Crookston Slope Stability

BYU CE EN Capstone Project, Final Report

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Executive Summary

Overview

A slope in Crookston, Minnesota has issues with mass wasting. Obelisk Engineering was requested to provide a design to prevent future movement of the slope. A solution was developed based on analysis using an educational version of a commonly used slope stability program, UTEXAS. The methodology and results as determined by this analysis are presented in this final report.

Project Team

Obelisk Engineering is a student team in the process of completing the capstone project according to course requirements for the Brigham Young University Department of Civil & Environmental Engineering. This team consists of the following individuals:

-Kenneth Hunter
-Alex Arndt
-Shawn Crawley

In addition, other individuals were involved in the design process to provide input based on experience and typical methodology as seen in the field. These individuals include:

-Kristen Ulmer- Graduate Mentor
-Norm Jones- Faculty Advisor
-Rick Deschamps- Nicholson Construction

Design Process

Utilizing the data that was provided by Nicholson Construction, Obelisk Engineering planned for several stages for the approach to the final design. The final order of these phases was as follows:

-Analyze site data and characterize the underlying problem, determine an appropriate factor of safety
-Identify possible solutions to stabilize the slope that fit within prescribed MnDot restrictions
-Select an appropriate solution and provide a preliminary design
-analyze options and potential alternatives to presented design
-provide, where possible, cost estimates and construction schedules for the proposed solution

Disclaimer

The work presented in the report does not necessarily represent any views of Brigham Young University or its faculty. As the work was completed by undergraduate students there is no engineering stamp to certify this work, and it is provided as is.

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Introduction

Objective

A slope failure occurrence in Crookston, Minnesota adjacent to Trunk Highway 2 requires a design-build solution in order to stabilize the slope. The Minnesota Department of Transportation (MnDOT) is contracting for this solution. MnDOT has set multiple objectives and required outcomes for the design. The major required outcomes were: Protecting Truck Highway 2, limiting the impact to the surrounding area both during and after construction, and preventing future slope movement. This work includes a solution for a feasible slope stabilization design. The analysis method will be discussed in depth, and the results pertaining to the analysis presented. Design variables as well as other pertinent design information for the specified solution will be presented in the results section of this report. Estimates for the design costs are also contained herein.

Site Description

The city of Crookston is located in Polk County in northwest Minnesota, and has a population of around 8000 people. Trunk Highway 2 passes through the center of the city, and is a major arterial highway for transportation in the area. The Red Lake River passes through the city, winding around residential areas of the city. The slope failure is located between Red Lake River and Trunk Highway 2 where these two features are parallel to each other, as can be seen in Figure 1. The distance from the highway to the river at its closest point is 260 feet, and the distance from the highway to the top of the failure is 35 feet. Almost 800 feet of slope adjacent to the highway has failed. The extents of the slope failure for this site can be observed in Figure 2.

Figure 1: Location of slope failure

Figure 2: Limits of slope failure

Geological Setting

The slope failure events that have taken place in Crookston are related to the unique geology of the Northern Minnesota area. This region is one of the youngest geological areas of the contiguous United States; as the Red River has only been running its current course for a few thousand years. This area was the location of the ancient glacial Lake Agassiz. After the most recent ice age, Lake Agassiz was a gigantic lake several times the size of the current great lakes. During the post glacial period the Red River Valley (where Crookston is located) became a lake bed where sediments from the glacial erosion slowly settled out. Over a period of time this created a thick clay layer, with a typical depth of 80 feet. This layer was later covered by various alluvial deposits (Schwert 2003). Banks along the Red River have often eroded to the point that the underlying clay layer has been exposed. This erosion typically occurs on the outside bends of the rivers where higher velocity water are present. Over time this erosion process forms slopes that are susceptible to mass movement as soil is removed from the toe of the slope (Dasenbrock 2010). These types of slopes are especially at risk for slumping, as gravitational force pushing down on the soil can overcome shear and frictional resisting forces in the soil. The University of Minnesota has done research into this type of occurrence as it is typical in the region. Figure 3 was provided by such a report, and is a visualization of the type of failure that is occurring at the Crookston slide site.

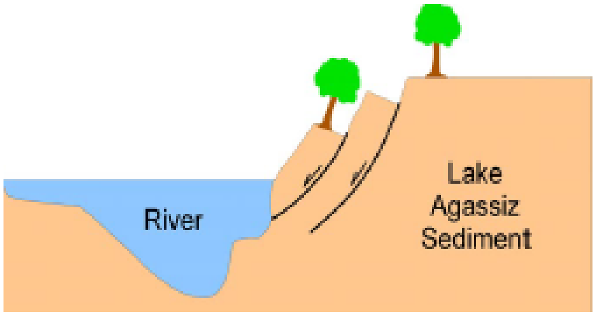

Figure 3: Slope slumping in Red River Valley

The geological formation that has been the source of most of the problems in Crookston is named the Huot Formation. This formation has an extremely high clay content for a glacial deposit. The formation also has very low shear strengths which has attributed to the mass wasting at this site (Arndt 1977). Thus the geological profile of the area is the cause for the failures associated with the Crookston site. Figure 4 shows the alluvial deposits as the shaded yellow area, the Huot clay layer as the shaded peach area, and the glacial till as the spotted orange area. Several parameters for each soil layer, such as unit weight and friction angle, are provided in Table 1 following Figure 4.

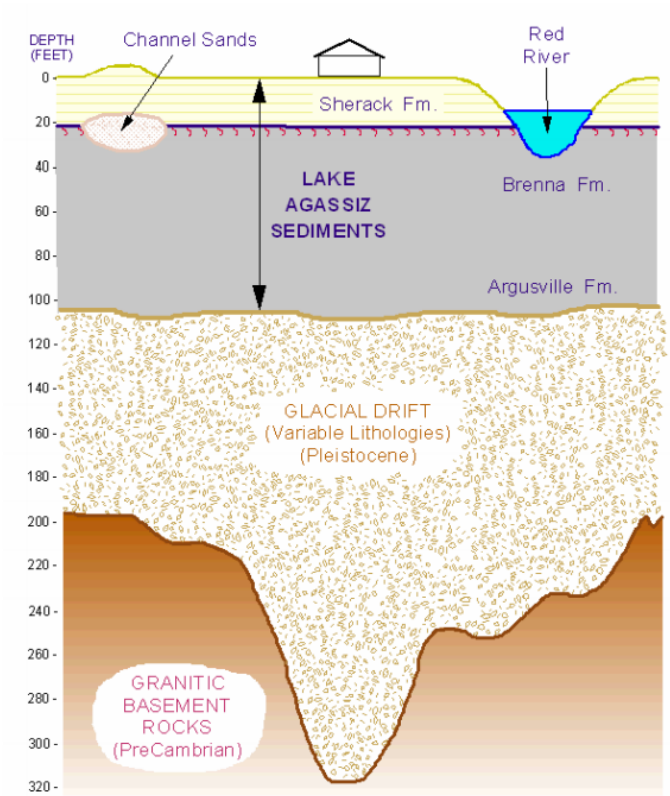

Figure 4: Cross section of soil layers in region

Table 1: Soil Layer Properties

Material	Unit Weight (pcf)	K (psf)	G (psf)	Drained C (psf)	Drained Φ'	Undrained C (psf)	Undrained Φ_{cu}
Alluvial Deposits	125	180,200	108,100	0	35	0	35
Huot Formation Clay	112	95,700	57,240	0	18.5	s_u/σ'_p = 0.22	
Red Lake Falls Formation	142	644,900	215,000	0	40	0	40
Failure Zone (Huot)	112	82,470	49,480	0	16	--	--

UTEXASED4 Process

In order to know the state of stability of the current slope, an analysis was performed to discover a factor of safety for various cross sections under critical conditions. The slope strength was then back calculated to determine the slope strength and the forces required to stabilize the slope. The program that was selected to perform this analysis is called UTEXASED4, developed at the University of Texas by Stephen G. Wright and his research assistants. UTEXASED4D is an educational version of the full program; as such it is limited in its functionality. All results obtained from the program will contain some degree of error as a result of the simplifications that were necessary to the cross-sectional geometry and soil properties. Analysis in this program in done in 2-D, which provides a conservative estimate for the factor of safety (Duncan 1992).

UTEXAS requires input of soil layer stratigraphy lines, layer properties (unit weight, cohesion, and friction angle), piezometric line, and distributed loads. These parameters were among the materials received from Nicholson Construction. Also included were various other materials that were utilized in their analysis. Three cross-sections of the project site were created based on the CPT borings that were performed in 2006, and are included in the appendix in Figure 12. The profile lines were extracted from these cross-sections, however the profile geometry had to be significantly simplified in order to input it into UTEXASED4 due to the educational version's restrictions. This gave each cross-section model a very linear and geometric grading, rather than the actual complex and irregular grading compared to the actual slope geometry. The points selected from each cross-section can be seen in Figures 5, 6, and 7. The original cross sections are provided in the appendix as Figures 13, 14, and 15.

To represent the weight of the highway with its accompanying traffic and the river with its accompanying hydrostatic forces, distributed loads were added to the model. Critical conditions were assumed to be during long term drought conditions, with high river drawdown levels that correspond to a dry riverbed, with the clay layers considered fully saturated. This assumption is consistent with historical conditions from the past failures (Schwert 2003). The loads along the highway were estimated for a conservative case in which traffic is stopped bumper to bumper adjacent to the slope as a 500 psf distributed load. The blue line in Figures 5-7 represents the water table elevation. The blue distributed loads on the left side of each figure represent the load applied by the vehicles. Cross sections A through C had original values for the factor of safety of 1.058, 1.052, and 1.157 respectively.

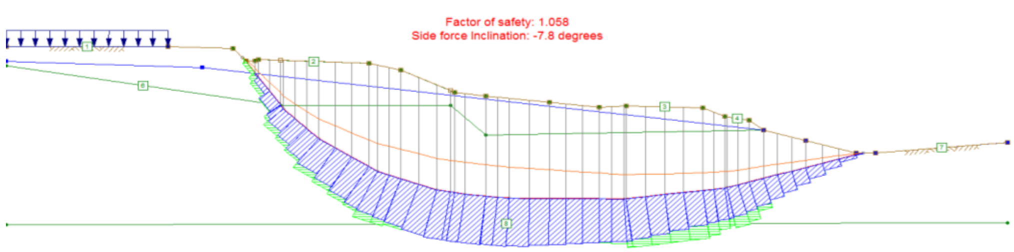

Figure 5: Modeled cross-section A

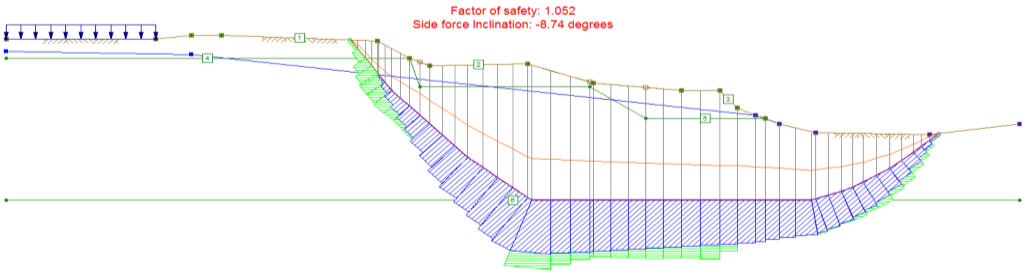

Figure 6: Modeled cross-section B

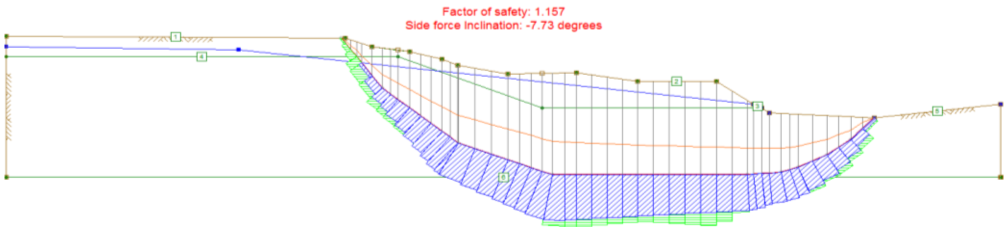

Figure 7: Modeled cross-section C

To obtain and enter the piezometric line, the CPT (cone penetration test) data from 2013 was analyzed. The point at which pore pressures were registered and assumed to propagate linearly through the soil was taken as the water table elevation. The referenced CPT was chosen as that which was located nearest to the vicinity of the cross-section being modeled. The x and y coordinates of that point of estimated water table height were estimated by comparing the mapping of the CPT boring to the mapping of the cross-section. The assumption was also made that the piezometric line (water table) varied linearly from the highest point of estimation until reaching the river, where the height of the river is the water table elevation.

Results

Design Solution

Obelisk Engineering identified a solution to the slope stability problem in Crookston as part of this design. The method of slope stabilization that was selected by the design team was a series of drilled shafts. These shafts are to be placed in a linear pattern across the problem area as outlined in Figure 8.

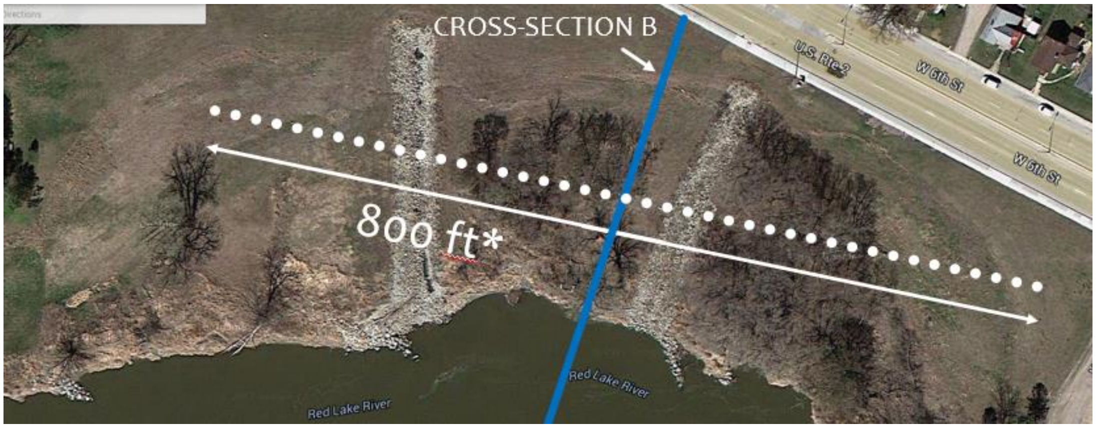

Figure 8: Location for drilled shafts

This type of system is useful to provide strength to the slope through the soil mechanics phenomenon of soil arching. Soil arching occurs when piles are introduced and the unyielding soil transfers some of its load to unyielding members, in this case the drilled shafts (Hosseinian 2013). A conceptual image of this phenomenon is presented in Figure 9. As can be observed from the figure, when shafts are arranged with an equal spacing between them, the soil transfers most of the lateral load of the soil onto the shafts. In the case presented in this report, this effect will improve the low shear strength of the native soil, and will provide sufficient strength to prevent future movement. This will also allow a suitable safety factor to be reached for the slope under the critical conditions.

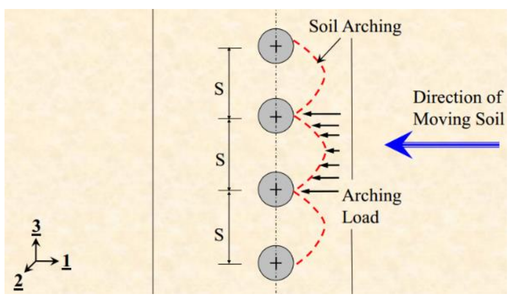

Figure 9: Soil arching conceptual description

L-Pile Design

In order to calculate the lateral reinforcement that would be required a separate computer analysis program called L-Pile was used. L pile is commonly used to model retaining walls or drilled shafts for foundations and can easily be utilized for this project. A very simplified cross section based on the parameters entered into UTEXAS was input into L-Pile, along with the appropriate soil properties as shown in Table 1. The alluvial deposits, Huot clay, and glacial till were modeled with the sand (Reese), soft clay, and API at depths of 10, 60, and 100 feet respectively. A drilled shaft was inserted into the model with the parameters defined as in Figure 11. Lateral displacements were entered in the soil layers to determine the resisting forces that would be exerted on the soil from the shaft. By entering the lateral forces as reinforcement lines in UTEXAS, the factor of safety for any given pile parameters can be determined. Multiple iterations of the design were performed before final dimensions were determined. Various plots for the forces and displacement of the shaft are provided in Figure 10.

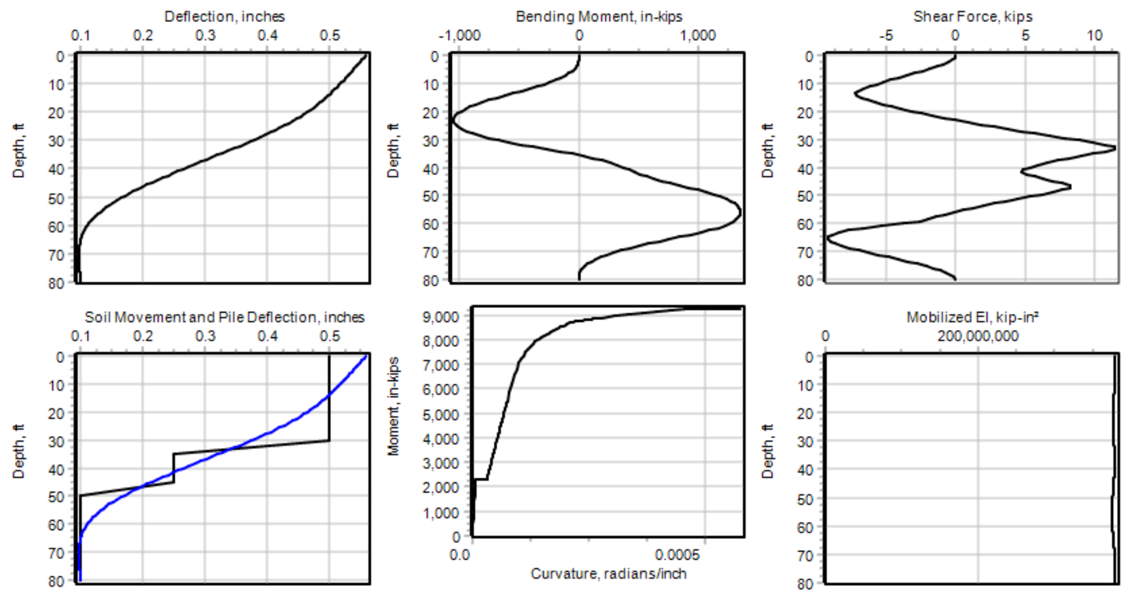

Figure 10: Deflection, bending moment and shear force diagrams from L-Pile output

From theses plots it can be determined that the max pile deflections that can be expected is less than 1 inch at the top of the shafts. Final factors of safety for cross-sections A, B, and C were 1.471, 1.326, and 1.467 respectively. These values are above the minimum value of 1.3 that was decided for this design. Cross-sections for these final safety factors can be observed in Figures 16, 17, and 18 in the appendix.

Constructability

To accomplish the soil arching phenomenon before described, it is necessary to introduce structural elements into the soil. Commonly known as "drilled shafts," these structural elements will consist of reinforced concrete columns placed towards the base of the slope failure, and will pass the limits of the failure area, as presented in Figure 8. Heavy equipment will be used to drill shafts 3 feet in diameter. Shafts will be spaced at 9 feet center to center for a distance of just over 800 feet, resulting in a total of 89 shafts. Required depths interpolated from the cross sections vary slightly, however a depth of 75 feet provides sufficient depth regardless of the location along the slope. This depth allows a minimum of 10 feet of the shaft to be embedded into the glacial till layer (Sherrack formation).

After the shafts are drilled, they will be filled with a slurry composed of material removed from the shafts and water. An analysis will be performed to ensure that these shafts will be able to maintain a stable slope while they are filled with slurry. The rebar cages will be constructed offsite and inserted after being brought to the location. Concrete will be pumped down into the shaft. Since the concrete will be denser than the slurry, the slurry will be displaced as the shaft fills with concrete. This method limits the amount of slope instability that will be introduced as part of the construction. Temporary measure to increase the factor of safety during construction may be necessary, however requirements for temporary methods of reinforcement will not be provided in this report as it is largely a function of equipment weight and weather conditions at the site.

Normal weight concrete is assumed for the shafts. Steel rebar is to be spaced symmetrically at a distance of 9.72 inches in the column as single bars, with a minimum edge cover of 3 inches to protect against corrosion of the steel, and 8 #10 bars will be used. This provides a 1.00% percentage of steel to area of the column. Further information is provided in Figure 11.

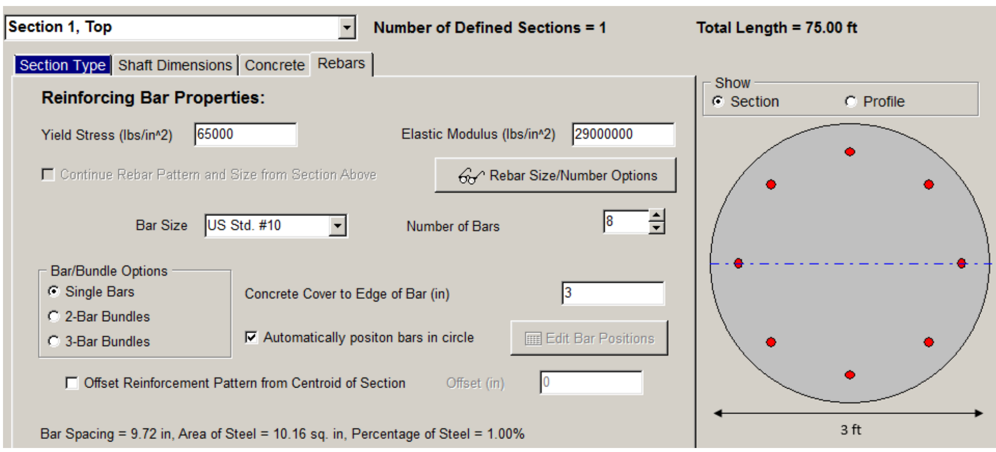

Figure 11: Concrete shaft parameters and cross-section

Cost

Projects with similar conditions typically are completed with a budget in the range of 5 million to 8 million USD. A prediction for the cost of materials such as the concrete and steel required at on typical current market value (2015) is $550,000 based on the shaft dimensions. Calculations for this value can be seen in the appendix in Figure 19. The two other large components that were considered were labor and equipment. Labor was estimated based on a projected construction time of 6 months, giving a value of $1,750,000. Equipment costs would include the drilling rig as well as the price to transport needed tools and materials to the site. This was estimated as $1,000,000. This gives a total estimated cost for the project as $3,300,000.

Alternatives

Slope-Grading

Due to the recent slope failures, the current state of the project site is very rough and scarped. This terrain has made future failures more likely in the areas of greatest experienced slope-change and instability. After completion of the drilled shaft installation, the slope will be graded in steps. This case has already been analyzed and considered in the UTEXASED4 program, and it was found that the grading of the slope in a stepped manner increases the factor of safety. Exact dimensions or the grading may be altered as need demands, though it is suggested to use a 1V:6H slope with a step width of around 50 feet. This slope grading would also allow the area to be landscaped, providing an aesthetic quality to the final build along the bank of the Red River.

Driven Piles

Driven piles were considered due to their close similarity to drilled shafts. Based on the required diameter required to prevent flow around failure for the Huot clay formation and the sand wedge failure criteria at the top of the reinforcement, driven piles were no longer considered as a feasible design option. The minimum diameter that could be utilized while avoiding both of these failure limits was 1.7 feet. Driven piles also generate large amounts of vibration as they are installed and MnDot placed strict vibration limits on the design to limit damage to surrounding structures. Due to the lack of programs available to asses these values and assuming the vibration could also destabilize the slope, this option was dropped. Driven piles were originally considered due to the cost and construction time that would have been saved.

Retaining Structure

Another possibility that was originally considered was a retaining structure designed to prevent mass movement merely by its weight. Such a structure would need to be placed at the foot of the expected slope failure plane, preventing further displacement of the toe of the failure. The weight would essentially serve the same function as that of higher river levels as described for the critical conditions for the slope. This method was deemed unusable as the failure plane extends into the river way. Placing a structure in the center of the river would change erosion patterns along the banks considerably, which is not within the MnDot desired outcomes, as well as potentially worsen future conditions by changing the location of the failure plane.

Conclusion

Obelisk Engineering thus recommends that the solution that should be used to prevent future slope movement is a series of drilled shafts. These drilled shafts will induce soil arching loads, and prevent the weak clay layer from displacing. Adherence to MnDot restrictions is maintained in the presented aspects of the design. With this solution, future damage to Trunk Highway 2 will be prevented.

Appendix

Extra Figures

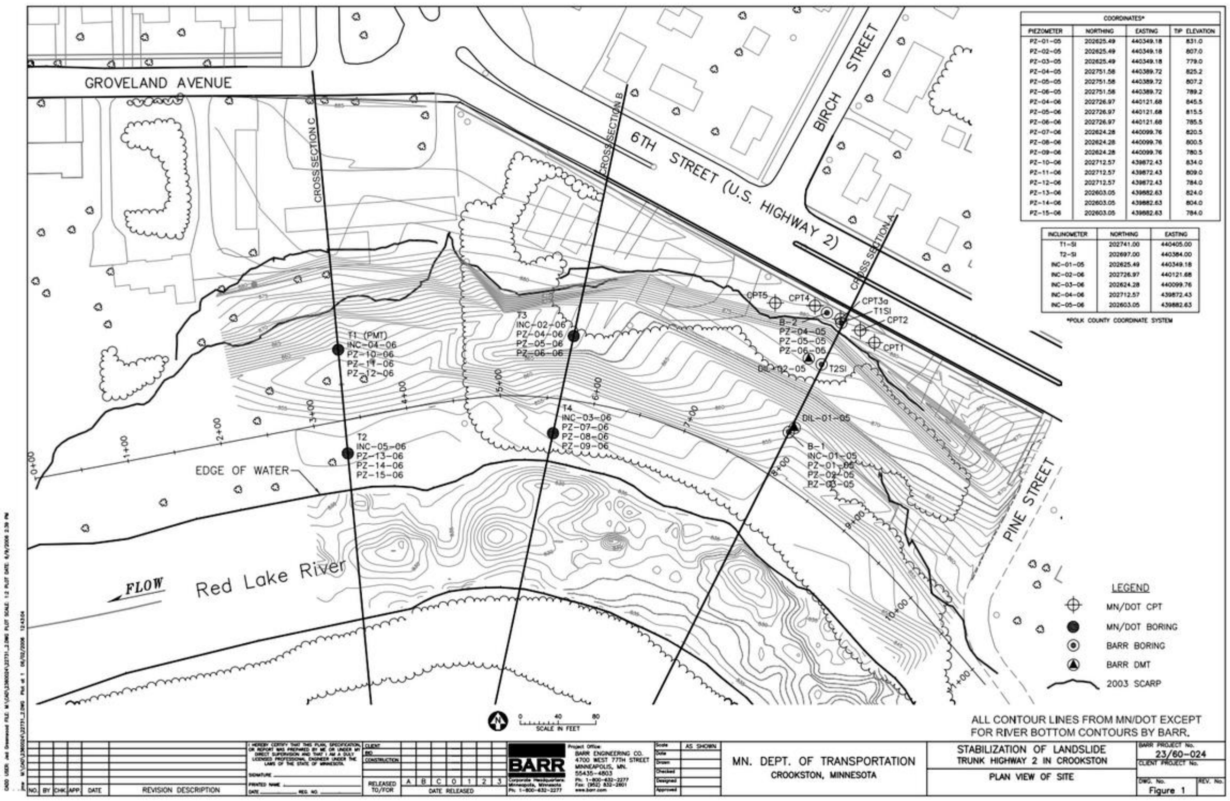

Figure 12: Cross section location and alignment

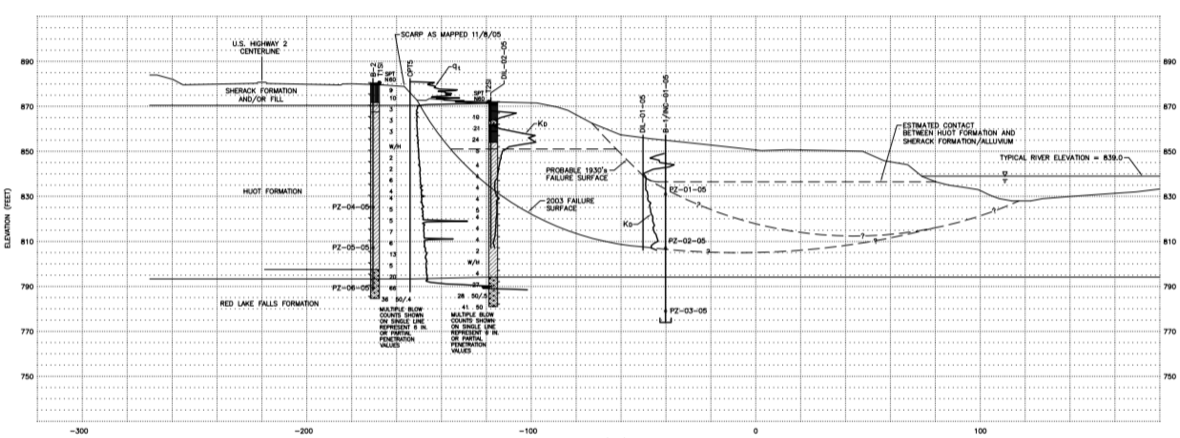

Figure 13: Original cross section A

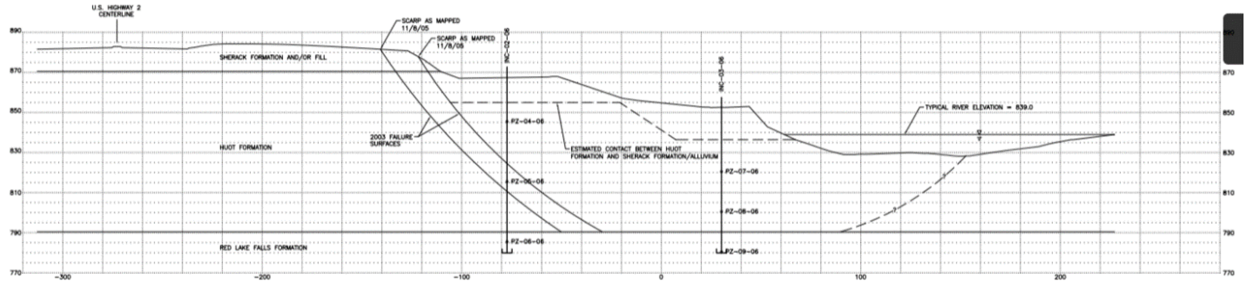

Figure 14: Original cross section B

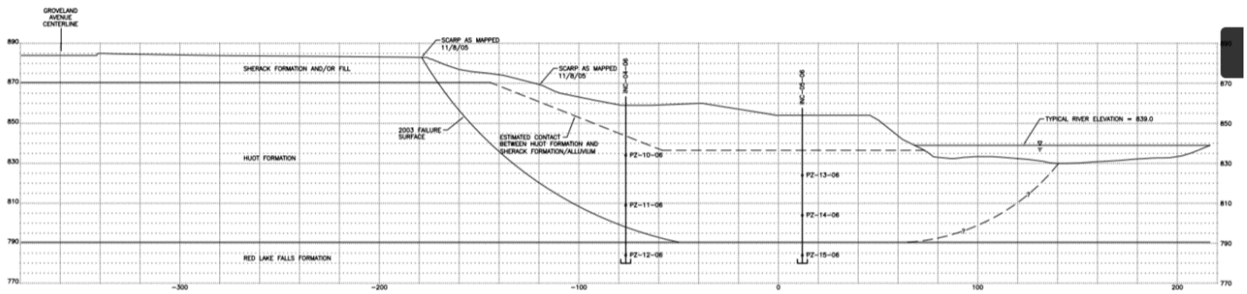

Figure 15: Original cross section C

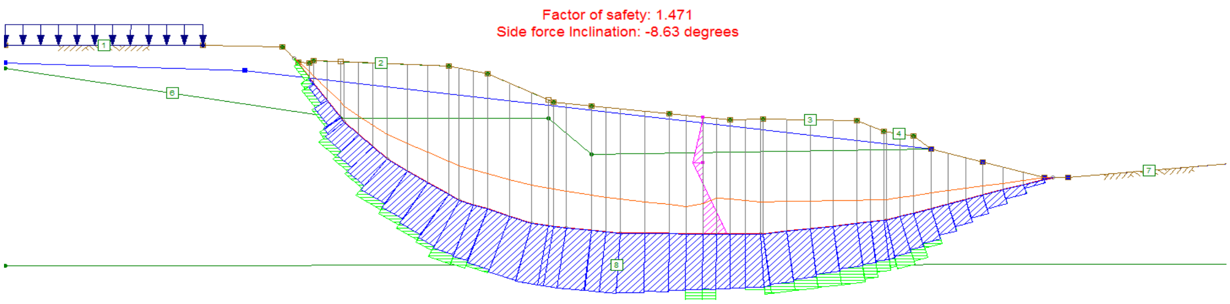

Figure 16: Final cross-section A

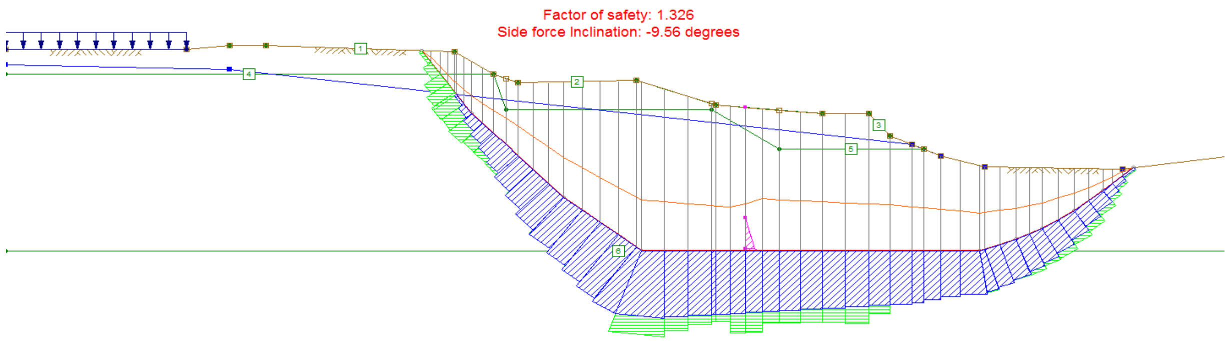

Figure 17: Final cross-section B

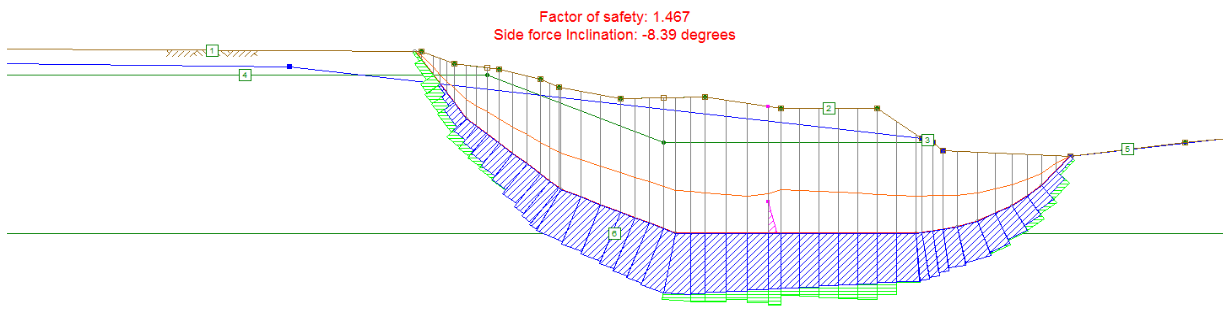

Figure 18: Final cross-section C

Concrete			Steel-Rebar			Steel-Ties		
# Shafts	89		# bars	10		Tie Spacing	12	in
Diameter	36	in	Bar #	10		Bar #	3	
Depth	75	ft	Weight	4.3	lb/ft	Weight	0.376	lb/ft
Cubic Yards Per Column	20	yd^3				Edge cover	3	
Cost	$ 90.00	/yd^3	Cost	$ 0.70	/lb	Cost	$ 0.70	/lb
Concrete Cost	$ 157,275.98		Rebar Cost	$200,917.50		Ties Cost	$ 182,138.17	
Materials Total	**$540,331.65**							

Figure 19: Example cost calculations

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