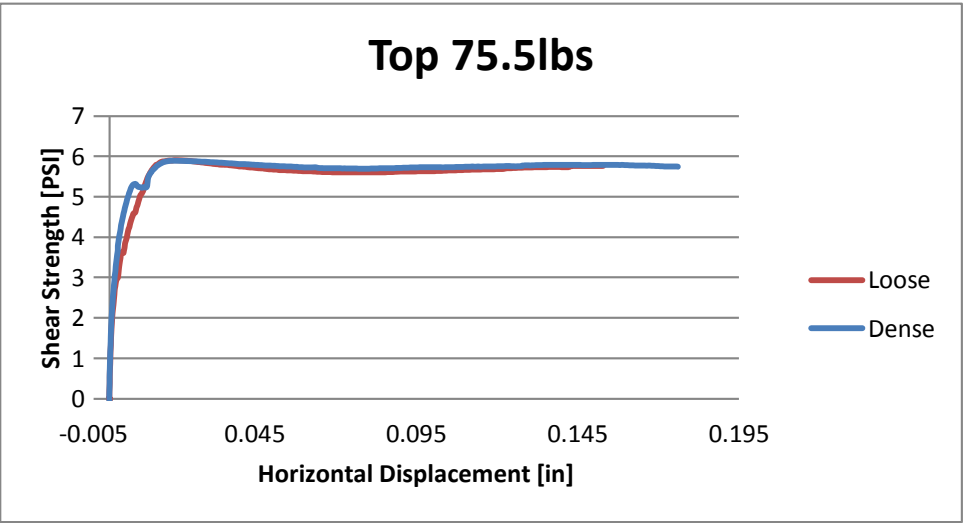
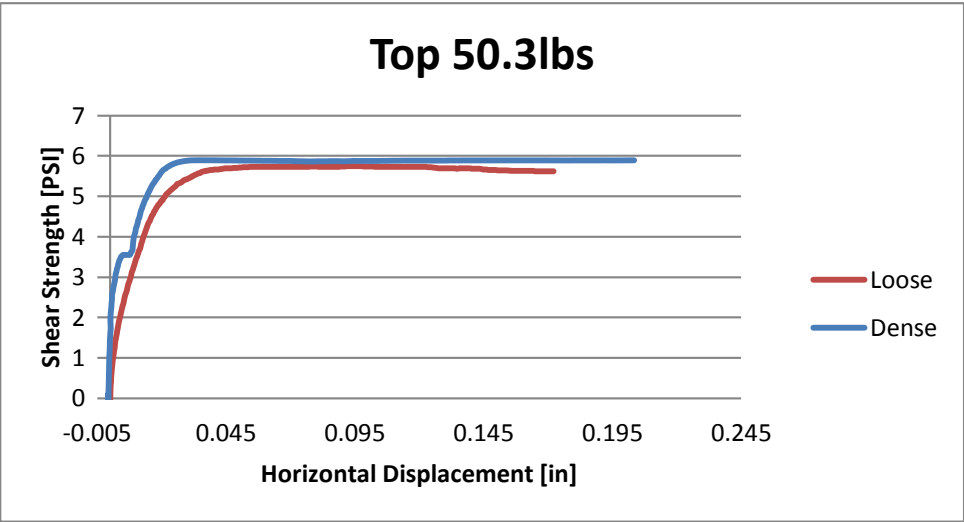
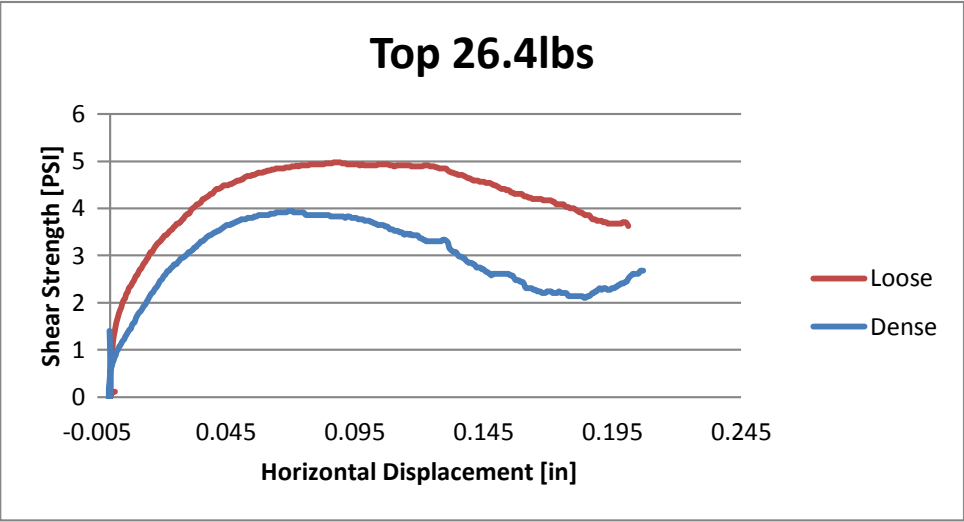


Figure 13: Shear Strength vs Horizontal Displacement for the Top Slough

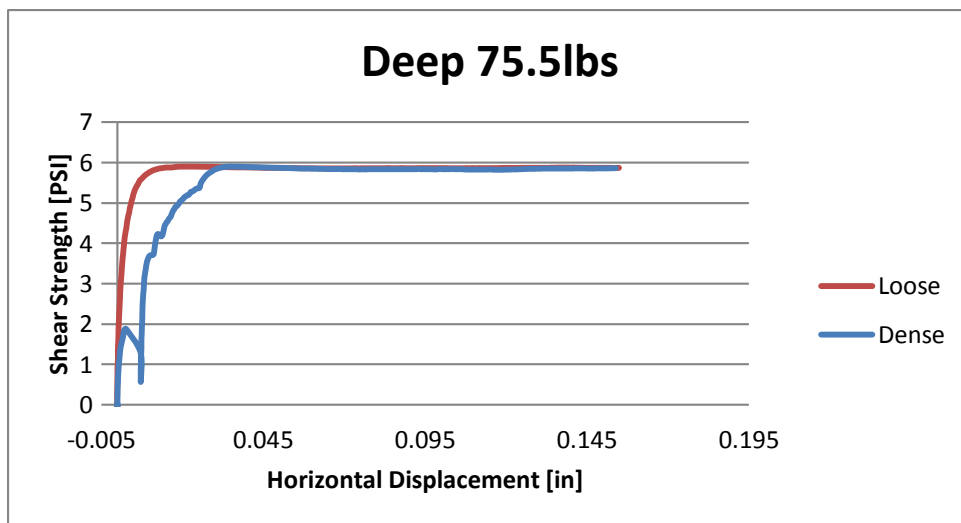
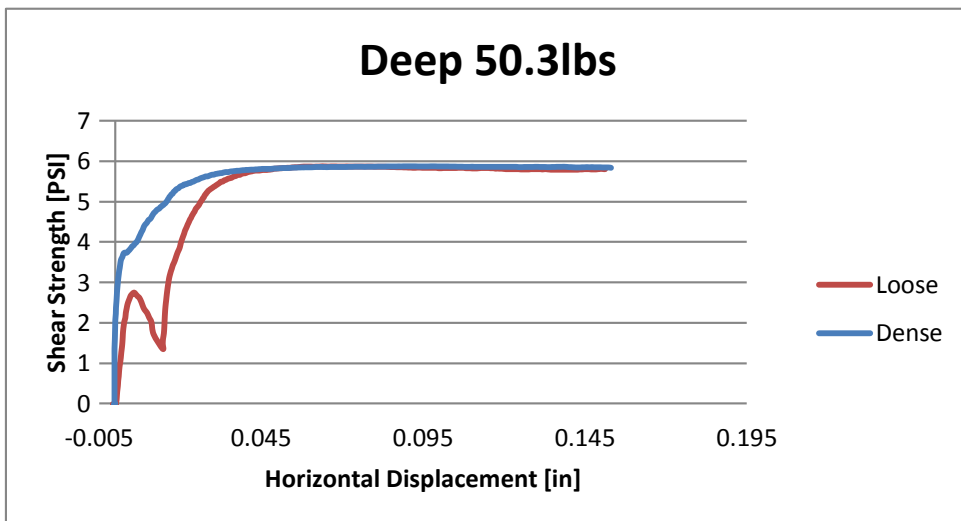
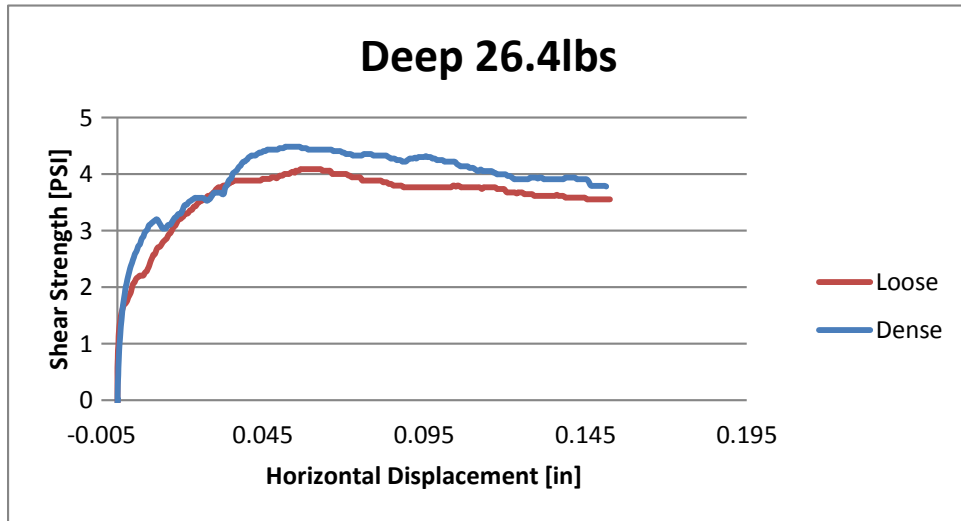


Figure 14: Shear Strength vs. Horizontal Displacement for 3' Below Mid Slough

After completing these tests, the values from DasyLab were copied over to an Excel file to be analyzed. Below, Figure 15 shows the result of the shear strength analysis; shear strength of the soil to normal stress in pounds per square inch.

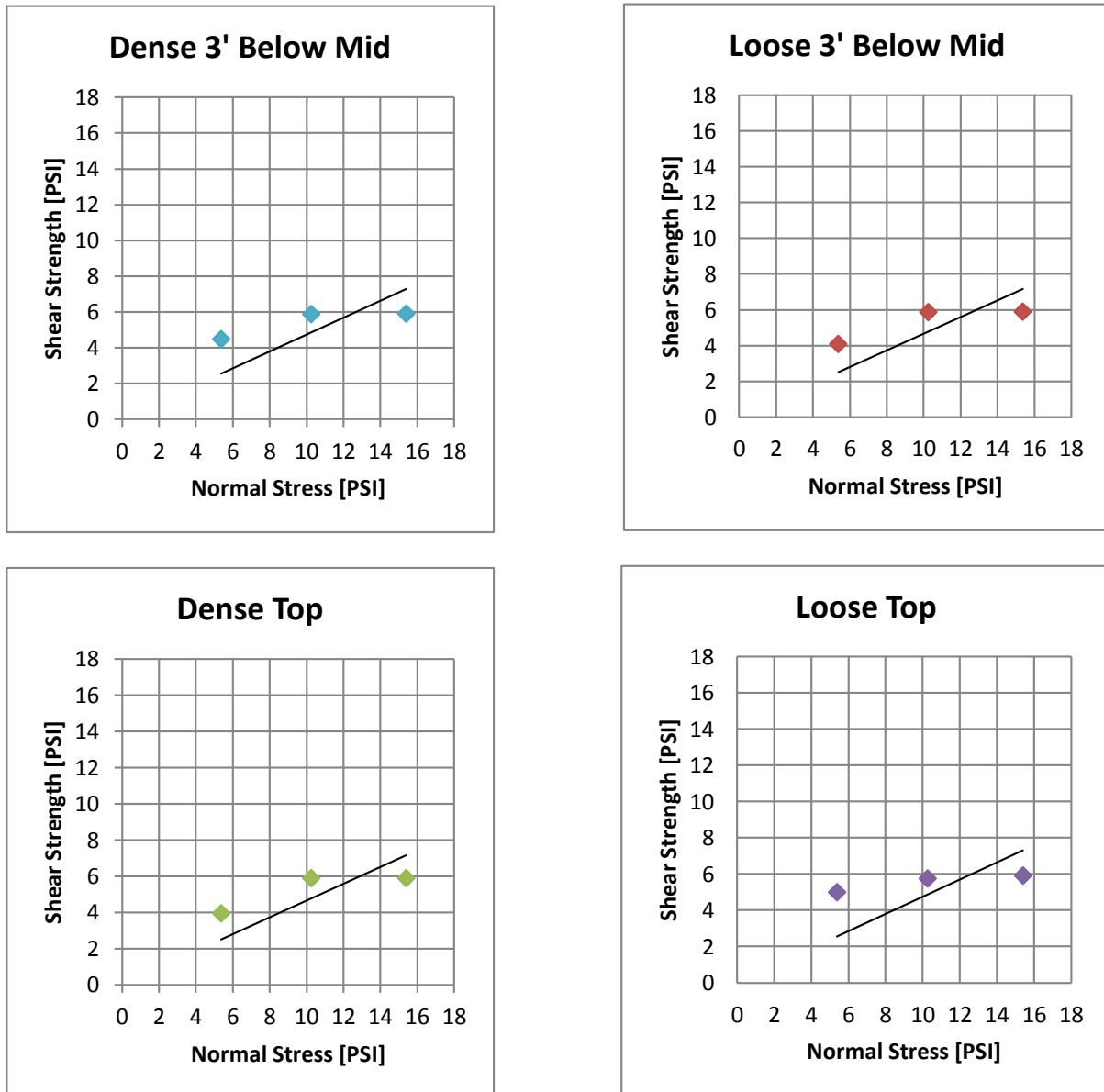


Figure 15: Normal vs. Shear Stress for Dense & Loose Samples of both Top and 3' below Mid Slough

These angles were graphically measured to have an average  $\phi$  of  $24^\circ$ . Considering a  $\phi$  angle of  $24^\circ$  is quite low, an average  $\phi$  was calculated to be about  $30^\circ$ . Table 2 below, shows the graphically interpreted  $\phi$  versus the calculated  $\phi$ .

Table 2: Summary of Phi Angles

Dense 3' Below Surface of Mid Slough				Loose 3' Below Surface of Mid Slough			
TAU [PSI]	N [LBS]	SIGMA [PSI]	PHI [DEG]	TAU [PSI]	N [LBS]	SIGMA [PSI]	PHI [DEG]
4.48845053	26.4	5.378163837	39.85	4.0861435	26.4	5.378163837	37.23
5.88067787	50.3	10.24703186	29.85	5.87500839	50.3	10.24703186	29.83
5.89773521	75.5	15.3807337	20.98	5.89773317	75.5	15.3807337	20.98
Average Phi (calculated)			30.23	Average Phi (calculated)			29.34
Average Phi (graphically interpreted)			24	Average Phi (graphically interpreted)			24

Dense Top Slough				Loose Top Slough			
TAU [PSI]	N [LBS]	SIGMA [PSI]	PHI [DEG]	TAU [PSI]	N [LBS]	SIGMA [PSI]	PHI [DEG]
3.94303138	26.4	5.378163837	36.25	4.97481174	26.4	5.378163837	42.77
5.89773521	50.3	10.24703186	29.92	5.74661695	50.3	10.24703186	29.28
5.89773317	75.5	15.3807337	20.98	5.8976965	75.5	15.3807337	20.98
Average Phi (calculated)			29.05	Average Phi (calculated)			31.01
Average Phi (graphically interpreted)			23	Average Phi (graphically interpreted)			24

When looking at Table 2 it becomes obvious that the calculated phi is much higher than the graphically interpreted phi. It is typical to gather useful phi angles from a graphical interpretation because the angle the trendline forms, relative to the horizontal, is the failure envelope. If you were to draw Mohr's circle on these graphs you would be able to tell where the strength of the soil will fail (in shear) with varying values of normal stress, as well as the angle of internal friction (phi)—assuming the grids are perfectly square. Also, since Figure 15 was adjusted manually and is not likely a perfect square; hence the fact that the graphically interpreted method (in this case) is probably not the most accurate.

## 4.0 Stability Analysis

This section discusses the stability of the slope with regarding the slough. Please note, as the model is rather complex most figures for this section can be found (linked) throughout this section to .

### 4.1 Post-Initial Investigation

After the initial investigation from MDOT was completed, the MDOT officials contacted Dr. Stan Vitton to determine if there were any pending stability problems. Meanwhile, a conventional survey was performed to gather the topography of the six specified cross sections. Using the topographical cross section titled “25FT WEST” (found in the Appendix B: Materials from MDOT, Ground Survey), the slope coordinates were put into Rocscience: Slide, forming an external boundary.

Shown below, Figure 16 is a cross-sectional view of the land slope. The y-axis is set to feet above sea level and the x-axis zero-point is the center line of the roadway. In this model there are about 16 different programmed materials, most of which are slight variations of sand: loose (coarse) at the top and medium dense to dense (medium coarse to some fine) further below the surface. Finally, the bottom (not solid) shaded areas represent bedrock: limestone, dolostone, sandstone and shale (most to least). A complete list of materials are provided in Appendix E: Material Properties. The [Original LEM Modeler](#) and [Original FEA Modeler](#) can also be found in Appendix F: Stability Analysis Models.

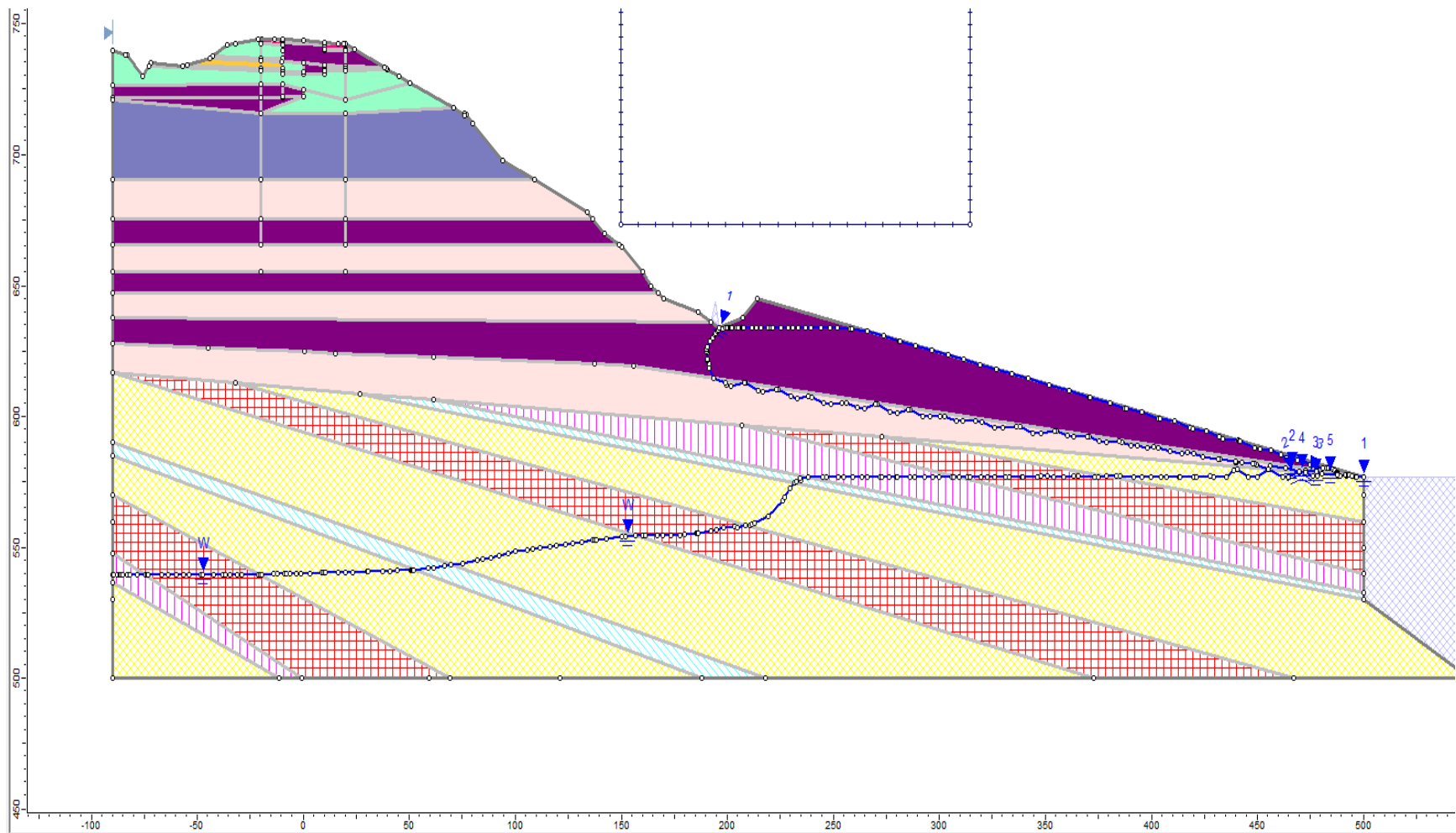


Figure 16: Multiple Water Tables

The water table is an entirely separate and an even more complex matter that will be discussed in 4.3 Ground Water Table Assumptions.

## 4.2 Reasoning behind Material Assumptions

As mentioned at the beginning, much of the data was assumed from the MDOT boring log data. It is assumed that the data collected by the nine borings is representative of the slope, therefore the model was expected to have all specified soil types throughout the slope.

The bedrock consisted of mostly limestone and dolostone with some shale and sandstone. This is an assumption based off of the Niagara Escarpment and its geologic makeup because this section of the roadway is along the top of the Niagara Escarpment. The Niagara Escarpment is a geologic structure that formed as a result of the Michigan Basin. This basin formed over a period of approximately 25 million years when Michigan was an ancient sea. As the carbon-based sea creatures lived and died throughout this period, their shells eventually settled to the bottom of the sea and mixed with the natural sea sediment of sand, silt and clay. Over time these materials were compressed and hardened into layers of shale, sandstone, siltstone, limestone and dolostone.

## 4.3 Ground Water Table Assumptions

Since there are possibly two water tables on this slope, both were incorporated into the slope stability model. However, considering the water table depth (at the time of the initial modeling phase) was still technically unknown, the GWT was set as unknown and a test was run through the Limit Equilibrium Method (LEM) to determine where the GWT would be. What was discovered with this test is shown as the GWT in the Figure 16. While the placement of the shown GWT could be true, it would not explain why there are artesian conditions happening nearly 100' above the GWT.

Because the GWT was initially unknown, it should also be mentioned that a couple of different water tables (depth and number of GWT) were simulated. In the Appendix [[Original LEM Modeler](#) and [DEQ LEM Modeler](#)] show the other water tables tested. However, since all water tables were determined to be below the slough, the stability analysis results did not change with the different GWT levels.

Subsequently, most of the tests that were performed used a single GWT that simply travel along an angle and barely touched the artesian well and continued downward to make contact with the lake. Shown below, Figure 17 is the LEM global minimum (Bishop simplified) analysis with a singular GWT resulted in a FS of 0.655 [[Original LEM](#)]. For clarification, global minimum implies that out of all failure surfaces, the one with the lowest value is the global minimum.

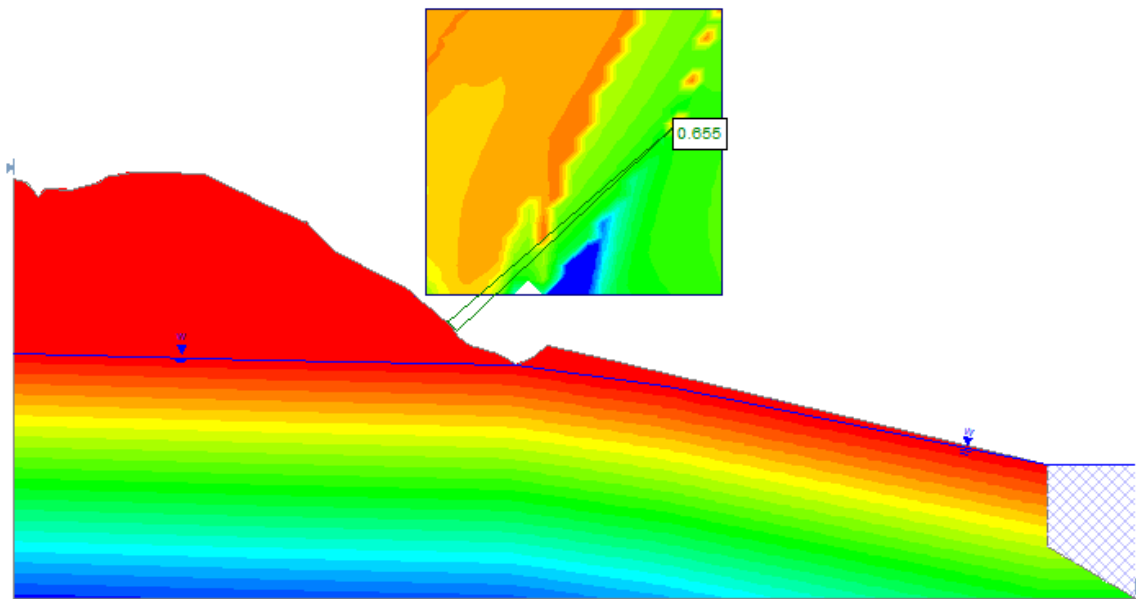


Figure 17: Bishop Simplified LEM FS

The cross-section that was programmed into Rocscience: Slide was later imported directly to Rocscience: Phase 2 to compare the strength reduction factor (SRF) to the LEM FS. Referencing [Original FEA](#), the overall result of the critical SRF of 0.67.

Although the SRF was determined to be slightly greater than the FS, they were more or less the same value, which was less than 1; implying that it is failing. While it is not the best news to see that every test run for this slope confirmed the slope is failing, it is encouraging to know that both very different methods give similar results, ensuring the quality of the analysis.

#### 4.4 Post-Initial Analyses

Considering there are many types of engineering properties and each material will change its engineering properties based on many things such as saturation and climate change. Because of this some more tests were run to develop a more accurate model of the slope. Some variances in these analyses included different types of failure search methods (for LEM) such as: [grid search](#), [slope search](#), block search and path search. Although these variations were completed, there was not a large enough difference in the FS results to go into more detail in this report.

Upon completing some further research regarding water well data from Michigan's DEQ, another model [[DEQ LEM](#)]/[DEQ FEA](#)] was developed using this data (found in Appendix C: DEQ Well Water Levels and Bedrock Depths). Although the sands were not more specific than “fine, medium and coarse” the benefit of this data was the thickness of each major material layer, as well as the fact that they actually recorded the depth at which the bedrock began. Once the model had been redone using DEQ data, the analysis had determined that the FS (for [DEQ LEM Grid Search](#)) had been lowered slightly, to 0.527 and 0.615 (for [DEQ Slope Search](#)). This was likely a different value due to the addition of various limestone engineering properties and the fact that the sand was all the same density. In the original model, there was only one version of limestone and it was programmed to be a typical strong limestone with little to no weathering. However, if the DEQ data is considered it is quickly understood that there is only one small section of durable limestone. Unfortunately, since there was no core samples of any of the variations of limestone, the Hoek-Brown criterion was still assumed.

Even though all the results give nearly the same FS, it should be understood that a probable explanation for all the results showing failure is likely because the slope is actually heavily wooded—however, none of the vegetation is programmed in any of these models, due to a lack of accurate vegetation knowledge. One last thing to note would be the depth of all the failure surfaces ( $FS < 1$ ) is no greater than 11' into the slope—the deeper the failure, the higher the FS. In Appendix F: Stability Analysis Models are all tests ran for both LEM and FEA, as well as the depth for deepest (LEM) slip surface and the global minimum slip surface.

## 5.0 Analysis and Conclusions

Instability has been observed along the slope between US-2 and Lake Michigan. The instability is a slough about midway up the slope. Artesian groundwater conditions were observed during the spring of 2012. An inspection sometime later did not observe artesian conditions. Instead a flow of water was observed emanating out of the slough area.

Sinkholes were present during the construction of the highway. It appears that for the most part the sinkholes have remained dormant. However, some soil movement appears to have occurred with the footprint of the existing sinkholes. The existing sinkholes appear to have all been mapped on the original highway construction plans.

Soil samples were obtained from the landslide site and analyzed. Soils consist of clean, uniform and rounded to sub-rounded sands, mostly from aeolian processes. The shear strength of the sand was found to be about 30°. The slope average angle was measured to be barely over 31° and is heavily wooded, indicating that the slope is nearly at its maximum slope angle.

The sand that makes up the slope had been deposited by glacial action and is associated with the Laurentide Ice Sheet (Wisconsin glaciation). Also, regarding the glacial effect of the area—the section of the slope that is currently sloughing appears to correlate with a glacial lake stage. It was during this point in history when a silt or clay layer developed and now acts as an impermeable layer guiding water out of the slope.

Although the analysis in this research indicates that the slope is at a point of failure, some other aspects should be considered. For one thing, the soil samples were rather hard to collect with a hand auger standing on a steep slope surrounded by uprooting trees, thus only samples between one and three feet deep could be obtained. Another important factor is the fact that there is little to no history of landslides in the area and there are slopes just as steep in the area (the Cut River Bridge) with no instability. One more important issue is the fact that vegetation is likely playing a key role to slope stability and in the model that was created for this report—all of resisting and driving

forces of the vegetation were neglected, undoubtedly giving a less accurate factor of safety.

For future work and recommendations, it is recommended to continue observing the slope as well as conducting further soil testing, deeper borings and a determination of the groundwater conditions to observe changes or irregularities. Since sinkholes are actually present in the area it cannot be ruled out that something is happening in the subsurface that is related to a change in the groundwater table—possibly in relation to karst. Unfortunately, since time did not allow for karst testing, a major future recommendation would be to use geophysics such as electrical resistivity (ERM) in order to test for karst topography.

The Electrical Resistivity Method (ERM) is an extremely useful device as it contains different electrical resistivity values for all types of soil and rock—directly in the instrument itself, making it as easy as possible for the engineer conducting the test. It works by sending an electrical wave into the ground and based on the resistive properties of the material it is penetrating, it will give different values for different materials as well as the depth of the layer. Specifically, the electrical resistivity values will decrease as fine and moisture content increases through the subsurface. In Appendix G: ERM (Electrical Resistivity Method) Example there is an example of what the karst imaging would look like after an ERM test was performed.

## 6.0 References

Bell, F. G. and M. G. Culshaw (2005). Sinkholes and subsidence: Karst and cavernous rocks in engineering and construction, Springer.

Drzyzga, S., et al. (2002). Revealing uncertainty in maps of Glacial Lake Algonquin. Advances in Spatial Data Handling, Springer: 377-389.

Heath, A. J. and P. F. Karrow (2007). "Northernmost (?) Glacial Lake Algonquin Series Shorelines, Sudbury Basin, Ontario." Journal of Great Lakes Research **33**(1): 264-278.

Lake, C. (2004). "St. Catharines 2004."

Larson, G. and R. Schaetzl (2001). "Origin and evolution of the Great Lakes." Journal of Great Lakes Research **27**(4): 518-546.

Sage, R. P. and V. L. Sage (2006). Glacial Lakes Algonquin and Nipissing Shoreline Bedrock Features: Mackinac Island, Michigan, Institute on Lake Superior Geology.

Schaetzl, R. J., et al. (2002). "Measurement, correlation, and mapping of Glacial Lake Algonquin shorelines in northern Michigan." Annals of the Association of American Geographers **92**(3): 399-415.

Sterrett, R. J. and T. B. Edil (1982). "Ground-Water Flow Systems and Stability of a Slope." Ground Water **20**(1): 5-11.

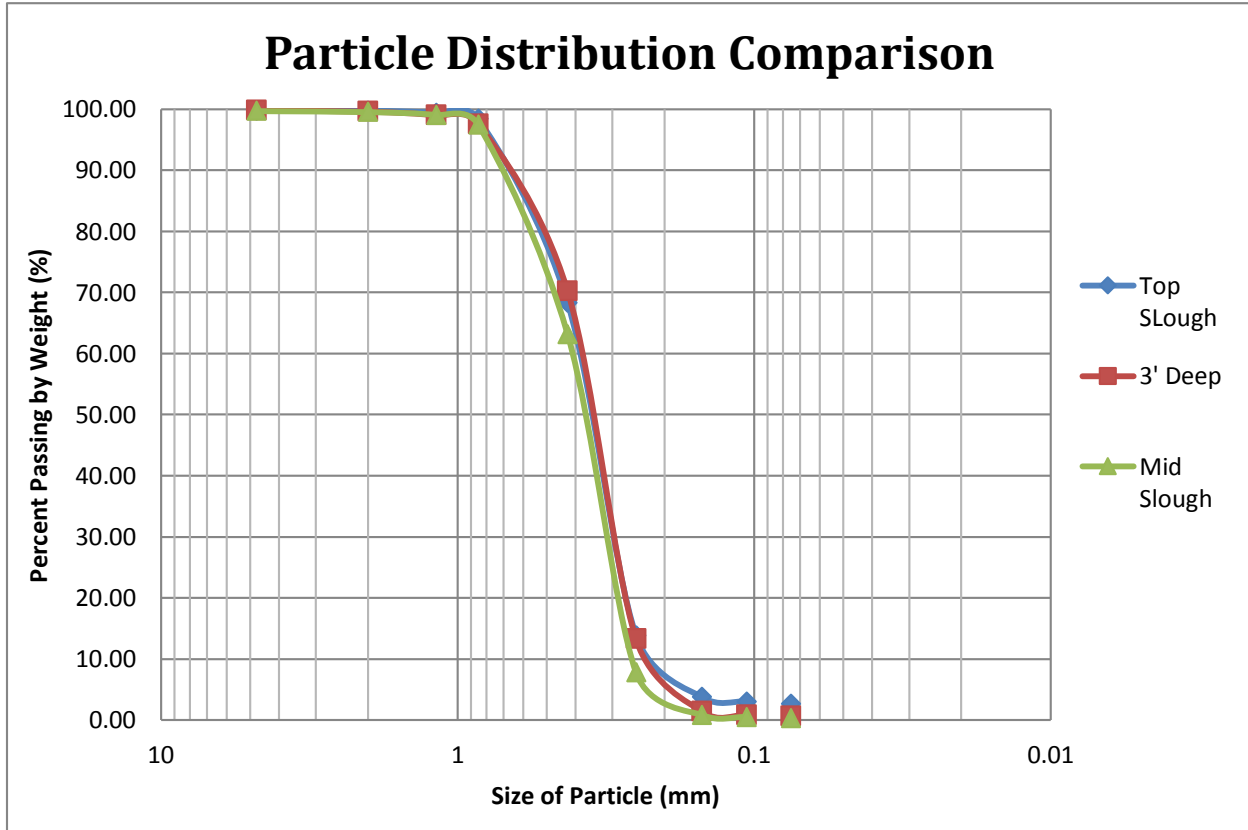
Tihansky, A. B. and L. A. Knochenmus (2001). "Karst features and hydrogeology in west-central Florida—a field perspective." US Geological Survey Karst Interest Group, Proceedings US Geological Survey Water-Resources Investigations, St Petersburg, FL: 198-211.

## Appendix A: Grain Size Distribution Tables & Comparison Chart

<b>Particle Distribution along the Top Slough</b>							
<b>Sieve Number</b>	<b>Sieve Size (mm)</b>	<b>Sieve Weight (g)</b>	<b>Sieve Weight + Soil (g)</b>	<b>Weight of Soil Retained (g)</b>	<b>Cumulative Soil Weight Retained (g)</b>	<b>Cumulative Percent Retained</b>	<b>Percent Passing</b>
4	4.75	718.3	718.5	0.2	0.2	0.25	99.75
10	2	433.2	433.2	0	0.2	0.25	99.75
16	1.18	454.2	454.5	0.3	0.5	0.63	99.37
20	0.85	407.9	408.7	0.8	1.3	1.64	98.36
40	0.425	389	412.9	23.9	25.2	31.74	68.26
60	0.25	316.5	359.7	43.2	68.4	86.15	13.85
100	0.15	352.7	360.7	8	76.4	96.22	3.78
140	0.106	331.6	332.2	0.6	77	96.98	3.02
200	0.075	291.7	292	0.3	77.3	97.36	2.64
pan	0	278.6	278.8	0.2	77.5	97.61	2.39

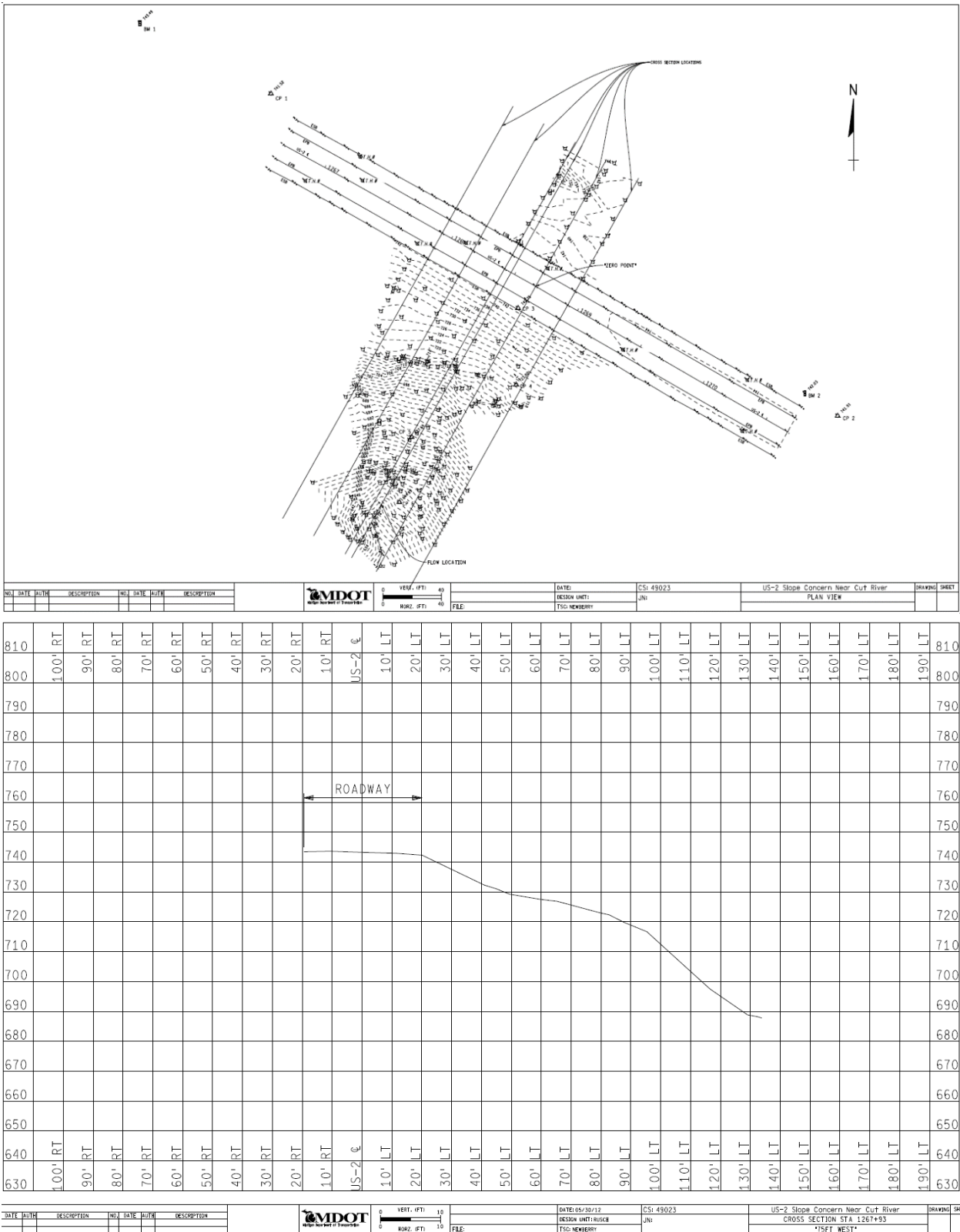
<b>Particle Distribution along the Surface of the Mid Slough</b>							
<b>Sieve Number</b>	<b>Sieve Size (mm)</b>	<b>Sieve Weight (g)</b>	<b>Sieve Weight + Soil (g)</b>	<b>Weight of Soil Retained (g)</b>	<b>Cumulative Soil Weight Retained (g)</b>	<b>Cumulative Percent Retained</b>	<b>Percent Passing</b>
4	4.75	718.1	718.3	0.2	0.2	0.32	99.68
10	2	433	433.1	0.1	0.3	0.49	99.51
16	1.18	454.1	454.4	0.3	0.6	0.97	99.03
20	0.85	407.7	408.7	1	1.6	2.59	97.41
40	0.425	388.8	410	21.2	22.8	36.89	63.11
60	0.25	316.3	350.5	34.2	57	92.23	7.77
100	0.15	352.5	356.8	4.3	61.3	99.19	0.81
140	0.106	331.4	331.6	0.2	61.5	99.51	0.49
200	0.075	291.6	291.7	0.1	61.6	99.68	0.32
pan	0	278.4	278.5	0.1	61.7	99.84	0.16

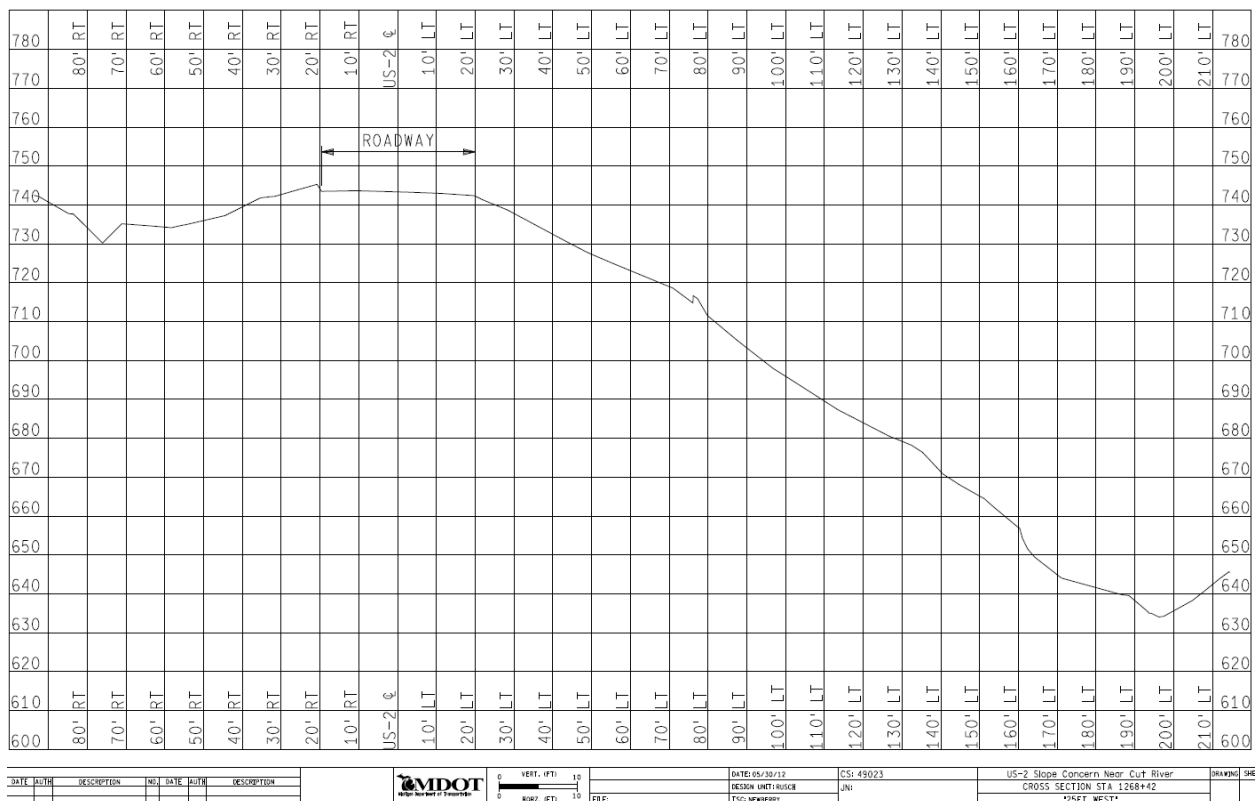
Particle Distribution 3' below the Surface of the Mid Slough							
Sieve Number	Sieve Size (mm)	Sieve Weight (g)	Sieve Weight + Soil (g)	Weight of Soil Retained (g)	Cumulative Soil Weight Retained (g)	Cumulative Percent Retained	Percent Passing
4	4.75	718.1	718.2	0.1	0.1	0.20	99.80
10	2	433.1	433.2	0.1	0.2	0.41	99.59
16	1.18	454.1	454.4	0.3	0.5	1.02	98.98
20	0.85	407.7	408.4	0.7	1.2	2.45	97.55
40	0.425	388.8	402.2	13.4	14.6	29.80	70.20
60	0.25	316.3	344.2	27.9	42.5	86.73	13.27
100	0.15	352.6	358.4	5.8	48.3	98.57	1.43
140	0.106	331.5	331.8	0.3	48.6	99.18	0.82
200	0.075	291.6	291.7	0.1	48.7	99.39	0.61
pan	0	278.4	278.5	0.1	48.8	99.59	0.41

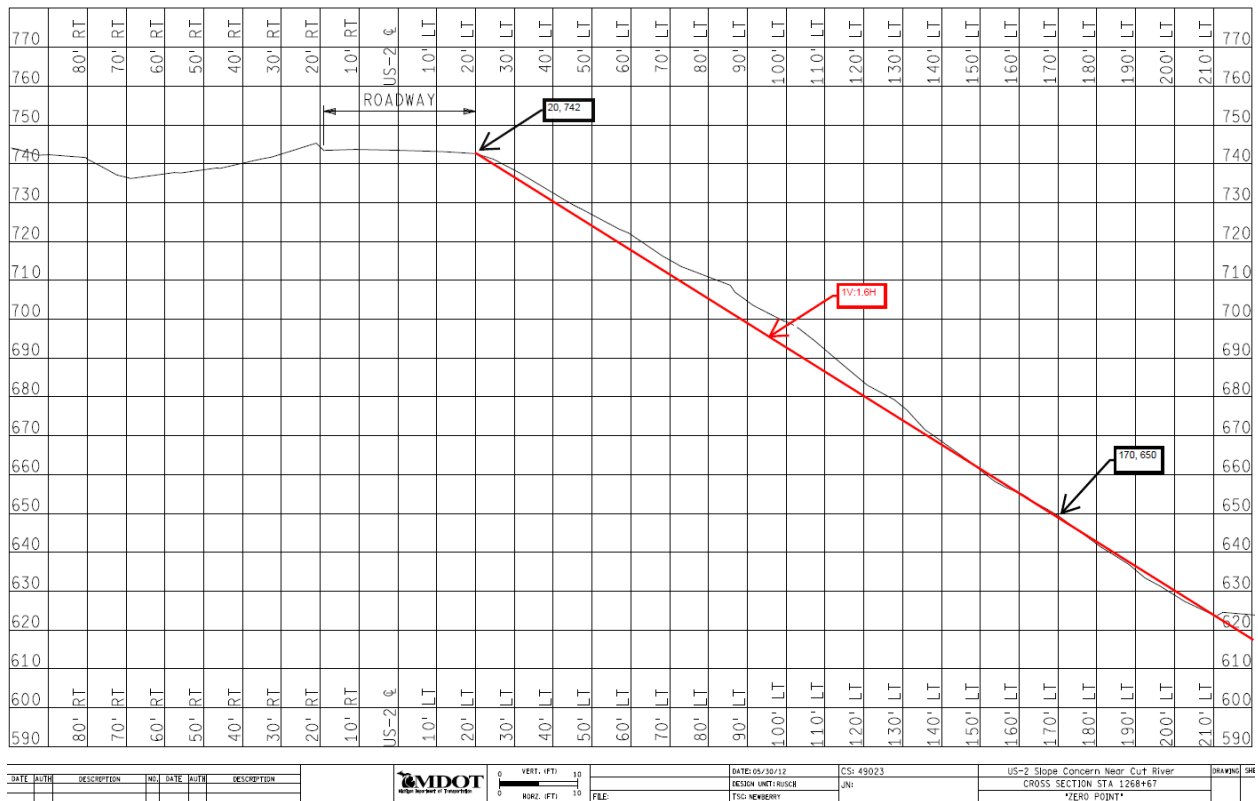
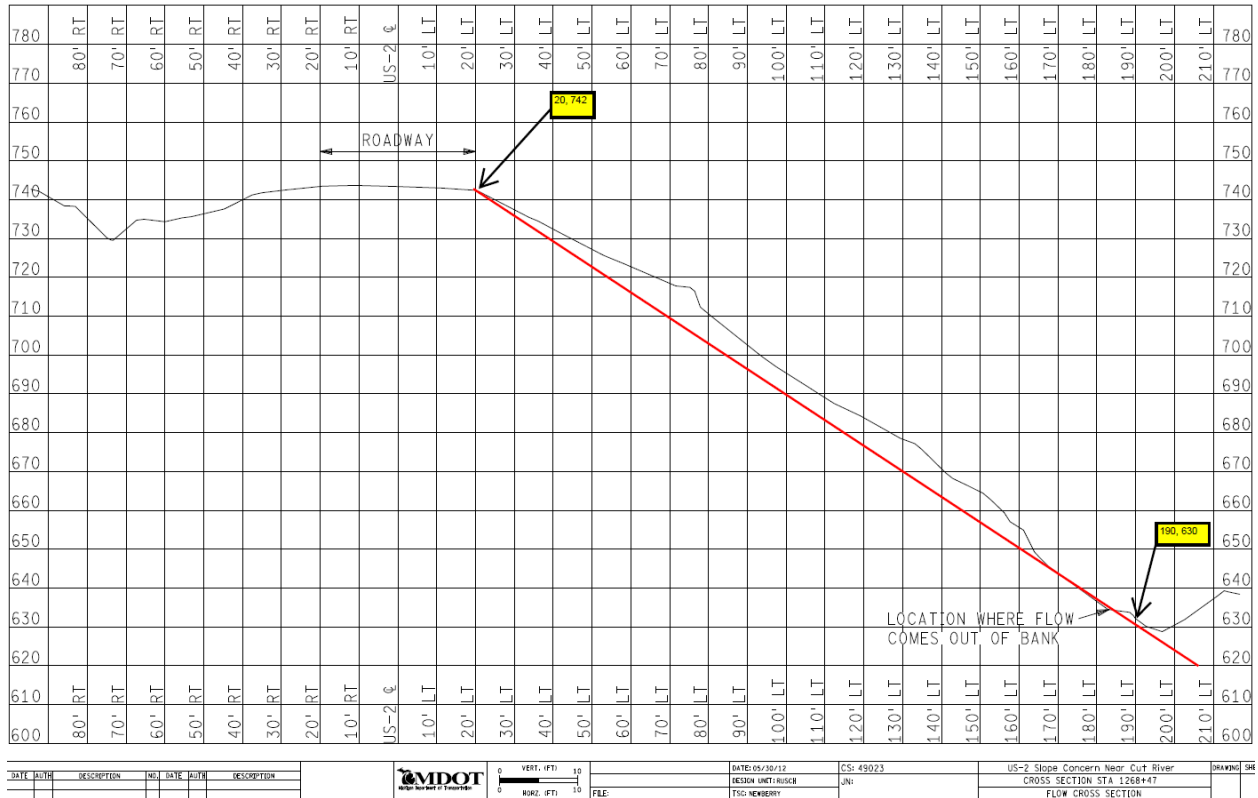


Appendix B: Materials from MDOT

Ground Survey









# Truth Boring Data

TEST HOLE NO. 1		TEST HOLE NO. 2		TEST HOLE NO. 2		TEST HOLE NO. 3		TEST HOLE NO. 4	
LOCATION STATION: 1267407 15 FT LT OF CENTERLINE US-2 N: 407753, E: 26733071 US-2 WEST OF THE CUT RIVER GROUND SURFACE ELEVATION: 742.88 FT		LOCATION STATION: 1267417 28 FT LT OF CENTERLINE US-2 N: 407753, E: 26733071 US-2 WEST OF THE CUT RIVER GROUND SURFACE ELEVATION: 743.60 FT		CONTINUED		LOCATION STATION: 1267407 4 FT LT OF CENTERLINE US-2 N: 407753, E: 26733071 US-2 WEST OF THE CUT RIVER GROUND SURFACE ELEVATION: 743.48 FT		LOCATION STATION: 1267402 14 FT LT OF CENTERLINE US-2 N: 407753, E: 26733071 US-2 WEST OF THE CUT RIVER GROUND SURFACE ELEVATION: 742.85 FT	
<p>742.88 742.5 742.1 741.7 741.3 737.9 735.9 735.5 733.9 732.9 727.5 722.0 721.0 E.O.B. 23.5 FT</p> <p>HMA MEDIUM DENSE BROWN FINE SAND WITH GRAVEL MEDIUM DENSE DARK BROWN FINE SAND MEDIUM DENSE BROWN FINE SAND VERY LOOSE BROWN FINE SAND- OCCASIONAL TOPSOIL LENSES LOOSE BROWN FINE SAND LOOSE BROWN FINE SAND</p>		<p>743.60 743.2 742.8 742.4 738.0 736.4 736.1 734.4 732.9 731.4 728.4 726.4 723.6 721.0 718.4 715.0 712.4 708.0 704.0 698.0 693.0 690.0 E.O.B. 96.5 FT</p> <p>HMA LOOSE BROWN FINE SAND WITH GRAVEL LOOSE BROWN FINE SAND LOOSE BROWN FINE SAND VERY LOOSE BROWN FINE SAND VERY LOOSE DARK BROWN FINE SAND WITH TOPSOIL LENSES LOOSE BROWN FINE SAND DENSE BROWN FINE SAND DENSE BROWN FINE SAND</p>		<p>743.48 742.8 742.5 741.4 740.4 738.5 736.4 735.5 729.5 728.5 725.5 723.5 722.0 E.O.B. 21.5 FT</p> <p>HMA CONCRETE MEDIUM DENSE BROWN FINE SAND LOOSE BROWN FINE SAND VERY LOOSE BROWN FINE SAND LOOSE BROWN FINE SAND MEDIUM DENSE BROWN FINE SAND MEDIUM DENSE BROWN FINE SAND</p>		<p>742.85 742.5 741.4 740.4 737.4 735.4 733.9 732.9 727.4 722.0 721.0 E.O.B. 23.5 FT</p> <p>HMA LOOSE BROWN FINE SAND WITH GRAVEL LOOSE BROWN FINE SAND VERY LOOSE BROWN FINE SAND LOOSE BROWN FINE SAND LOOSE BROWN FINE SAND</p>			
BORING DATE: 5/22/12 FREE WATER WAS NOT OBSERVED BELOW HMA SURFACE DURING DRILLING. TEST HOLE WAS DRY AFTER COMPLETION OF DRILLING (AUGERS OUT). DRILLER NOTES: "PROBABLE FILL TO 9 FT" LAT: 46.050885°, LONG: -85.135580°		BORING DATE: 5/22/12 FREE WATER WAS NOT OBSERVED BELOW HMA SURFACE DURING DRILLING. TEST HOLE WAS DRY AFTER COMPLETION OF DRILLING (AUGERS OUT). DRILLER NOTES: "PROBABLE FILL TO 12 FT" LAT: 46.050892°, LONG: -85.135432°		BORING DATE: 5/22/12 FREE WATER WAS NOT OBSERVED BELOW HMA SURFACE DURING DRILLING. TEST HOLE WAS DRY AFTER COMPLETION OF DRILLING (AUGERS OUT). DRILLER NOTES: "PROBABLE FILL TO 12 FT" LAT: 46.050890°, LONG: -85.135423°		BORING DATE: 5/22/12 FREE WATER WAS NOT OBSERVED BELOW HMA SURFACE DURING DRILLING. TEST HOLE WAS DRY AFTER COMPLETION OF DRILLING (AUGERS OUT). DRILLER NOTES: "PROBABLE FILL TO 9 FT" LAT: 46.050759°, LONG: -85.135238°			
PLAN REVISIONS NO. DATE AUTH DESCRIPTION NO. DATE AUTH DESCRIPTION		PLAN REVISIONS NO. DATE AUTH DESCRIPTION NO. DATE AUTH DESCRIPTION		MDOT NO SCALE		DRAWN BY: CHD BY: CORR BY: DATE: 6/6/12 FILE: us-2 west of cut river.dgn TSC: NEWBERRY		CS: 49023 SOIL BORING DATA US-2 WEST OF CUT RIVER SUBSURFACE INVESTIGATION	
								BROADEN SHEET	

TEST HOLE NO. 5		TEST HOLE NO. 6		TEST HOLE NO. 7		TEST HOLE NO. 8		TEST HOLE NO. 9	
LOCATION STATION: 1267402 2 FT LT OF CENTERLINE US-2 N: 407753, E: 26733071 US-2 WEST OF THE CUT RIVER GROUND SURFACE ELEVATION: 743.45 FT		LOCATION STATION: 1267402 15 FT LT OF CENTERLINE US-2 N: 407753, E: 26733071 US-2 WEST OF THE CUT RIVER GROUND SURFACE ELEVATION: 743.75 FT		LOCATION STATION: 1267402 9 FT LT OF CENTERLINE US-2 N: 407644, E: 26733040 US-2 WEST OF THE CUT RIVER GROUND SURFACE ELEVATION: 744.02 FT		LOCATION STATION: 1270402 17 FT LT OF CENTERLINE US-2 N: 407590, E: 26733050 US-2 WEST OF THE CUT RIVER GROUND SURFACE ELEVATION: 743.90 FT		LOCATION STATION: 1270402 15 FT LT OF CENTERLINE US-2 N: 407590, E: 26733050 US-2 WEST OF THE CUT RIVER GROUND SURFACE ELEVATION: 743.97 FT	
BORING DATE: 5/22/12 FREE WATER WAS NOT OBSERVED BELOW HMA SURFACE DURING DRILLING. TEST HOLE WAS DRY AFTER COMPLETION OF DRILLING (AUGERS OUT). DRILLER NOTES: "PROBABLE FILL TO 12 FT" LAT: 46.050712°, LONG: -85.135144°		BORING DATE: 5/22/12 FREE WATER WAS NOT OBSERVED BELOW HMA SURFACE DURING DRILLING. TEST HOLE WAS DRY AFTER COMPLETION OF DRILLING (AUGERS OUT). DRILLER NOTES: "PROBABLE FILL TO 12 FT" LAT: 46.050713°, LONG: -85.137100°		BORING DATE: 5/22/12 FREE WATER WAS NOT OBSERVED BELOW HMA SURFACE DURING DRILLING. TEST HOLE WAS DRY AFTER COMPLETION OF DRILLING (AUGERS OUT). DRILLER NOTES: "PROBABLE FILL TO 9 FT" LAT: 46.050912°, LONG: -85.137132°		BORING DATE: 5/22/12 FREE WATER WAS NOT OBSERVED BELOW HMA SURFACE DURING DRILLING. TEST HOLE WAS DRY AFTER COMPLETION OF DRILLING (AUGERS OUT). DRILLER NOTES: "PROBABLE FILL TO 13.5 FT" LAT: 46.050312°, LONG: -85.137372°		BORING DATE: 5/23/12 FREE WATER WAS NOT OBSERVED BELOW HMA SURFACE DURING DRILLING. TEST HOLE WAS DRY AFTER COMPLETION OF DRILLING (AUGERS OUT). DRILLER NOTES: "PROBABLE FILL TO 23 FT" LAT: 46.050424°, LONG: -85.137180°	
PLAN REVISIONS NO. DATE AUTH DESCRIPTION NO. DATE AUTH DESCRIPTION		PLAN REVISIONS NO. DATE AUTH DESCRIPTION NO. DATE AUTH DESCRIPTION		MDOT NO SCALE		DRAWN BY: CHD BY: CORR BY: DATE: 6/6/12 FILE: US-2 WEST OF CUT RIVER.dgn TSC: NEWBERRY		CS: 49023 SOIL BORING DATA US-2 WEST OF CUT RIVER SUBSURFACE INVESTIGATION	

## NOTES:



NUMBERS IN CIRCLES DENOTE NUMBER OF BLOWS REQUIRED TO DRIVE A 2" O.D., 120" L.O., SPLIT SPION SAMPLER 3 SUCCESSIVE 6" INCREMENTS USING A JAR HAMMER FALLING 30".

CONSISTENCY WAS DETERMINED BY INSPECTION OF SAMPLES AND SUBSTANTIATED BY SOILS RESISTANCE TO DRILLING TURNS.

THE SOIL BORING LOGS REPRESENT POINT INFORMATION. PRESENTATION OF THIS INFORMATION IN NO WAY IMPLIES THAT SUBSURFACE CONDITIONS ARE THE SAME AT LOCATIONS OTHER THAN THE EXACT LOCATION OF THE BORING.



1845

REPORT OF TEST  
SOIL ANALYSIS

FILE 300

BME

Control Section	
Identification	49023
Job No.	
Laboratory No.	12S-129
Date	June 18, 2012

Report of Sample of	Soil		
Date Sampled	May 24, 2012	Date Received	May 29, 2012
Source of Material	US-2, West of the Cut River		
Sampled From	TH#2, Sample 2, Sta 1267+17, 18' Left of Centerline, Depth: 15' to 16.5'		
Submitted By	S. Green, State Wide Soil Boring Crew Chief		
Intended Use	Information	Ground Elevation	743.6 ft

TEST RESULTS

	ASTM D 2487 Unified Soils Classification	Sieve		Cumulative Percent Passing	Percent Retained	Soil Constants	
		Size	Opening mm				
S I L T S	Coarse Gravel	3 inch	75.0			Liquid Limit	
		2-1/2 inch	63.0			Plasticity Index	
		2 inch	50.0			Specific Gravity	
		1-1/2 inch	37.5			Shrinkage Limit	
		1 inch	25.0			Shrinkage Ratio	
	Fine Gravel	3/4 inch	19.0				
		1/2 inch	12.70			Organic Content by Loss on Ignition, percent by weight	
		3/8 inch	9.52				
	Coarse Sand	No. 4	4.75				
		No. 8	2.36			Natural Moisture, percent by weight (MTM 407)	
	Medium Sand	No. 10	2.00	100	0		
		No. 16	1.180	99		Compaction and Density of Soils (AASHTO T-99 Method A)	
		No. 20	0.850	95		Maximum Density, dry, lb per cu ft	
		No. 30	0.600	88	14	Optimum Moisture, percent by weight	
Hydro- meter	Fine Sand	No. 40	0.425	59			
		No. 50	0.300	7		Compaction and Density of Soils (MDOT)	
		No. 60	0.250			Cone Density, lb per cu ft	
		No. 100	0.150			Moisture Content, percent by weight	
		No. 140	0.106				
	Silt- Clay	No. 200	0.075	2	84	Loss by Washing, percent	1.4
			0.050				
			0.005		2		
			0.002				
			0.001				

REMARKS: Except where noted, laboratory testing is performed in accordance with current AASHTO procedures.  
Sample was tested for information.  
SP - Poorly Graded Sand

Foundation Analysis Engineer

cc: File  
Soil Testing  
D. Endres

# Appendix C: DEQ Well Water Levels and Bedrock Depths

WATER WELL RECORD ACT 294 PA 1988			MICHIGAN DEPARTMENT OF PUBLIC HEALTH		
1 LOCATION OF WELL		3 OWNER OF WELL: <u>Mrs. Mrs. Roy Bedell</u>			
County <u>MACKINAC</u>	Township Name <u>HENDRICKS</u>	Fraction <u>1/4 NW 1/4 NW 1/4</u>	Section Number <u>11</u>	Town Number <u>42 N/A</u>	Range Number <u>7 E/W</u>
Distance And Direction from Road Intersections		Address <u>Box 57</u> <u>NAUBURNWAY, MICH</u>			
Street Address & City of Well Location Locate with "X" in section below		4 WELL DEPTH: (completion) <u>280</u> ft. Date of Completion <u>9-28-79</u>			
Sketch Map: 		5 <input type="checkbox"/> Cable tool <input checked="" type="checkbox"/> Rotary <input type="checkbox"/> Driven <input type="checkbox"/> Dig <input type="checkbox"/> Hollow rod <input type="checkbox"/> Jetted <input type="checkbox"/> Bored <input type="checkbox"/>			
		6 USE: <input checked="" type="checkbox"/> Domestic <input type="checkbox"/> Public Supply <input type="checkbox"/> Industry <input type="checkbox"/> Irrigation <input type="checkbox"/> Air Conditioning <input type="checkbox"/> Commercial <input type="checkbox"/> Test Well <input type="checkbox"/>			
		7 CASING: <input checked="" type="checkbox"/> Threaded <input type="checkbox"/> Welded <input type="checkbox"/> Height <u>Align</u> Below Surface <u>1</u> ft. FORMATION PACKED Diam. <u>PVC CEMENT</u> Weight <u>SDR21</u> lbs./ft. <u>5</u> in. to <u>1 1/4</u> ft. Depth <u>194</u> ft. Depth Driven Shoe? Yes <input type="checkbox"/> No <input checked="" type="checkbox"/>			
2 FORMATION		8 SCREEN:			
THICKNESS OF STRATUM		Type: _____ Dia: _____			
DEPTH TO BOTTOM OF STRATUM		Slot/Gauge _____ Length _____			
<u>COARSE RED SAND</u> <u>15'</u>		Set between _____ ft. and _____ ft.			
<u>COARSE BROWN SAND FINE GRAVEL</u> <u>20'</u>		Fittings: _____			
<u>MED BROWN SAND</u>		9 STATIC WATER LEVEL <u>114</u> ft. below land surface			
<u>W/A FEW FINE SAND LENSES</u> <u>120'</u>		10 PUMPING LEVEL below land surface			
<u>FINE BROWN SAND (SILTY)</u> <u>10'</u>		<u>140</u> ft. after <u>1</u> hrs. pumping <u>3</u> g.p.m.			
<u>GRAY CLAY</u> <u>10'</u>		<u>160</u> ft. after <u>1</u> hrs. pumping <u>10</u> g.p.m.			
<u>GRAVEL</u> <u>8'</u>		<u>180</u> ft. after <u>1</u> hrs. pumping <u>12</u> g.p.m.			
<u>BROKEN LIMESTONE OR</u>		11 WATER QUALITY in Parts Per Million:			
<u>DOLOMITE (WHITE/GRAY)</u> <u>7'</u>		Iron (Fe) _____ Chlorides (Cl) _____			
<u>WHITE CHERRY LIMESTONE</u> <u>14'</u>		Hardness _____ Other _____			
<u>WHITE LIMESTONE W/</u>		12 WELL HEAD COMPLETION: <input type="checkbox"/> In Approved Pit			
<u>GREEN SANDY CRYSTALS</u> <u>10'</u>		<input type="checkbox"/> Pitless Adapter <input checked="" type="checkbox"/> 12" Above Grade			
<u>WHITE/BLUE GRAY LIME</u>		13 Well Grouted? <input type="checkbox"/> Yes <input checked="" type="checkbox"/> No			
<u>OR DOLOMITE (SOME SANDY</u>		<input type="checkbox"/> Neat Cement <input type="checkbox"/> Bentonite <input type="checkbox"/>			
<u>STREAKS</u> <u>66'</u>		Depth: From _____ ft. to _____ ft.			
		14 Nearest Source of possible contamination: <u>75</u> feet <u>W</u> Direction <u>SEPTIC</u> Type			
		Well disinfected upon completion <input checked="" type="checkbox"/> Yes <input type="checkbox"/> No			
		15 PUMP: <input checked="" type="checkbox"/> Not installed			
		Manufacturer's Name _____			
		Model Number _____ HP _____ Volts _____			
		Length of Drop Pipe _____ ft. capacity _____ G.P.M.			
		Type: <input type="checkbox"/> Submersible <input type="checkbox"/> Reciprocating			
		<input type="checkbox"/> Jet <input type="checkbox"/>			
16 Remarks, elevation, source of data, etc. <u>RECOMMEND PUMP SETTING OF 190'</u>		17 WATER WELL CONTRACTOR'S CERTIFICATION: This well was drilled under my jurisdiction and this report is true to the best of my knowledge and belief. <u>Korckey Well Drilling Co</u> <u>1120</u> REGISTERED BUSINESS NAME REGISTRATION NO. Address <u>Box 191 GULLIVER, MICH 49840</u> Signed <u>John Korckey</u> <u>9-28-79</u> AUTHORIZED REPRESENTATIVE Date			

D67d

100M (Rev. 12-68)

OCT 17 1979

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# WATER WELL RECORD

ACT 294 PA 1965

MICHIGAN DEPARTMENT  
OF  
PUBLIC HEALTH

1 LOCATION OF WELL		County		Township Name		Fraction		Section Number		Town Number		Range Number	
Mackinac		Hendricks		SW 1/4		11		42		7		N	
Distance And Direction from Road Intersections													
Street address & City of Well Location													
Locate well "X" in section below													
Sketch Map:													
3 OWNER OF WELL: Mr. & Mrs. Rob't Nowak													
Address: Box 76, Nantunaw, Mich													
4 WELL DEPTH: (completed) 220 ft. Date of Completion: Oct 2 1979													
5 <input type="checkbox"/> Cable tool <input checked="" type="checkbox"/> Rotary <input type="checkbox"/> Driven <input type="checkbox"/> Dug													
<input type="checkbox"/> Hollow rod <input type="checkbox"/> Jolted <input type="checkbox"/> Bored													
6 USE: <input type="checkbox"/> Domestic <input type="checkbox"/> Public Supply <input type="checkbox"/> Industry													
<input type="checkbox"/> Irrigation <input type="checkbox"/> Air Conditioning <input checked="" type="checkbox"/> Commercial													
<input type="checkbox"/> Test Well													
7 CASING: Threaded <input type="checkbox"/> Welded <input type="checkbox"/> Height Above/Below Surface: 9 ft.													
Diam. PVC Cement 5 in. to 19 in. Depth Weight SDR2 lbs./ft.													
Slot/Gauge _____ Length _____													
Set between _____ ft. and _____ ft.													
Fittings: _____													
8 STATIC WATER LEVEL: 112 ft. below land surface													
10 PUMPING LEVEL below land surface: 122 ft. after 2 hrs. pumping 8 G.P.M.													
_____ ft. after _____ hrs. pumping _____ G.P.M.													
11 WATER QUALITY in Parts Per Million:													
Iron (Fe) _____ Chlorides (Cl) _____													
Hardness _____ Other _____													
12 WELL HEAD COMPLETION: <input type="checkbox"/> Is Approved Pat <input checked="" type="checkbox"/> 12" Above Grade													
<input type="checkbox"/> Pitless Adapter <input checked="" type="checkbox"/> No													
13 Well Grouted? <input type="checkbox"/> Yes <input checked="" type="checkbox"/> No													
<input type="checkbox"/> Neat Cement <input type="checkbox"/> Bentonite <input type="checkbox"/> _____													
Depth: From _____ ft. to _____ ft.													
14 Nearest Source of possible contamination: 100 feet E Direction SCPTC Type													
Well disinfected upon completion <input checked="" type="checkbox"/> Yes <input type="checkbox"/> No													
15 PUMP: <input checked="" type="checkbox"/> Not installed													
Manufacturer's Name _____													
Model Number _____ HP _____ Volts _____													
Length of Drop Pipe _____ ft. capacity _____ G.P.M.													
Type: <input type="checkbox"/> Submersible <input type="checkbox"/> Jet <input type="checkbox"/> Reciprocating													
16 Remarks, elevation, source of data, etc. Set Pump @ 140'													
17 WATER WELL CONTRACTOR'S CERTIFICATION:													
This well was drilled under my jurisdiction and this report is true to the best of my knowledge and belief.													
Kopetski Well Drilling Co. 1120													
REGISTERED BUSINESS NAME REGISTRATION NO.													
Address: Box 191 SULLIVER, Mich 49840													
Signed: J. Kopetski Date: 10-2-79													
AUTHORIZED REPRESENTATIVE													

067d

10/10M (Rev. 12-68)

OCT 17 1979

GEOLOGICAL SURVEY COPY



# Water Well And Pump Record



Completion is required under authority of Part 127 Act 368 PA 1978.

Failure to comply is a misdemeanor.

Import ID: 49420711001

Tax No: 4900501100610	Permit No:	County: Mackinac	Township: Hendricks
<b>Well ID: 490000001075</b>  Elevation: 750 ft.  Latitude: 46.0535104981  Longitude: -85.1524138294  Method of Collection: Interpolation-Map		Town/Range: 42N 07W	Section: 11
		Well Status:	WSSN:
		Source ID/Well No:	
		Distance and Direction from Road Intersection:	
		Well Owner: BEDELL, ROY	
		Well Address: BOX 57 NAUBINWAY, MI 49762	Owner Address: BOX 57 NAUBINWAY, MI 49762

Drilling Method: Rotary Well Depth: 280.00 ft. Well Type: Replacement Casing Type: PVC plastic Casing Joint: Welded Casing Fitting: None  Diameter: 5.00 in. to 194.00 ft. depth  Borehole:	Pump Installed: No Pressure Tank Installed: No Pressure Relief Valve Installed: No																																										
Static Water Level: 114.00 ft. Below Grade Well Yield Test: Pumping level 140.00 ft. after 1.00 hrs. at 3 GPM Yield Test Method: Unknown Pumping level 160.00 ft. after 1.00 hrs. at 10 GPM	<table border="1"> <thead> <tr> <th>Formation Description</th> <th>Thickness</th> <th>Depth to Bottom</th> </tr> </thead> <tbody> <tr><td>Red Sand Coarse</td><td>15.00</td><td>15.00</td></tr> <tr><td>Brown Sand &amp; Gravel Coarse</td><td>20.00</td><td>35.00</td></tr> <tr><td>Brown Sand Medium</td><td>120.00</td><td>155.00</td></tr> <tr><td>Brown Sand Fine Silty</td><td>10.00</td><td>165.00</td></tr> <tr><td>Gray Clay</td><td>10.00</td><td>175.00</td></tr> <tr><td>Gravel</td><td>8.00</td><td>183.00</td></tr> <tr><td>Limestone Broken</td><td>7.00</td><td>190.00</td></tr> <tr><td>White Limestone</td><td>14.00</td><td>204.00</td></tr> <tr><td>White Limestone W/Sand</td><td>10.00</td><td>214.00</td></tr> <tr><td>White Limestone Sandy</td><td>66.00</td><td>280.00</td></tr> <tr><td> </td><td> </td><td> </td></tr> <tr><td> </td><td> </td><td> </td></tr> <tr><td> </td><td> </td><td> </td></tr> </tbody> </table>	Formation Description	Thickness	Depth to Bottom	Red Sand Coarse	15.00	15.00	Brown Sand & Gravel Coarse	20.00	35.00	Brown Sand Medium	120.00	155.00	Brown Sand Fine Silty	10.00	165.00	Gray Clay	10.00	175.00	Gravel	8.00	183.00	Limestone Broken	7.00	190.00	White Limestone	14.00	204.00	White Limestone W/Sand	10.00	214.00	White Limestone Sandy	66.00	280.00									
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Red Sand Coarse	15.00	15.00																																									
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Gray Clay	10.00	175.00																																									
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Limestone Broken	7.00	190.00																																									
White Limestone	14.00	204.00																																									
White Limestone W/Sand	10.00	214.00																																									
White Limestone Sandy	66.00	280.00																																									
Screen Installed: No Intake: Bedrock Well	Geology Remarks:																																										
Well Grouted: No																																											
Wellhead Completion: Other, 12 inches above grade																																											
Nearest Source of Possible Contamination: Type: Septic tank Distance: 75 ft. Direction: West	Drilling Machine Operator Name: Employment: Unknown																																										

## Appendix D: Visual Walkthrough

*Dip on Roadway* (color represents specified zoom)







*Sinkhole*





*Slough May 2012*



*Water Flow 2012 (Looking North)*



*Small Stream 2012, 2013 respectively (Looking South)*



*Slough June 2013*





*Slough April 2014*



*Water Flow April 2014 (Looking North)*



*Stream April 2014*


















## *Artesian Conditions*















## Appendix E: Material Properties

### *Original LEM Model Hydraulic Properties*

Material Name	Color	Model	KS (ft/s)	K2/K1	K1 Angle (deg)	WC (ft <sup>3</sup> /ft <sup>3</sup> )	Soil Type
Med, Fine Sand w/ Gravel		Simple	0.0009	1	0	0.4	Sand
Med, Fine Dark Brown Sand		Simple	6.6e-006	1	0	0.4	Sand
Med, Fine & Med Sand		Simple	3e-006	1	0	0.4	Sand
Very Loose Fine Sand		Simple	0.00066	1	0	0.4	Sand
Very Loose Dark Fine Sand w/ Topsoil		Simple	0.00066	1	0	0.4	Sand
Loose Fine Sand		Simple	6.6e-005	1	0	0.4	Sand
Loose Fine Sand w/ Gravel		Simple	0.00066	1	0	0.4	Sand
Concrete		Simple	3.28e-010	1	0	0.4	General
HMA		Simple	3.28e-010	1	0	0.4	General
Dense, Fine Sand		Simple	6.6e-007	1	0	0.4	Sand
Med, Fine Sand		Simple	6.6e-006	1	0	0.4	Sand
Limestone		Simple	3.28e-009	1	0	0.4	General
Dolostone		Simple	3.28e-012	1	0	0.4	General
Shale		Simple	3.28e-013	1	0	0.4	General
Sandstone		Simple	3.28e-009	1	0	0.4	General
Clay		Simple	3e-010	1	0	0.4	Clay

### *DEQ LEM Model Hydraulic Properties*

Material Name	Color	Model	KS (ft/s)	K2/K1	K1 Angle (deg)	WC (ft <sup>3</sup> /ft <sup>3</sup> )	Soil Type
Red Sand Coarse		Simple	3e-008	1	0	0.4	Sand
Brown Sand & Gravel Coarse		Simple	0.0007	1	0	0.4	Sand
Brown Sand Medium		Simple	1.8e-008	1	0	0.4	Sand
Brown Sand Fine Silty		Simple	6e-005	1	0	0.4	Silt
Gray Clay		Simple	3e-010	1	0	0.4	Clay
Gravel		Simple	0.06	1	0	0.4	General
Limestone Broken		Simple	0.003	1	0	0.4	General
White Limestone		Simple	3e-013	1	0	0.4	General
White Limestone W/Sand		Simple	3e-010	1	0	0.4	General
White Limestone Sandy		Simple	3e-009	1	0	0.4	General

















*Original LEM Model Material Properties*

Material Name	Color	Unit Weight (lbs/ft <sup>3</sup> )	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	m	s	a	Water Surface	Hu Type	Hu	Phi b (deg)	Air Entry (psf)
Med, Fine Sand w/ Gravel		112	Mohr-Coulomb	0.02	36					Water Surface	Custom	1	0	0
Med, Fine Dark Brown Sand		107	Mohr-Coulomb	0.02	34					Water Surface	Custom	1	0	0
Med, Fine & Med Sand		97	Mohr-Coulomb	0.02	35					Water Surface	Custom	1	0	0
Very Loose Fine Sand		85	Mohr-Coulomb	0.02	27					Water Surface	Custom	1	0	0
Very Loose Dark Fine Sand w/ Topsoil		82	Mohr-Coulomb	0.02	26					Water Surface	Custom	1	0	0
Loose Fine Sand		90	Mohr-Coulomb	0.02	29					Water Surface	Custom	1	0	0
Loose Fine Sand w/ Gravel		95	Mohr-Coulomb	0.02	30					Water Surface	Custom	1	0	0
Concrete		150	Generalised Hoek-Brown			626600	3.52122	0.00386592	0.505734	Water Surface	Custom	1	0	0
HMA		145	Generalised Hoek-Brown			600000	3.52122	0.00386592	0.505734	Water Surface	Custom	1	0	0
Dense, Fine Sand		110	Mohr-Coulomb	0.02	37					Water Surface	Custom	1	0	0
Med, Fine Sand		105	Mohr-Coulomb	0.02	35					Water Surface	Custom	1	0	0
Limestone		153	Generalised Hoek-Brown			1.5e+006	1.67677	0.00386592	0.505734	Water Surface	Custom	1	0	0
Dolostone		175	Generalised Hoek-Brown			3.5e+006	1.5091	0.00386592	0.505734	Water Surface	Custom	1	0	0
Shale		172	Generalised Hoek-Brown			700000	1.00606	0.00386592	0.505734	Water Surface	Custom	1	0	0
Sandstone		137	Generalised Hoek-Brown			1.5e+006	2.85051	0.00386592	0.505734	Water Surface	Custom	1	0	0
Clay		115	Mohr-Coulomb	312.5	0					Water Surface	Custom	1	0	0











*DEQ LEM Model Material Properties*

Material Name	Color	Unit Weight (lbs/ft <sup>3</sup> )	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	m	s	a	Water Surface	Hu Type	Phi b (deg)	Air Entry (deg)
Red Sand Coarse		90	Mohr-Coulomb	0.02	36					Water Surface	Constant	0	0
Brown Sand & Gravel Coarse		103	Mohr-Coulomb	0.02	37					Water Surface	Constant	0	0
Brown Sand Medium		75	Mohr-Coulomb	0.02	35					Water Surface	Constant	0	0
Brown Sand Fine Silty		93.6	Mohr-Coulomb	459.5	32					Water Surface	Constant	0	0
Gray Clay		109	Mohr-Coulomb	683	0					Water Surface	Constant	0	0
Gravel		120	Mohr-Coulomb	0	40					Water Surface	Constant	0	0
Limestone Broken		97	Generalised Hoek-Brown			1.5e+006	0.983084	0.00101905	0.51302	Water Surface	Constant	0	0
White Limestone		163	Generalised Hoek-Brown			4e+006	4.95753	0.0319225	0.501463	Water Surface	Constant	0	0
White Limestone W/Sand		156	Generalised Hoek-Brown			3.5e+006	2.32112	0.00602938	0.504342	Water Surface	Constant	0	0
White Limestone Sandy		120	Generalised Hoek-Brown			3.25e+006	1.01701	0.000815988	0.514908	Water Surface	Constant	0	0

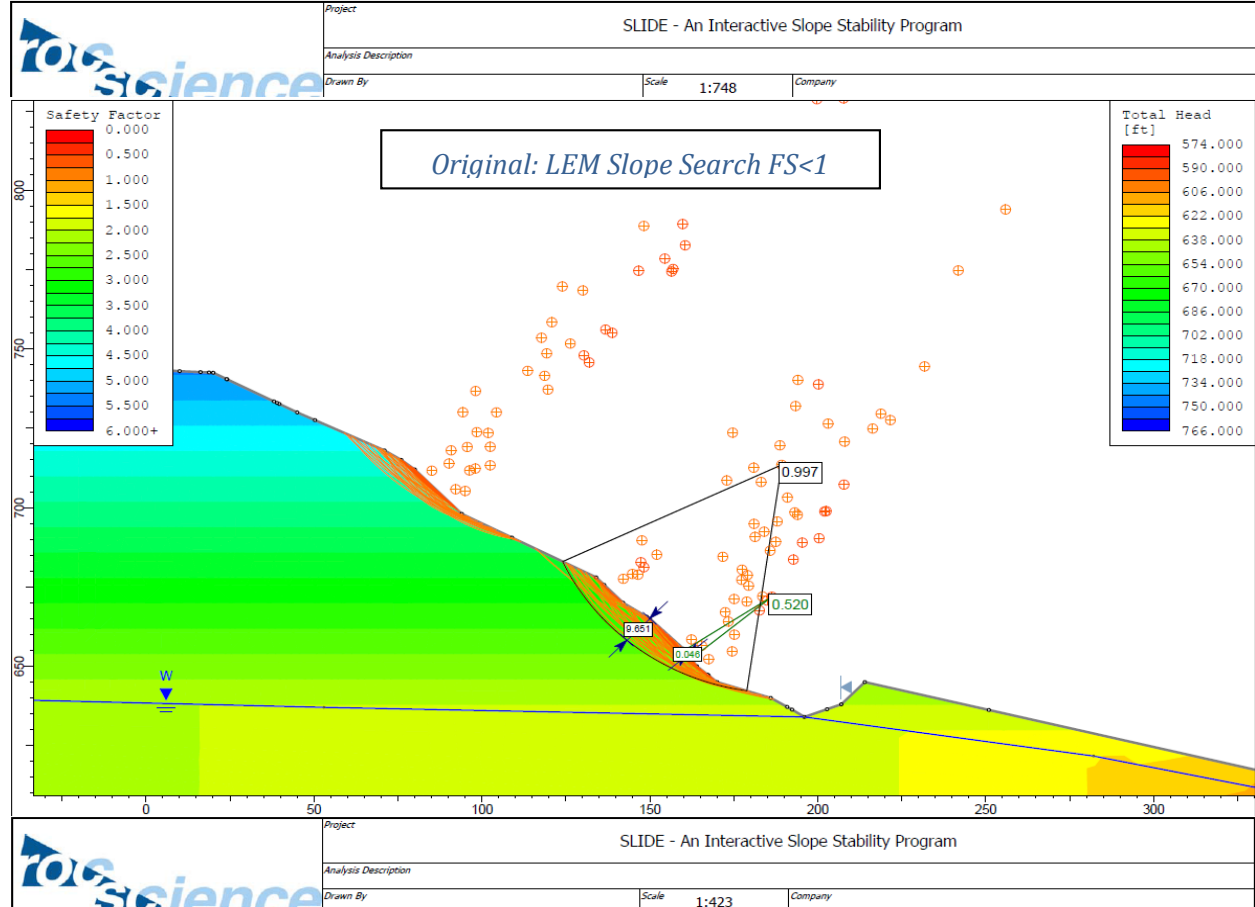
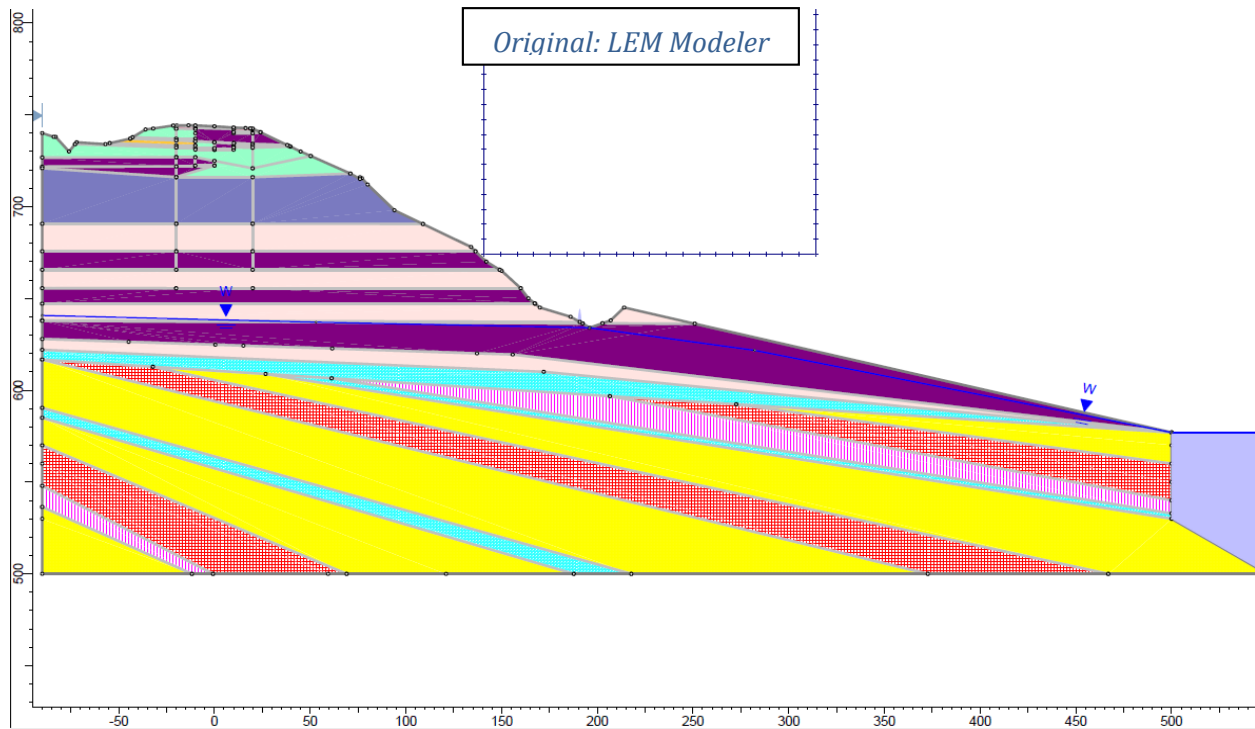
## Original FEA Model Material Properties

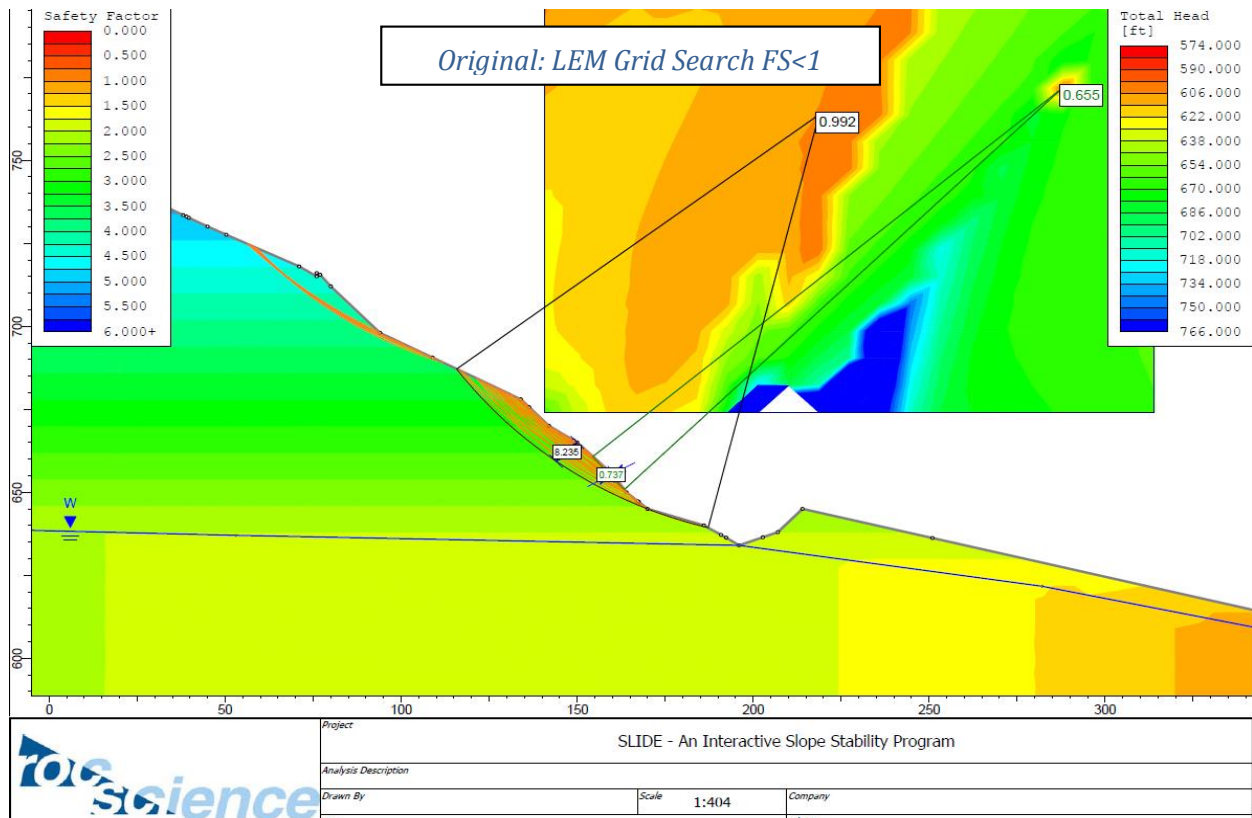
Material Name	Color	Initial Element Loading	Unit Weight (lbs/ft3)	Elastic Type	Young's Modulus (psf)	Poisson's Ratio	Failure Criterion	Material Type	Tensile Strength (psf)	Tensile Strength (residual) (psf)	Dilation Angle (deg)	Friction Angle (peak) (deg)	Friction Angle (residual) (deg)	Cohesion (peak) (psf)	Cohesion (residual) (psf)	Intact Compressive Strength (psf)	mb (peak)	mb (residual)	s (peak)	s (residual)	a (peak)	a (residual)	Dilation Parameter	Piezo Line	Hu	Ru
Med, Fine Sand w/ Gravel		Field Stress and Body Force	112	Isotropic	1.2531e+006	0.3	Mohr Coulomb	Plastic	0.02	0	0	36	36	0.02	0.02									None	0	
Med, Fine Dark Brown Sand		Field Stress and Body Force	107	Isotropic	1.2531e+006	0.3	Mohr Coulomb	Plastic	0.02	0	0	34	34	0.02	0.02									None	0	
Med, Fine & Med Sand		Field Stress and Body Force	97	Isotropic	1.2531e+006	0.3	Mohr Coulomb	Plastic	0.02	0	0	35	35	0.02	0.02									None	0	
Very Loose Fine Sand		Field Stress and Body Force	85	Isotropic	300000	0.25	Mohr Coulomb	Plastic	0.02	0	0	27	27	0.02	0.02									None	0	
Very Loose Dark Fine Sand w/ Topsoil		Field Stress and Body Force	82	Isotropic	300000	0.25	Mohr Coulomb	Plastic	0.02	0	0	26	26	0.02	0.02									None	0	
Loose Fine Sand		Field Stress and Body Force	90	Isotropic	355045	0.28	Mohr Coulomb	Plastic	0.02	0	0	29	29	0.02	0.02									None	0	
Loose Fine Sand w/ Gravel		Field Stress and Body Force	95	Isotropic	400000	0.25	Mohr Coulomb	Plastic	0.02	0	0	30	30	0.02	0.02									None	0	
Concrete		Field Stress and Body Force	150	Isotropic	1e+006	0.4	Generalized Hoek-Brown	Plastic								626600	3.52122	3.52122	0.003865920	0.003865920	0.505734	0.505734	0	None	0	
HMA		Field Stress and Body Force	145	Isotropic	1e+006	0.4	Generalized Hoek-Brown	Plastic								600000	3.52122	3.52122	0.003865920	0.003865920	0.505734	0.505734	0	None	0	
Dense, Fine Sand		Field Stress and Body Force	110	Isotropic	1.04425e+006	0.4	Mohr Coulomb	Plastic	0.02	0	0	37	37	0.02	0.02									None	0	
Med, Fine Sand		Field Stress and Body Force	105	Isotropic	1.2531e+006	0.3	Mohr Coulomb	Plastic	0.02	0	0	35	35	0.02	0.02									None	0	
Limestone		Field Stress and Body Force	153	Isotropic	1.15e+009	0.33	Generalized Hoek-Brown	Plastic								1.5e+006	1.67677	1.67677	0.003865920	0.003865920	0.505734	0.505734	0	None	0	
Dolostone		Field Stress and Body Force	175	Isotropic	1.18e+009	0.3	Generalized Hoek-Brown	Plastic								3.5e+006	1.5091	1.5091	0.003865920	0.003865920	0.505734	0.505734	0	None	0	
Shale		Field Stress and Body Force	172	Isotropic	1.04e+009	0.25	Generalized Hoek-Brown	Plastic								700000	1.00606	1.00606	0.003865920	0.003865920	0.505734	0.505734	0	None	0	
Sandstone		Field Stress and Body Force	137	Isotropic	3.13e+008	0.29	Generalized Hoek-Brown	Plastic								1.5e+006	2.85051	2.85051	0.003865920	0.003865920	0.505734	0.505734	0	None	0	
Clay		Field Stress and Body Force	115	Isotropic	208850	0.45	Mohr Coulomb	Plastic	312.5	0	0	0	0	312.5	312.5									1	1	

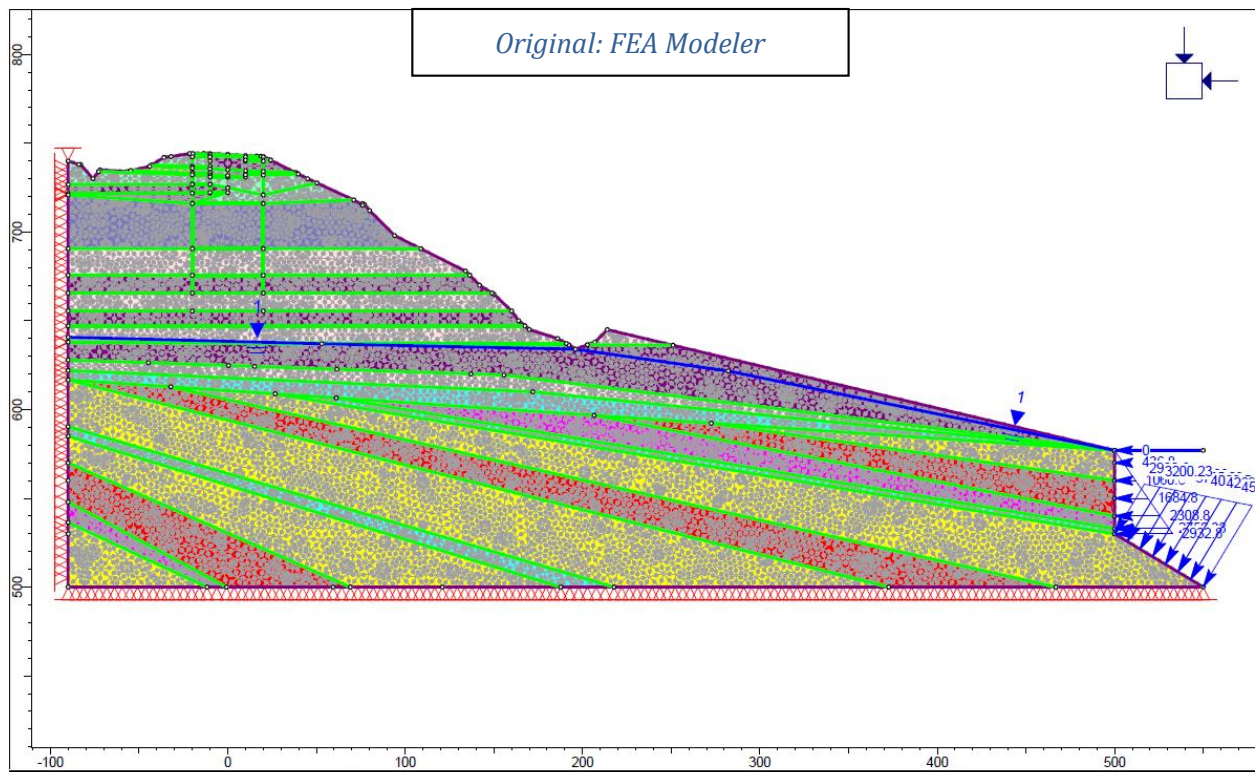
### DEQ FEA Model Material Properties

Material Name	Color	Initial Element Loading	Unit Weight (lbs/ft <sup>3</sup> )	Elastic Type	Young's Modulus (psf)	Poisson's Ratio	Failure Criterion	Material Type	Tensile Strength (psf)	Tensile Strength (residual) (psf)	Dilation Angle (deg)	Friction Angle (peak) (deg)	Friction Angle (residual) (deg)	Cohesion (peak) (psf)	Cohesion (residual) (psf)	Intact Compressive Strength (psf)	mb (peak)	mb (residual)	s (peak)	s (residual)	a (peak)	a (residual)	Dilation Parameter	Piezo Line	Hu
Red Sand Coarse		Field Stress and Body Force	90	Isotropic	313275	0.4	Mohr Coulomb	Plastic	0.02	0	0	36	36	0.02	0.02									1	1
Brown Sand & Gravel Coarse		Field Stress and Body Force	103	Isotropic	1.61859e+006	0.3	Mohr Coulomb	Plastic	0.02	0	0	37	37	0.02	0.02									1	1
Brown Sand Medium		Field Stress and Body Force	75	Isotropic	730975	0.3	Mohr Coulomb	Plastic	0.02	0	0	35	35	0.02	0.02									1	1
Brown Sand Fine Silty		Field Stress and Body Force	93.6	Isotropic	939825	0.25	Mohr Coulomb	Plastic	459.5	0	0	32	32	459.5	459.5									1	1
Gray Clay		Field Stress and Body Force	109	Isotropic	208850	0.45	Mohr Coulomb	Plastic	683	0	0	0	0	683	683									1	1
Gravel		Field Stress and Body Force	120	Isotropic	2.5062e+006	0.3	Mohr Coulomb	Plastic	0	0	0	40	40	0	0									1	1
Limestone Broken		Field Stress and Body Force	97	Isotropic	3.13e+008	0.18	Generalized Hoek-Brown	Plastic								1.5e+006	0.983084	0.983084	0.00101905	0.00101905	0.51302	0.51302	0	1	1
White Limestone		Field Stress and Body Force	163	Isotropic	1.15e+009	0.33	Generalized Hoek-Brown	Plastic								4e+006	4.95753	4.95753	0.0319225	0.0319225	0.501463	0.501463	0	1	1
White Limestone W/Sand		Field Stress and Body Force	156	Isotropic	6.27e+008	0.28	Generalized Hoek-Brown	Plastic								3.5e+006	2.32112	2.32112	0.00602938	0.00602938	0.504342	0.504342	0	1	1
White Limestone Sandy		Field Stress and Body Force	137	Isotropic	7.31e+008	0.25	Generalized Hoek-Brown	Plastic								3.25e+006	1.01701	1.01701	0.000815988	0.000815988	0.514908	0.514908	0	1	1

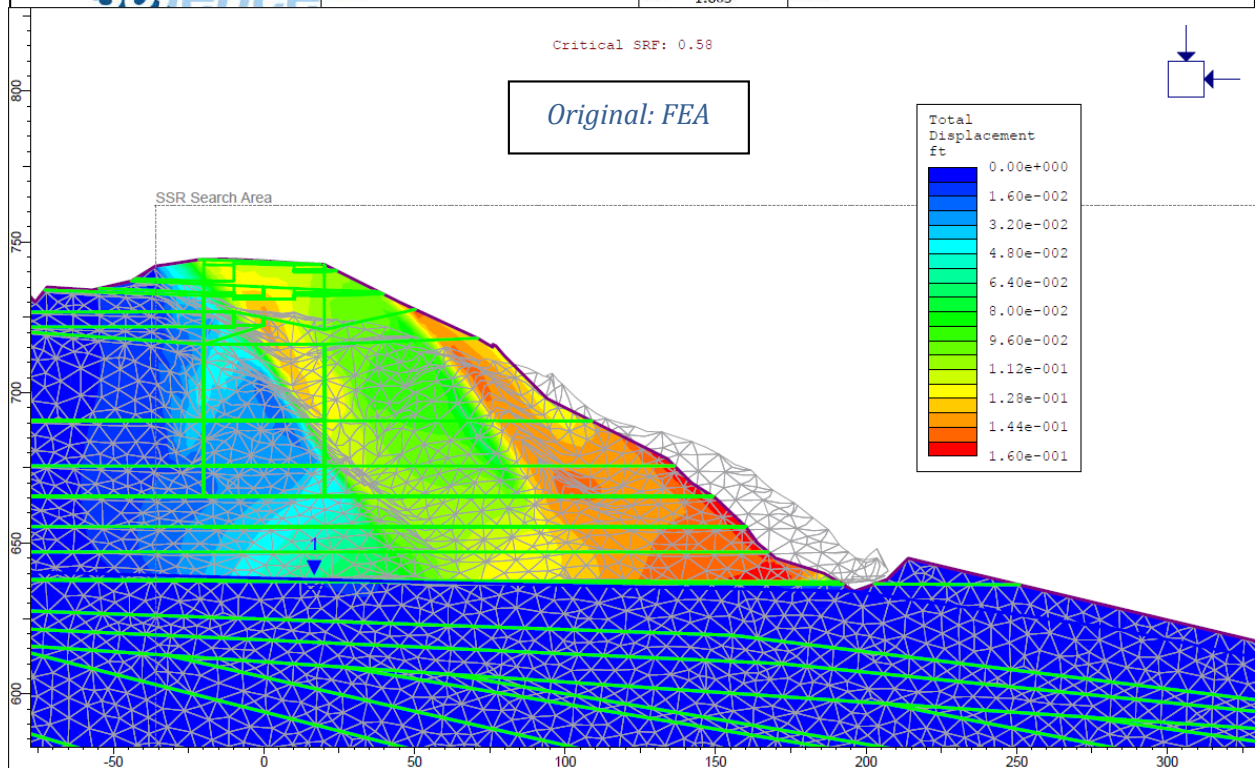
## Appendix F: Stability Analysis Models



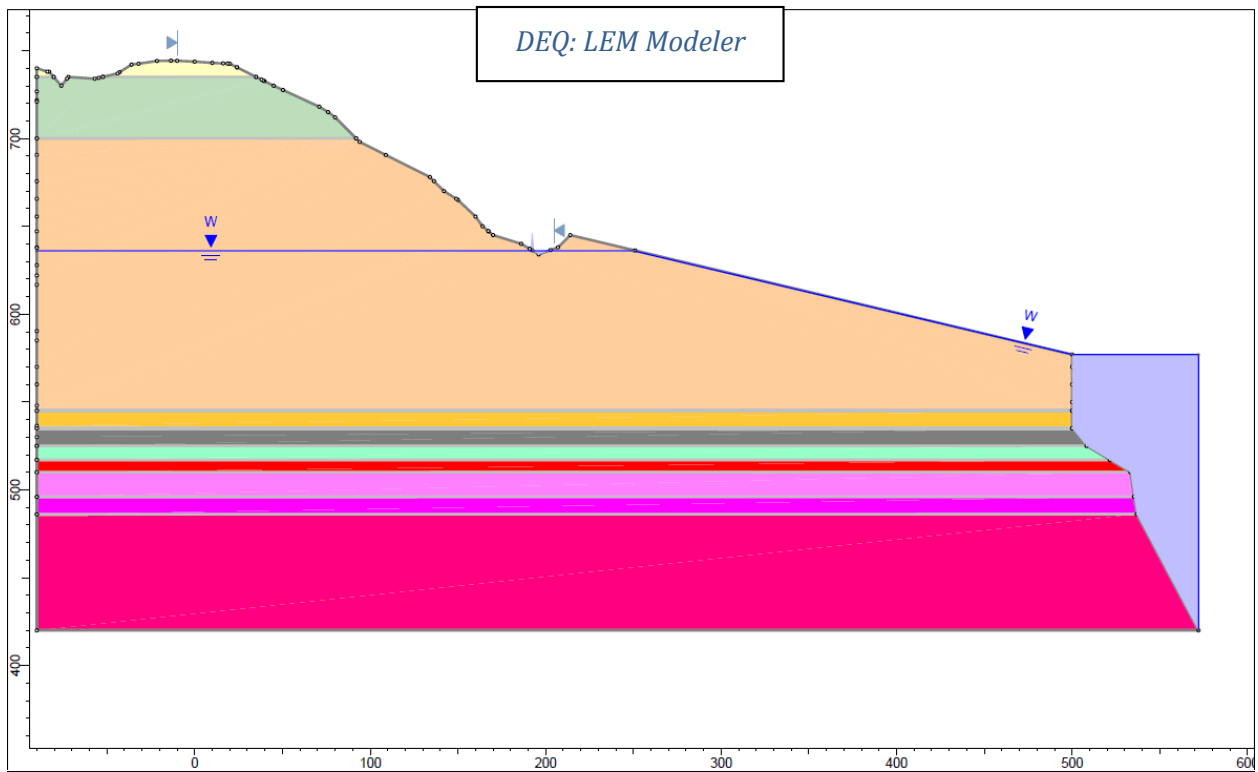




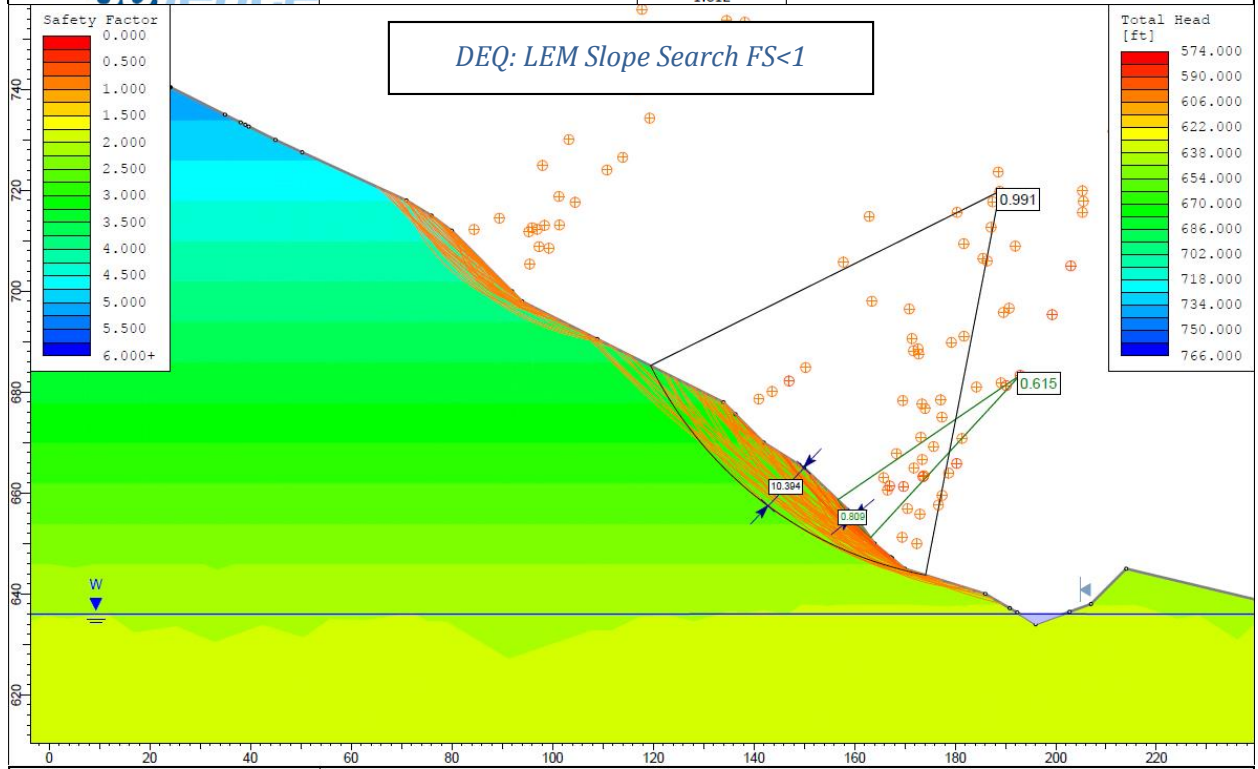
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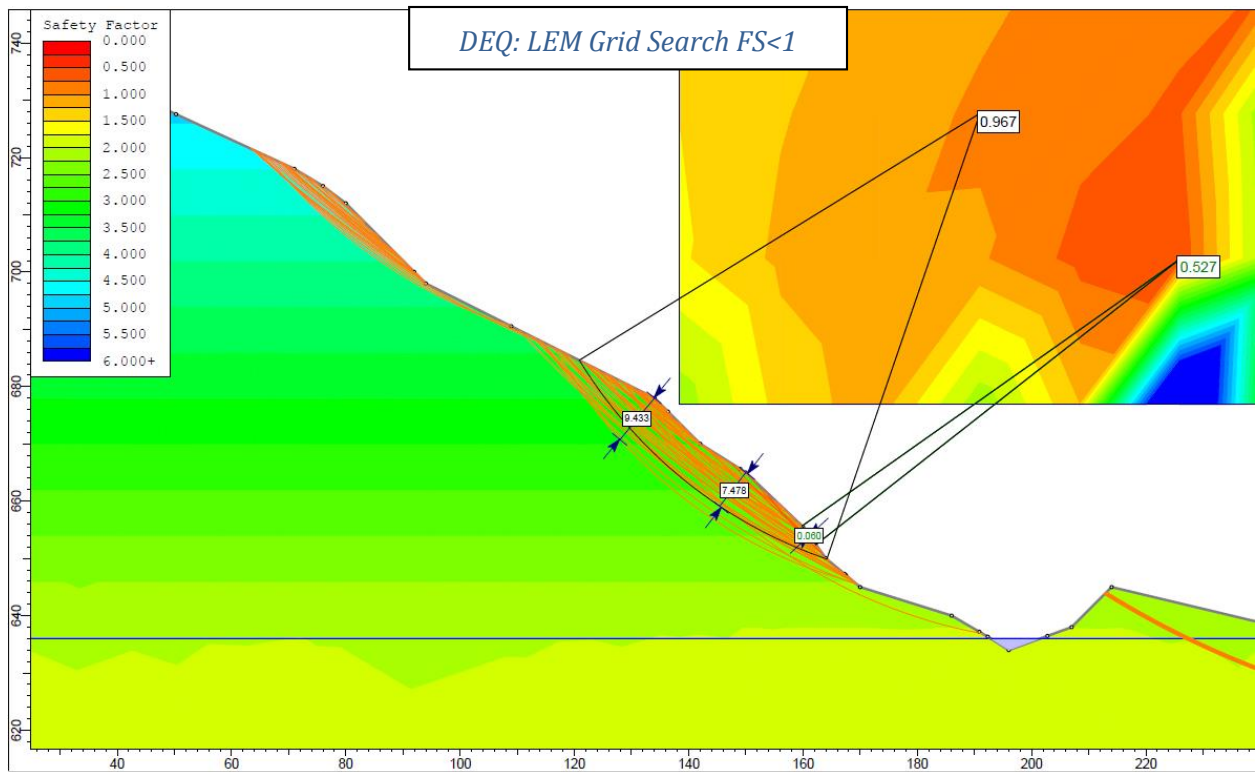
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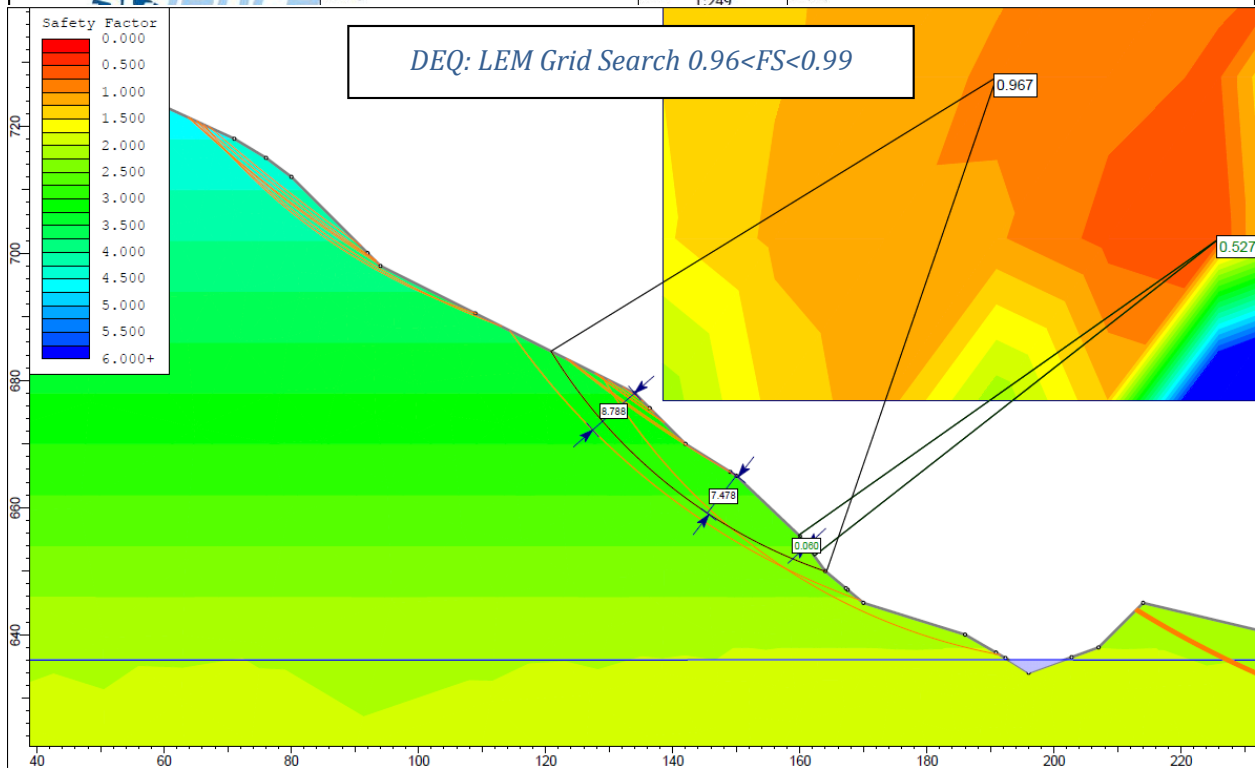
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	Analysis Description		
Drawn By	Scale	Company	
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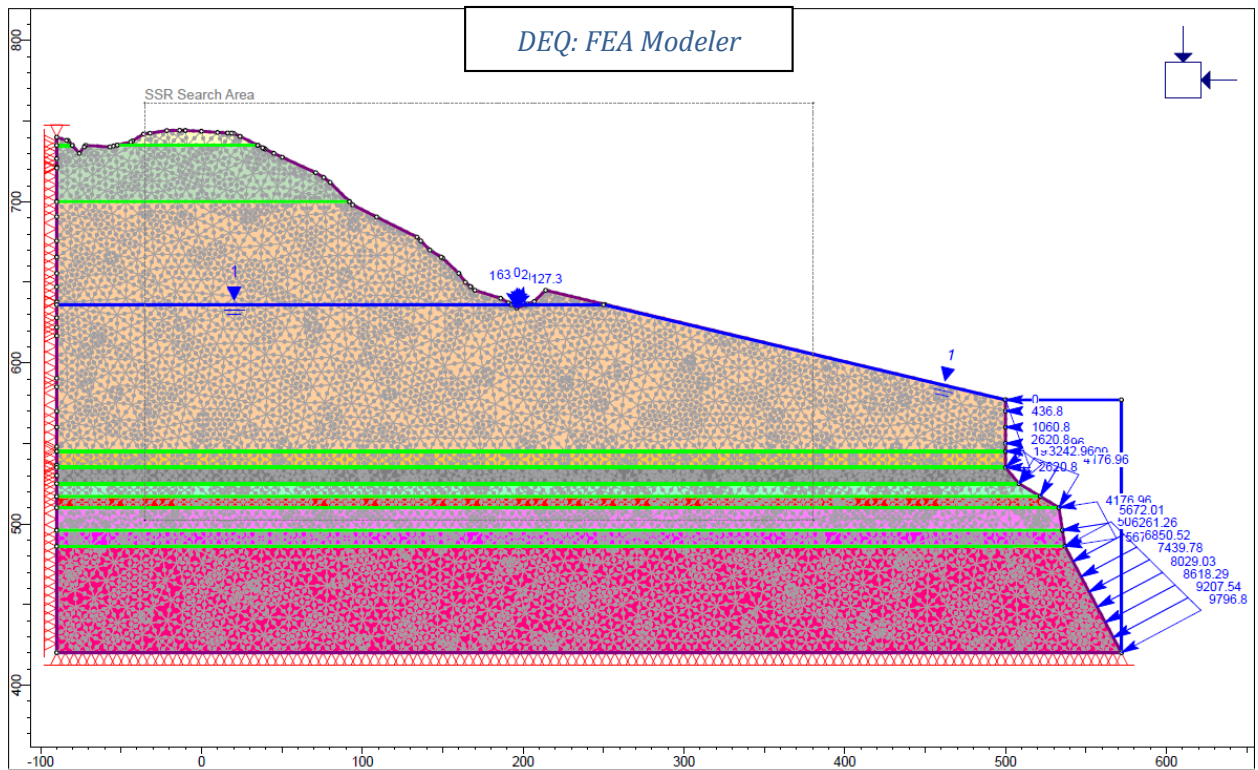
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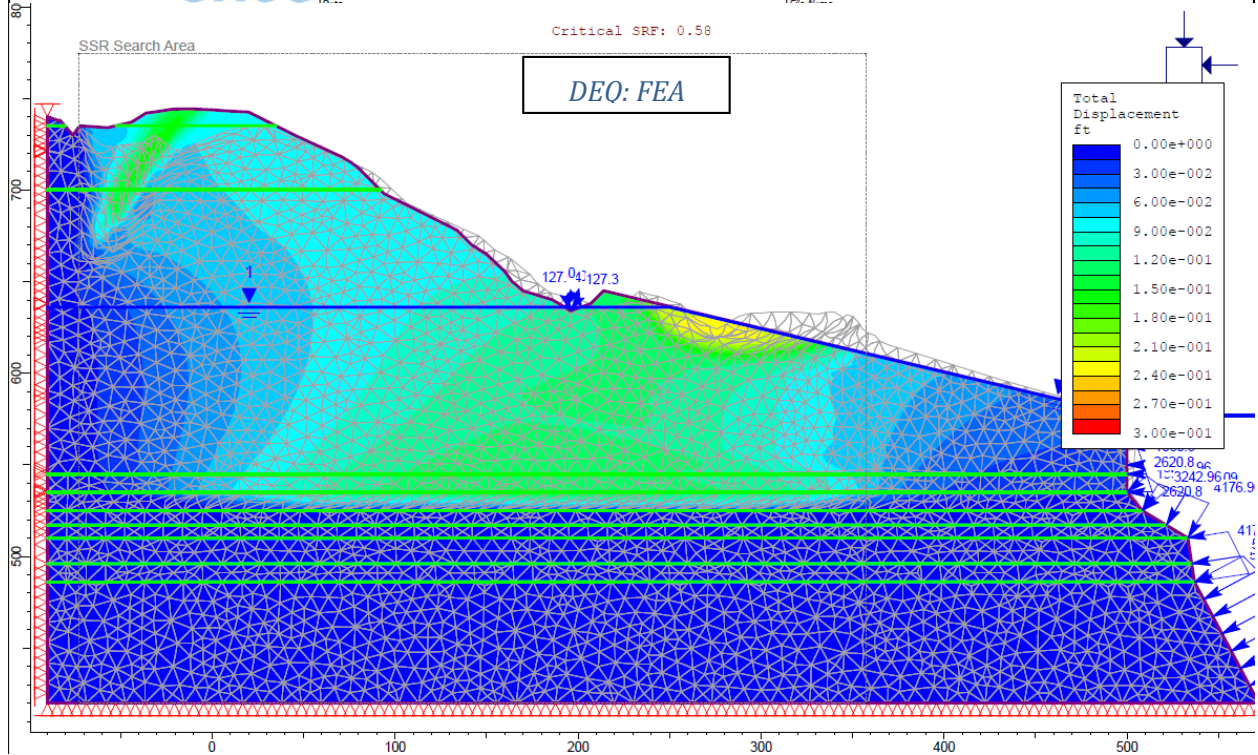
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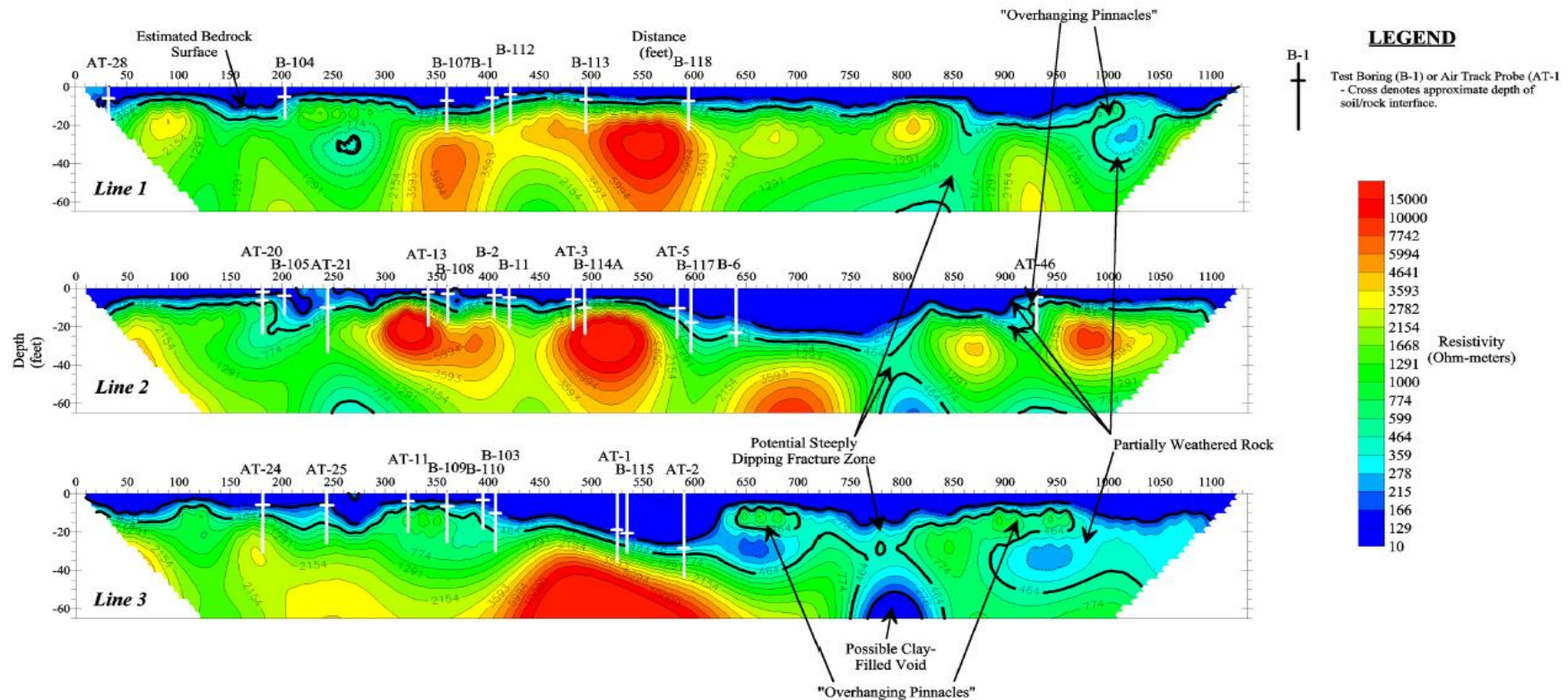
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		1:779	

## Appendix G: ERM (Electrical Resistivity Method) Example

### Bedrock Mapping in Karst Terrain



- › Boring lines show approximate locations and depths. Most borings were not located directly over the resistivity traverse and therefore some variation from the resistivity profile should be expected.
- › Resistivity measurements were obtained at discrete electrode spacings.
- › Resistivity contours were interpolated from the data points.
- › Data was recorded in a dipole-dipole electrode array, using a Sting/Swift automatic resistivity system, with electrodes at 5 meter intervals.

- › The three two-dimensional resistivity profiles presented show correlation between test borings and air track probes and the estimated bedrock surface from resistivity data in karst terrain. The thickened contours show estimated dissintegrated rock and bedrock surfaces, respectively. Areas of very high resistivity were probed and determined to be dry, competent limestone.

Courtesy of  
**Schnabel  
Engineering**

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