US Army Corps
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St. Paul District

Wild Rice Hydraulic Structure and I-29/CH16 Interchange Micrositing
Appendix F

Fargo Moorhead Metropolitan Area
Flood Risk Management Project

Supplemental Environmental
Assessment Document

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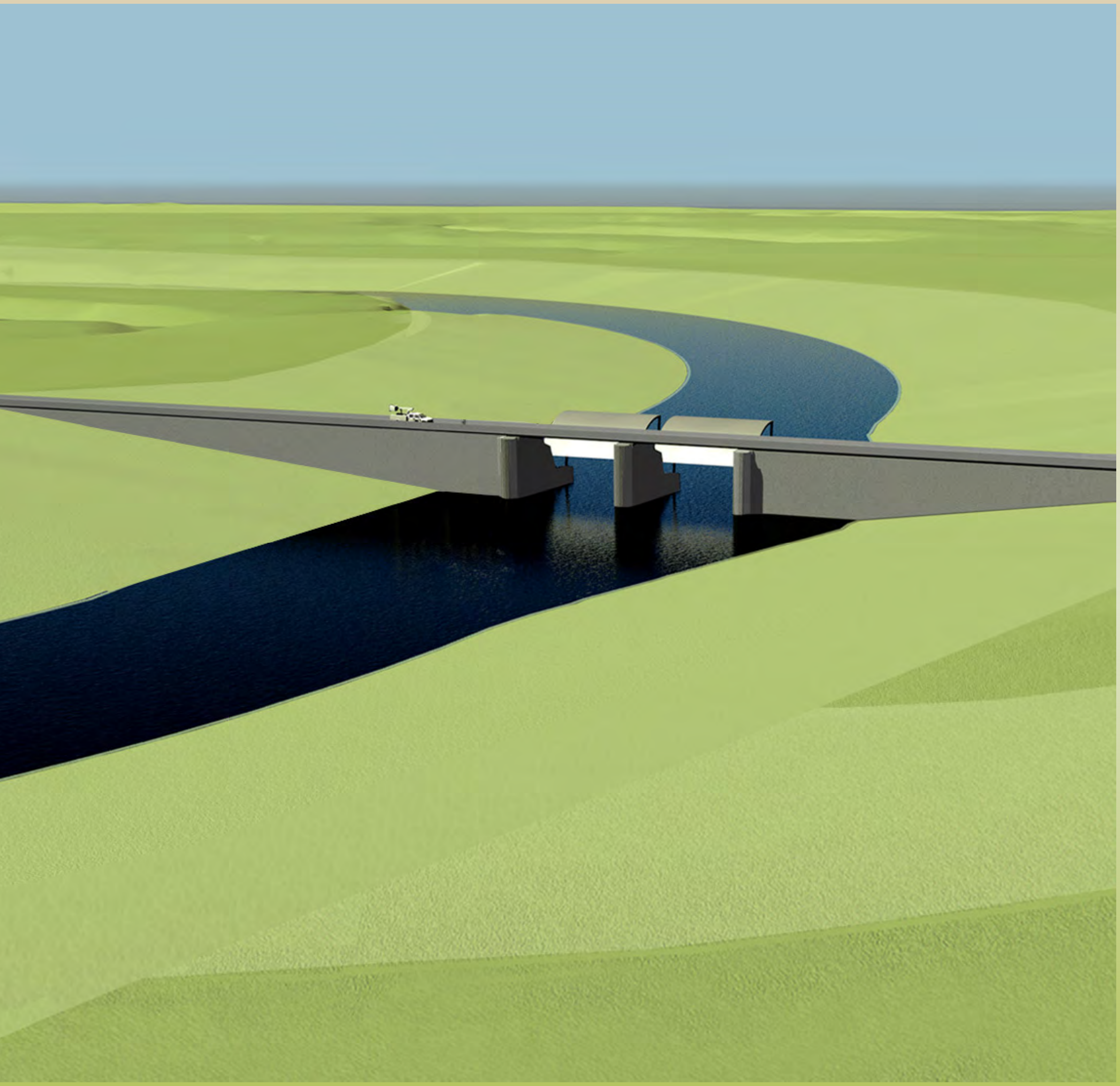
FINAL TECHNICAL MEMORANDUM

FM DIVERSION POST-FEASIBILITY ALTERNATIVES ASSESSMENT: WILD RICE HYDRAULIC STRUCTURE AND I-29/CH16 INTERCHANGE MICROSITING

June 18, 2014 (July 29, 2014 revision)



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EXECUTIVE SUMMARY

This study evaluates and compares micrositing layout alternatives for the Wild Rice Hydraulic Structure and I-29/CH-16 Interchange. These are referred to as the WRR micrositing alternatives. The analysis includes micrositing for the area extending from 1 mile west of the I-29/CH-16 interchange to 1 mile east of the interchange, including the Wild Rice Hydraulic Structure, realigned Wild Rice River, dam/embankment, and a portion of the staging area.

The alternatives to the VE13A siting that was presented in the October 10, 2012 “Final Technical Memorandum – FM Diversion Post-Feasibility Southern Alignment Analysis” (PFSAA) and included in the September, 2013 “Supplemental Environmental Assessment – Design Modifications to the Fargo-Moorhead Metropolitan Area Flood Risk Management Project” (Supplemental EA), have the potential to increase project value by further optimizing the project functionality relative to the estimated cost and impacts, by reducing cost or by minimizing risks.

An initial screening-level assessment of a base case and six (6) layout alternatives was developed in August, 2013. From these options, the three (3) most preferable layouts were targeted for the micrositing evaluation presented in this report.

The perceived value of further investigating each of the three micrositing alternative layouts is described below:

1. **Alternative 0**, shown in Figure 1 was conceptually identified during 2012 post-feasibility value engineering work as the VE13A alignment. The base case for this analysis assumes the I-29/CH-16 interchange is relocated northward to avoid construction of interchange ramps over the dam/embankment. As a surrogate for VE13A, this is the base case for comparison purposes. Alternative 0 differs slightly from the Recommended Alternative in that the connecting channel between the Diversion inlet and the Wild Rice River (WRR) connects on the west side of Interstate 29, which eliminates the connecting channel and bridges through Interstate 29. This results in conveyance to the west towards the Diversion inlet passing through the WRR bridge. Alternative 0 also eliminates the connecting channel reach between the WRR and Red River of the North (RRN). The channel included in this area is sized to handle local drainage.
2. **Alternative 3**, shown in Figure 2 was conceptually identified in 2013. The alternative possibly adds value by adding residential structures to the flood risk reduction area and eliminating relocation of the I-29/CH-16 interchange. This alternative locates the Wild Rice Hydraulic Structure on the east side of I-29.
3. **Alternative 5**, shown in Figure 3 was conceptually identified in 2013. The alternative possibly adds value by adding residential structures to the flood risk reduction area, eliminating relocation of the I-29/CH-16 interchange. This alternative locates the Wild Rice Hydraulic Structure on the west side of I-29.

The evaluation is partly based on a set of assessment criteria developed during an August 22, 2013 HMG design meeting and modified during the August 22, 2013 Local Sponsor Local Consultant Technical Team (LSLCTT) meeting with participation of U.S. Army Corps of Engineers (USACE). The assessment criteria include:

1. Construction costs
2. Risk reduction considerations (length of dam)
3. Property impacts (number of residential structures)
4. Environmental considerations (project extents and 404B OHWM Permit impacts)
5. Geotechnical considerations

6. Constructability
7. Transportation safety considerations

Estimated costs for construction features as well as for lands and easements for each alternative were developed for consideration in conjunction with the characterization of the other assessment criteria listed above. The design considerations used for comparison of the WRR micrositing alternatives are generally consistent with methodology and assumptions presented in the Phase 4 Federally Recommended Plan (FRP) April 19, 2011 A/E deliverable to the USACE and the October 10, 2012 PFSAA.

Comparing the assessment factors with the cost can assist in deciding which WRR micrositing alternative best meets project objectives and provides the greatest value. Alternatives 3 and 5 were identified as technically favorable alternatives during the December 5, 2013 LSLCTT meeting, and based on this, a draft Technical Memorandum was developed on December 20, 2013. Additional geotechnical investigations were performed during winter 2014 to gather site-specific data for the Alternative 5 Wild Rice Control Structure location. This report update incorporates technical review comments provided by the USACE and local project sponsors on the December 5, 2013 draft report along with results from the subsequent geotechnical investigations.

The following table summarizes the alternatives comparison. Additional details are presented in the main text that may be of use in selecting an alternative and informing detailed design efforts.

Table 0.1: Assessment Factor Summary and Cost Comparison for WRR Micrositing Alternatives

Assessment Factor	Unit	Alternative 0	Alternative 3	Alternative 5
Estimated construction cost	$	$131 million	$120 million	$118 million
Risk reduction considerations; length of dam from CR17 to Red River	LF	30,300	33,152	34,400
Property impacts (number of residential structures)	#	Base Case	4 fewer impacted (4 transferred to risk reduction area)	4 fewer impacted (4 transferred to risk reduction area)
Environmental considerations (constructed project features and staging area extents compared to Alternative 0) [1]	AC	Base Case	251 acres less	419 acres less
Environmental considerations, 404B Permit impacts (length of Wild Rice River impact)	LF	3,200	5,500	3,300
Geotechnical considerations	N/A	See text	See text	See text
Hydraulic considerations	N/A	WRR and RRN Structures both East of I-29	WRR and RRN Structures both East of I-29	WRR Structure West of I-29, RRN Structure East of I-29, increases resiliency of the system
Constructability	N/A	Challenges associated with complete interchange relocation	Favorable	Favorable, makes existing I29 bridge over Wild Rice River independent of the project. Challenges associated with proximity of WRR structure to I29.
Transportation safety considerations (I-29 bridges) [2]	No.	2	2 or 4	4
Transportation safety considerations (length of I-29 grade raise)	Miles	4.48	4.03	3.79

[1] Change to project footprint and staging area within an assumed micrositing extent in vicinity of the WRR Control Structure.

[2] Future efforts could investigate if two bridges for Alt. 5 over the Wild Rice River north of the dam could instead be culverts

1 BACKGROUND AND OVERVIEW

The Fargo-Moorhead Area Diversion (FM Diversion) Project was developed by the U.S. Army Corps of Engineers (USACE) to provide flood risk reduction for the Fargo-Moorhead area and is presented in the Integrated Final Feasibility Report and the Final Environmental Impact Statement (FR/FEIS) dated July 2011. Readers unfamiliar with the project should reference these documents for additional detail about the project. The project consists of a 20,000 cfs diversion channel with upstream staging and storage, and was referred to as the Locally Preferred Plan (LPP, aka North Dakota Diversion) in the FR/FEIS. This plan was also known as the Federally Recommended Plan (FRP).

Following the Record of Decision on the FR/FEIS, a number of studies were initiated to look at ways to improve the project. This included a review of the southern alignment for the project based on the results of a Value Engineering (VE) Study. Several southern alignment alternatives to the FRP were studied and presented in the October 10, 2012 “Final Technical Memorandum – FM Diversion Post-Feasibility Southern Alignment Analysis” (PFSAA). Ultimately, this study resulted in the selection of the VE13A alternative for the southern alignment. This change to the alignment from the FR/FEIS, was included in the September, 2013 “Supplemental Environmental Assessment – Design Modifications to the Fargo-Moorhead Metropolitan Area Flood Risk Management Project” (Supplemental EA). The design modifications outlined in the Supplemental EA are also referred to as the Recommended Alternative.

The VE13A alignment differs from the FRP only east of CH17. Unlike the FRP, which follows a line southeast of CH17 to the Wild Rice River, the VE13A alignment follows a line parallel to and approximately 1/8 mile south of CH16. VE13A includes a tie-back embankment which ties off to existing high ground directly east of the Red River of the North (RRN) Control Structure in Minnesota. Micrositing of the Wild Rice Hydraulic Structure and the I-29/CH16 interchange was not performed as part of the VE study.

This report was developed to compare micrositing alternatives for the I-29/CH16 Interchange and Wild Rice Hydraulic Structure. Following an initial screening of seven (7) micrositing options, three preferable options were targeted for further investigation. The three micrositing alternatives evaluated in this study are Alternative 0, Alternative 3 and Alternative 5. Alternative 0 should be considered the baseline scenario that is an updated surrogate for VE13A.

1.1 PURPOSE OF STUDY

The purpose of this study is to evaluate and compare WRR micrositing alternatives that have the potential to increase project value. Value is defined by USACE in Report No. CEMVP-VE-FY12-02_FMM Outlet, *Value Based Design Charrette, Outlet Structure & Diversion Reach 1,* dated December 2011 as:

> “…the relationship between functions and resources where function is measured by the performance requirements of the customer and resources are measured in materials, labor, price, time, etc. required to accomplish that function. Therefore, this process focuses on creating a best value solution by identifying the most resource efficient way to reliably accomplish the functions that meet the

performance expectations for the project".

1.2 SCOPE AND INITIAL SCREENING OF ALTERNATIVES

Seven (7) conceptual WRR micrositing alternatives were developed at an August 22, 2013 HMG design meeting. The seven (7) concepts (termed base case Alternative 0 and Alternatives 1 through 6) were presented at the August 22, 2013 Local Sponsor Local Consultant Technical Team (LSLCTT) meeting for comments. The USACE and Local Project Sponsor Representatives provided feedback that eliminated four of the six alternatives (Alternatives 1, 2, 4 and 6) and recommended investigating three further (Alternatives 0, 3 and 5). For initial descriptions of the base case and six alternatives as well as the design criteria and constraints used to screen them, see Appendix A.

1.3 ALTERNATIVES

This study includes one base case (Alternative 0) and two WRR micrositing alternatives (Alternative 3 and Alternative 5).

The WRR micrositing alternatives as defined at this time are shown in Figures 1, 2 and 3. Alignment details may be revised during final design. The alternatives are identical outside of the study extent (1 mile on either side of the I-29/CH-16 interchange). The design considerations, cost estimating methodology, and staging impact mitigation methodology is the same for all alternatives, allowing for some objective comparison, although some criteria are subjective.

1.3.1 ALTERNATIVE 0

Alternative 0, shown in Figure 1, was conceptually identified during 2012 post-feasibility value engineering work as the VE13A alignment. It has an interchange refinement. It has an interchange refinement applied to the Recommended Alternatives/VE13A alignment as presented in the Supplemental EA. The base case for this analysis assumes the I-29/CR-16 interchange is relocated northward to avoid construction of interchange ramps over the dam/embankment. This alternative requires complete reconstruction of the I-29/CR-16 interchange north of its existing location. This is the base case that is an updated surrogate for VE13A for comparison purposes. Alternative 0 differs slightly from the Recommended Alternative in that the connecting channel between the Diversion inlet and the Wild Rice River (WRR) connects on the west side of Interstate 29, which eliminates the connecting channel and bridges through Interstate 29. This results in conveyance to the west towards the Diversion inlet passing through the WRR bridge. Alternative 0 also eliminates the connecting channel reach between the WWR and Red River of the North (RRN). The channel included in this area is sized to handle local drainage.

1.3.2 ALTERNATIVE 3

Alternative 3, shown in Figure 2, was conceptually identified in 2013. The alternative possibly adds value and reduces cost by adding residential structures to the flood risk reduction area and eliminating relocation of the I-29/CH-16 interchange. This alternative locates the Wild Rice Hydraulic Structure on the east side of I-29.

1.3.3 ALTERNATIVE 5

Alternative 5, shown in Figure 3, was conceptually identified in 2013. The alternative possibly adds value and reduces cost by adding residential structures to the flood risk reduction area, eliminating relocation of the I-29/CH-16 interchange. This alternative locates the Wild Rice Hydraulic Structure on the west side of I-29.

2 MICROSITING METHODOLOGY AND CONSIDERATIONS

The design considerations used for the WRR micrositing alternatives are the same as those used in the Phase 4 FRP April 19, 2011 A/E deliverable to the USACE, except as noted in this report. Feasibility-level micrositing was performed for alternative alignments and hydraulic structure locations as discussed in this section. Assumptions behind the design considerations are intended to allow for consistent comparison to the Recommended Alternative as Presented in the Supplemental EA. The purpose of feasibility design was to generate alternative specific costs and assess differences in the alternatives as compared to the Recommended Alternative as Presented in the Supplemental EA.

2.1 GEOTECHNICAL CONSIDERATIONS

A stability analysis was performed to evaluate geotechnical issues and provide a basic framework for the alternative layouts and grading evaluated for the location of the Wild Rice River Control Structure. A settlement analysis was performed to provide insights into the favorability of alternatives in terms of I-29 grade raise and construction costs. See Appendix B for a more in-depth explanation of the geotechnical parameters used for this study.

Preliminary geotechnical stability analysis was performed in 2013 which relied on soil strength parameters based on regional soil data. The analysis was updated in 2014 to include stratigraphy and laboratory results from one soil boring performed on the site near where the Alternative 5 dam crosses I-29. The findings of this field investigation are incorporated into this report. The general approach to the geotechnical assessment is listed below:

- **Stability Analysis 1**: stability analysis of the north abutment of the I-29 bridge over the Wild Rice River (including I-29 grade raise);
- **Stability Analysis 2**: for alternative 5 layout, stability analysis of the existing Wild Rice River banks;
- **Stability Analysis 2**: for alternative 5 layout, stability analysis of where the right bank of the Wild Rice River approaches the proposed I-29 grade raise;
- **Stability Analysis 3**: slopes at the wingwalls of the control structure;
- **Preliminary Settlement Analysis**: analysis provides information to assist in planning settlement mitigation measures for the I-29 grade raise, investigating construction sequencing and constructability;
- **Preliminary Wick Drain Design**: this analysis provides guidelines for conceptual wick drain spacing and a basis for comparing settlement mitigation costs associated with the three micrositing alternatives;

When the project moves into detailed design, additional site specific information should be obtained. This analysis does not take into account the effect of previous failures that have occurred along the Wild Rice River and does not account for any changes to the current river water surface profiles. A detailed field reconnaissance has not been performed, but is recommended to evaluate any existing scarps and perform a back-analysis to determine the effects of previous failures that have occurred in the area.

2.2 HYDRAULIC CONSIDERATIONS

Alternative 3 is consistent with Alternative 0 in that both the Red River Control Structure and and Wild Rice Control Structure are east of I-29. Thus, it depends on functionality of a flow linkage across the I-29 embankment to convey flood flows from the staging area pool west of the I-29 embankment eastward beneath I-29 toward the control structures. The same is true for local drainage.

Alternative 5 places the Wild Rice Control Structure west of I-29, while the Red River Control Structure is east of I-29. This places one control structure on the portion of the staging area pool east of the I-29 roadway embankment, and one on the west side. This may present advantages for operation of the structures and reduce dependence on the flood flow linkage across the I-29 embankment. Local drainage would also follow this same flow pattern.

Hydraulic estimates are used to estimate the top-of-structure elevations for control structures. During the SDEIS the top of structures on the Red River of the North and Wild Rice River were based on the 0.2-percent annual chance event (water surface elevation of 922.1) plus 5-feet of freeboard (927.1). Following modifications for the PFSAA VE13A alternative, which included modifications to the staging area and locations for the hydraulic structures on the Red River of the North and Wild Rice River, the top of structure was determined by the 103,000 cfs event peak water surface elevation (924.8) plus 4-feet of freeboard or the Probable Maximum Flood (PMF) peak water surface elevation (925.5) plus 3 feet of freeboard. Based on this criteria, a top of embankment and structure elevation of 928.8 was used previously. Due to ongoing updates to the PMF, the top of embankment and structure elevations for this study were based on the PMF peak water surface elevation plus 5 feet (930).

For additional hydraulic considerations factored into this analysis see Appendix D.

2.3 STRUCTURAL DESIGN CONSIDERATIONS

Structural design was not revisited as part of this micrositing comparative analysis. Revisions to the top-of-structure elevations were included as pro-rated cost increases in the cost estimates, similar to PFSAA cost methodology.

A top-of-embankment elevation of 930 feet (Probable Maximum Flood (PMF) plus five (5) feet) is assumed for all alternatives. Due to ongoing updates to the PMF, the top of embankment and structure elevations for this study were based on the PMF peak water surface elevation plus 5 feet (930).

2.4 CIVIL DESIGN CONSIDERATIONS

In general, the hydraulic structures remained approximately the same order-of-magnitude size as their comparable structure in the FRP. Major design considerations generally follow FRP methodology, including:

- Control structures are assumed to be constructed in dry conditions (off the existing river channel).
- A 300 foot minimum buffer between the proposed control structure and the existing river channel is assumed.

- Excavation slope grading for realigned river channel was assumed to be 7H:1V.
- Access roadways, maintenance buildings, and SCADA are included in cost estimates.
- Stratigraphy and elevations of the soil layers for determining earthwork quantities for the hydraulic structures was not available for the alternative sites and is assumed to be 50% Type 1 and 50% Type 2.
- Deed restricted properties were considered and avoided when developing layouts for the hydraulic structures. No deed restricted properties are located within the site extent of this analysis.
- A permanent easement of 30 feet, offset from the extents of the grading work, is assumed for the site work at each hydraulic structure. Greater than 30 feet was used for some areas to simplify property line angles.
- A temporary easement of 15 feet, offset from the extent of the permanent easement, is assumed for the site work at each hydraulic structure. Greater than 15 feet was used for some areas to simplify property line angles.
- A separate fish passage structure/system is not assumed at the Wild Rice Hydraulic Structure.

The civil layouts presented are intended to provide a basis for identifying parcels on which to perform additional investigations going forward. The civil layouts are preliminary. Channel realignment degrees of curvature, angles and detailed grading will need to be investigated in greater detail during detailed design. Consideration should be given to how channel geometries influence flow patterns and contribute to areas of bank erosion and channel scour. Alignment of the dam/embankment will need to be revisited during detailed design to optimize dam length and right-of-way (ROW) acquisitions.

2.5 CONSTRUCTION COST CONSIDERATIONS

The estimated costs presented in this report are intended to be used for evaluating if the alternative alignments are cost competitive with the baseline Alternative 0.

Relocating the WRR hydraulic structure for alternatives 3 and 5 results in different estimated river invert elevation and a slightly shorter structure. The cost of the structure was prorated to account for this slight change. Prorated cost of $500,000 per vertical-linear-foot was used (based on feasibility’s Phase 3 and Phase 4 control structure comparative costs).

Costs presented do not include escalation or operations and maintenance costs.

It is worth noting that costs associated using excavation slopes flatter than 7H:1V and any costs associated with ground improvements for geotechnical purposes (such as to steepen slopes) are not included at this time, but should be revisited once site-specific field investigation and further geotechnical analyses are performed. Assumed excavation and embankment slopes generally affect all three alternatives globally and allow for a reasonable comparison.

The feasibility level construction cost estimate provided in this report is made on the basis of HMG’s experience and qualifications and represents our best judgment as experienced and qualified professionals familiar with the project. This opinion is based on project-related information available to HMG at this time, current information about probable future costs and a feasibility level design of the project.

2.6 LAND ACQUISITION

Lands and damages cost methodology are consistent with assumptions used by USACE in the FR/FEIS. The lands and damages are separated into two categories: lands needed for the project features and lands impacted by staged water. A more detailed description of the methodology used for estimating costs follows.

The lands and damages cost for the areas required for the project features include a combination of permanent and temporary easements. Permanent easements are assumed for areas directly within the footprint of the project features, and an additional 30 feet beyond the footprint. In addition to permanent easements, the cost estimate assumes a temporary easement 15 feet wide outside of the permanent easement. The permanent easement cost assumptions include an average cost per acre for all acreage. In addition to the acreage cost, average values are used for all residential and non-residential structures. Similar to the permanent easements, an average cost per acre is used for temporary easements. The cost estimate also assumes a 5% administrative cost and a 25% contingency cost in addition to relocation costs as outlined in the Draft Real Estate Cost Estimate competed under Task Order #6.

Cost assumptions for lands impacted by staged water are based on the type of property, as well as the depth of inundation in the impacted area. The impacted area has been defined by USACE as any inundated area in which the increased water surface elevation during a 1-Percent Chance Event is at least one foot higher than under existing conditions. The land categories, similar to the lands required for project features, are split into three categories: acreage, residential structures, and non-residential structures. Within the impacted area, properties are divided based on the total depth of inundation during a 1-Percent Chance Event. The depth categories used include those areas and structures with three feet or more of total inundation and those areas and structures with less than 3 feet of total inundation.

The cost estimate assumes that all acreage within the impacted area on which 3 feet or more of total inundation occurs during the 1-Percent Chance Event would be purchased at the same average per acre rate as stated in the Draft Real Estate Cost Estimate completed under Task Order #6. The cost of flowage easements on land with less than 3 feet of total inundation is 25% of the value used for areas inundated with 3 feet or more. The acreages presented in Table 3.1 assume the staging area extent is unchanged. A higher 1% event staging elevation for Alternatives 3 and 5 may result in adjustment of the staging area extent. Based on hydraulic modeling performed to date, the difference in staging elevations between the alternatives is small (0.2’ or less). Therefore, the staging elevation will not significantly differentiate the alternatives and the difference in staging elevation was not taken into account in calculating lands and easement acreages.

Similar to structures purchased within the project area, average values are used for all residential and non-residential structures that are inundated by 3 feet or more during the 1-Percent Chance Event. Separate, smaller average values are used for structures inundated by less than 3 feet of water. The cost estimate assumes an average administrative cost per parcel and another per structure.

Table 3.1 Lands and Easements: Acreage Comparison Summary

Cost Category	Alternative 0	Alternative 3	Alternative 5
Right-of-Way (acres)	546	529	533
Easement (acres)	286	183	113
Staging Area (acres)	1,565	1,277	1,081
Total(acres)	2,397	1,989	1,727

3 ASSESSMENT FACTORS

The methodology and characterization of each alternative in terms of assessment factors are discussed below.

3.1 OPINION OF PROBABLE CONSTRUCTION COST

A summary of estimated land acquisition costs for each alternative is summarized in Table 3.2.

Table 3.2 Lands and Easements: Opinion of Cost Comparison Summary

Cost Category	Alternative 0	Alternative 3	Alternative 5
Right-of-Way ($)	3,276,000	3,174,000	3,198,000
Easement ($)	257,400	164,700	101,700
Staging Area ($)	7,042,500	5,746,500	4,864,500
Relocation Cost ($)	210,000	195,000	110,000
Parcel Cost Subtotal ($)	10,785,900	9,280,200	8,274,200
Admin. Legal (5%) ($)	539,295	464,010	413,710
Contingency (25%) ($)	2,831,299	2,436,053	2,171,978
Total Land Cost ($)	14,156,494	12,180,263	10,859,888

Note: Cost values based on Diversion Authority Land Purchase along I-29 in November, 2013 and values presented in the Draft Real Estate Cost Estimate completed under Task Order 6.

A comparison of estimated construction costs for alternative alignments for the WRR micrositing alternatives is presented in Table 3.3. The costs are for features within the assumed analysis extent. Some project-wide costs (such as mitigation, utility relocations, etc.) were not included as part of this effort and are not presented in Table 3.3, but are not anticipated to differentiate the alternatives.

Table 3.3 Opinion of Cost Comparison Summary

Cost Category	Alternative 0 ($)	Alternative 3 ($)	Alternative 5 ($)
Lands and Easements	14.2 million	12.2 million	10.9 million
Construction Cost: Transportation Features	43.5 million	32.5 million	29.4 million
Construction Cost: Hydraulic Structure and Sitework	52.5 million	55.6 million	58.3 million
Planning, Engineering, Design (15%)	14.4 million	13.2 million	13.2 million
Construction Management (7%)	6.7 million	6.2 million	6.1 million
Construction Cost Subtotal for Comparison (1) (2)	**131 million**	**120 million**	**118 million**

(1) 26% Contingency included in the opinion of construction cost, PED and CM. 25% contingency included in the Lands and Easements cost based on Task Order #6 Draft Real Estate Cost Estimate.

(2) Categorical costs excluded: Mitigation Area Easements, Utility Relocations, Fish and Wildlife Facilities, Recreation Facilities.

Alternative 0 is the highest capital cost due to acquisition of four residential properties in the analysis extent, the costly CR16/I29 interchange reconstruction and the longer I-29 grade raise costs.

Alternative 5 is the lowest capital cost. The savings achieved by not acquiring four residential properties and constructing less length of I-29 grade raise appear to more than offset additional capital costs of the longer dam and a taller control structure.

For a detailed breakdown of estimated costs for each alternative see Appendix C.

3.2 RISK REDUCTION

Limiting risk to the public by constructing a system that limits risk of failure of critical features is important to consider when evaluating alternatives. Once constructed, the project will be in operation for many years. Long term risk considerations are necessary to evaluate alternatives.

Risk characterization generally considers the risk of failure or robustness of an alternative, and the consequences that could result from a possible failure. The lowest cost or easiest to construct alternative is not necessarily the most favorable from a risk-management perspective. Increased risk exists where, for a given set of alignment features, the flood risk reduction features have a greater potential exposure to the risk of being compromised and ultimately failing. The risk is greater where a potential failure can cause greater loss of life or damage to property.

Risk factors identified during the workshops with input from Diversion Authority and USACE representatives were not intended to be a complete list of all potential risk factors. However, for a relative comparison of alternatives, the risk factors considered in this assessment provide a preliminary characterization of initiators and failure mechanisms, with consequence of failure being equally high for all alternatives. This assessment does not replace or supersede a more formal Potential Failure Mode Analysis (PFMA), which was outside the scope of the analysis agreed upon by the working group.

For comparison of risk, several big-picture risk parameters were identified for consideration as risk factors. The characterization of risk reduction presented in the PFSAA serves as a basis from which this analysis borrows to assess alternatives. At the August 22, 2013 LSLCTT meeting the following risk characterizations were developed by the group for the WRR micrositing alternatives:

- Minimizing the length of dam would be viewed as a reduction of risk.
- Minimizing the length of I-29 road raise would be viewed as a reduction of risk.
- Minimizing the number of I-29 bridge crossings would be viewed as a reduction of risk.
- Minimizing the flow rate and velocity of Wild Rice River flows under I-29 would be viewed as a reduction of risk.

Table 3.4 Risk Assessment Factors

	unit	Alt. 0	Alt.3	Alt. 5
Length of Dam (CR17 to Red River)	LF	30,300	33,152	34,400
I-29 Road Raise Length (Total)	Miles	4.48	4.03	3.79
Number of I-29 Bridge crossings	No.	2	2 or 4	4
Dependence on Staging Area Conveyance Under I-29	Relative	More	More	Less

Alternative 0 is favorable from a risk perspective because it has a shorter dam length than either Alternative 3 or 5. Alternative 0, however, requires the longest length of I-29 road raise of the alternatives.

Alternative 3 has a longer dam length than Alternative 0, but a shorter dam length than Alternative 5. Similarly, the length of I-29 embankment is shorter than Alternative 0, but longer than Alternative 5.

Alternative 5 is favorable from a risk perspective considering the placement of the Wild Rice Control Structure west of the I-29 embankment. If for some reason this opening became plugged or compromised, it may be advantageous to have one control structure able to operate on the eastern portion of the staging area (the Red River Control Structure), and one on the western portion of the staging area (the Wild Rice Control Structure). Alternative 5 requires the shortest length of I-29 road raise required. Alternative 5, however, has the longest dam length of the three alternatives.

3.3 PROPERTY IMPACTS

The number of structures (in particular-residential structures) impacted varies between the alternatives. For the purpose of this analysis, an impacted structure is defined as a structure that is located under the footprint of the project and would require purchase or is within the defined staging area and would require purchase or other mitigation measures. Additionally, the residential structures within the staging area are categorized based on estimated depth of inundation. In general, the fewer number of residential structures impacted, the more favorable an alternative.

Table 3.5 Property Impact Summary

Cost Category	Alternative 0	Alternative 3	Alternative 5
Number of Residential Properties Impacted in the Extent of Analysis	4	0	0

Both Alternative 3 and 5 allocate four (4) additional residential properties to the risk reduction area instead of impacting those same properties in the staging area. In this way, Alternatives 3 and 5 are equally more favorable than Alternative 0.

3.4 ENVIRONMENTAL IMPACTS

The scope of work for this task did not extend to preparing an extended analysis of environmental impacts, so a detailed comparison of these impacts cannot be performed. However, several factors were calculated to roughly compare the scalar change in environmental impacts for the WRR micrositing alternatives:

- Wetland Impact: Wetlands were not field or photo-delineated in this phase of work. Actual field delineation of wetlands is preferred to estimate wetland impacts. However, given similar land use, constructed project footprint area can be used as a rough proxy. In general, a project with a larger footprint has a greater chance to impact wetlands.
- River miles impacted: Closure structures resulted in the abandonment of some river channel, resulting in riparian habitat impacts. Due to the configuration of each alternative, some impacted more river channel than others. The assumption of filling the abandoned channel is consistent with impacts presented in HMG's estimates provided to USACE in Dec. 2012 for the SDEIS.
- Cultural resources surveys have not yet been performed for this area, but will be completed for areas that will be impacted. Areas along the river have a higher probability of having cultural resources.

Detailed impact and construction quantities set forth in the 404B Permit were not calculated as part of this effort, but will be required for a preferred alternative in future efforts.

Table 3.6 Environmental Impact Considerations

Alternative	Length of Existing Wild Rice River realigned (LF)
Alternative 0	3,200
Alternative 3	5,500

Alternative	Length of Existing Wild Rice River realigned (LF)
Alternative 5	3,300

The Alternative 0 impact is the baseline for comparison. Both Alternative 3 and 5 have a greater potential length of impact to the existing Wild Rice River. The Alternative 3 channel realignment potentially impacts 2,300 L.F. of additional river, which can be viewed as unfavorable. The Alternative 5 channel realignment potentially impacts 100 L.F. of additional river, and perhaps with detailed design efforts this could be reduced.

3.5 GEOTECHNICAL

The findings of this field investigation are incorporated into this report. Key findings of the geotechnical assessment are as follows:

- **Stability Analysis 1**: stability analysis of the north abutment of the I-29 bridge over the Wild Rice River (including I-29 grade raise) indicates that a 7H:1V embankment and excavation slope meets the required factor-of-safety based on this preliminary analysis. This analysis does not differentiate any of the three micrositing alternatives as more or less favorable.
- **Stability Analysis 2**: for alternative 5, stability analysis of the existing Wild Rice River banks are an issue of concern that should be investigated in more detail during detailed design. This finding is further corroborated by the fact that just downstream of the site, bank failures along the existing Wild Rice River can currently be observed. This analysis does not differentiate any of the three micrositing alternatives as more or less favorable.
- **Stability Analysis 2**: this analysis of Alternative 5 where the right bank of the Wild Rice River approaches the proposed I-29 grade raise indicates that a global failure of the I-29 grade raise into the Wild Rice River is not a concern with the offsets shown. This analysis indicates that larger offset of the Wild Rice Control Structure from the I-29 grade raise could be viewed as favorable.
- **Stability Analysis 3**: at the wingwalls of the control structure, there is concern about the influence of the dam/embankment on the stability of excavated slopes. Modeling based on currently anticipated conditions suggest that the dam and re-aligned Wild Rice River channel with no offset between the channel crest and the dam toe are stable and meet the required factor-of-safety values for drained and undrained conditions. This analysis does not differentiate any of the three micrositing alternatives as more or less favorable.
- **Preliminary Settlement Analysis**: this analysis provides information to assist in planning settlement mitigation measures for the I-29 grade raise, investigating construction sequencing and constructability.
- **Preliminary Wick Drain Design**: this analysis provides guidelines for conceptual wick drain spacing and a basis for comparing settlement mitigation costs associated with the three micrositing alternatives. Less length of I-29 grade raise constitutes a potential savings due to less footprint area of embankment requiring settlement mitigation.

In general, fewer abutments, fewer bridges, less I-29 road raise and less dam length are favorable from a geotechnical perspective. Future design efforts may benefit from evaluating the favorability of alternatives based on construction sequencing and staging considerations.

Alternative 0 is less favorable because it has the longest length of I-29 road raise of the alternatives. However, Alternative 0 is also favorable because it has the shortest length of dam.

Alternative 3 has an I-29 road raise length less than Alternative 0, but greater than Alternative 5. The Alternative 3 dam length is greater than Alternative 0, but less than Alternative 5.

Alternative 5 is favorable because it has the shortest length of I-29 road raise of the alternatives. However, it has the highest number (4) of I-29 roadway bridges requiring abutments. Alternative 5 has the longest length of dam, which is unfavorable.

Estimated excavation and embankment slope generally affect all three alternatives similarly and are not a factor of major differentiation between the alternatives.

3.6 HYDRAULICS

Realigning the dam southward removes acreage from the staging area. A preliminary analysis of the effect of realigning the dam southward was performed to estimate the order-of-magnitude effect on staging elevation for the alternatives. Based on hydraulic modeling performed to date, the difference in staging elevations between the alternatives is small (0.2’ or less). Therefore, the staging elevation will not significantly differentiate the alternatives and the difference in staging elevation was not taken into account in calculating lands and easement acreages.

Table 3.7 Impact to 1-percent Staging Elevation

	Alt. 0	**Alt.3**	**Alt. 5**
Volume Removed from Staging Area (acre-feet)	0	3,191	5,249
Surface Area Removed from Staging Area (acre)	0	251	419
Remaining Staging Area Surface Area (acre)	31,708	31,456	31,289
Impact to Staging Elevation for 1-percent Event (feet)	0.0	0.1	0.2

Alternatives 0 and 3 both construct the Wild Rice Control Structure east of I-29. In Alternative 5, it is constructed west of I-29. Construction of the Wild Rice River control structure west of I-29 may also offer some advantages from a resilience perspective, providing flow redundancy for the staging area by constructing a control structure both east and west of I-29. This could be advantageous for construction phasing, or in the event that the staging area flow connection under I-29 were to be compromised for any reason, or in the event that one of the control structures (at the Wild Rice River and the Red River) is under rehabilitation or repair in the future.

From a resiliency standpoint, there may be an advantage to constructing it west of I-29 because of how I-29 splits the staging area into two separate pools for events up to the 1-percent event. An opening under I-29 allows water to equalize on both sides of the I-29 embankment. If for some reason this opening became plugged or compromised, it may be advantageous to then have one control structure able to operate on the eastern portion of the staging area (the Red River Control Structure), and one on the western portion of the staging area (the Wild Rice Control Structure).

Alternative 5 reduces the flow rate of the Wild Rice River under I-29 crossing over the river. This is advantageous in terms of the open section required at the crossing as well as possibly lessening the costs associated with erosion protection at the crossing.

3.7 TRANSPORTATION SAFETY

The degree to which alignment alternatives alter the transportation safety characteristics of the roads and railroads in the project vicinity is characterized versus the Recommended Alternative. In general, alternatives with less water alongside roadways and fewer bridges across the connecting channel pose fewer transportation safety issues from the perspective of this investigation. The analysis for this study generally includes micrositing for the area 1 mile west of the I-29/CH-16 interchange to 1 mile east the interchange, including the Wild Rice Hydraulic Structure, realigned Wild Rice River, and dam/embankment staging area. Therefore, the transportation safety considerations focused on differences in this corridor.

A consistent set of simplified criteria was used for developing transportation feature modifications for the alternatives considered in the micrositing study. The alternatives include raising the minimum edge of driving lanes for Interstate 29 to the maximum staging elevation for the 1-percent chance flood event (922.2). Based on hydraulic modeling performed to date, the difference in staging elevations between the alternatives is small (0.2' or less). Therefore, the staging elevation will not significantly differentiate the alternatives and the difference in staging elevation was not taken into account in calculating transportation impacts.

The alternatives will require the construction of bridges at major roadways and railroads along the connecting channel. The bridge design criteria utilized for this analysis is the same as for the Phase 4 technical reports prepared in support of the FR/FEIS and used for the PFSAA. Major transportation routes that will be impacted in the corridor evaluated for this study are Interstate 29 and Cass County Highway 16. The impacts are summarized below:

Alternative 0: For Alternative 0 within the study corridor, Interstate 29 will be raised through the upstream staging area and the Cass County Highway 16 overpass on I-29 will be shifted north and reconstructed to accommodate the transition from the raised grade in the staging area to the normal grade within the flood damage reduction area. New bridges will be constructed on Interstate 29 (NB and SB) at the Wild Rice River. During normal conditions, these bridges will allow flow from the Wild Rice River to pass east towards the Wild Rice River Control Structure and through the Flood Damage Reduction Area. During project operation, water will pass west through the bridges towards the Diversion inlet. This alternative also includes the construction of a temporary detour bridge over the Wild Rice River west of I-29 to accommodate traffic during project construction.

A plan and profile of I-29 for this alternative is presented in Figure 4.

Alternative 3: For Alternative 3 within the study corridor, Interstate 29 will be raised through the upstream staging area, however the Cass County Highway 16 overpass is far enough north of the embankment that it will not need to be relocated. New bridges will be constructed on Interstate 29 (NB and SB) at the re-aligned Wild Rice River. During normal conditions, these bridges will allow flow from the Wild Rice River to pass east towards the Wild Rice River Control Structure and through the Flood Damage Reduction Area. During project operation, water will pass west through the bridges towards the Diversion inlet. The existing Interstate 29 bridges at the Wild Rice River will be removed and replaced by culverts to handle local drainage. This alternative also includes the construction of a temporary detour bridge over the Wild Rice River west of I-29 to accommodate traffic during project construction. A plan and profile of I-29 for this alternative is presented in Figure 5.

Alternative 5: For Alternative 5 within the study corridor, Interstate 29 will be raised through the upstream staging area, however the Cass County Highway 16 overpass is far enough north of the embankment that it will not need to be relocated. New bridges will be constructed on Interstate 29 (NB and SB) at the connecting channel. During normal conditions, these bridges will not pass water other than local drainage; however, during project operation water will pass east and west through the bridges towards the Diversion inlet and to balance the staging area across the divide of the interstate embankment. The existing Interstate 29 bridges at the Wild Rice River will remain at their current location. This alternative also includes the construction of a temporary detour bridge over the Wild Rice River west of I-29 to accommodate traffic during project construction. A plan and profile of I-29 for this alternative is presented in Figure 6.

The safety of the traveling public was the primary consideration when comparing alternative. Since the number of bridges is similar for all three alternatives, the length of grade raise in the upstream staging area was the primary consideration. Drivers lack escape routes when crossing bridges and traveling in areas with water adjacent to the roadway. This increases the potential for accidents. Table 3.8 provides a summary of transportation safety considerations for the alternatives evaluated as part of this micrositing study.

Table 3.8 Transportation Safety Considerations

Alternative	Number of I-29 Bridges	Length of I-29 Grade Raise (miles)
Alternative 0	2	4.48
Alternative 3	2[A]	4.03
Alternative 5	4	3.79

[A] Two or four, depending on the feasibility of culverts replacing the existing I-29 bridges over the Wild Rice River.

4 CONCLUSIONS

The WRR micrositing alternatives were subjected to the assessment criteria comparative analysis. This report includes a general comparison of the assessment factors outlined above based on project features and operation comparable to the updated FRP.

The purpose of this study is to evaluate and compare WRR micrositing alternatives. The study includes a quantitative and qualitative comparison of micrositing options based on several assessment factors. The intent of this characterization is to provide decision makers with an assessment of these factors and estimated costs to aide in selecting a preferred micrositing layout. Key conclusions from this study are as follows:

- It is recommended that the I29/CR16 interchange and ramps be protected in place at its current siting. The NDDOT has identified this as an advantage going forward.
- It is recommended that the dam alignment be shifted to allocate the four (4) residential properties east of the I29/CR16 interchange into the flood risk reduction area (north of the dam).
- Detailed design of construction staging, geotechnical slopes and offsets and mitigation options require site-specific field investigations and may affect the comparative favorability of the micrositing alternatives.

Because of both the estimated construction cost and assessment factors, Alternatives 3 and 5 are technically favorable compared to Alternative 0, based on the project definition available at this time.

The main findings of this analysis were presented at the April 17th, 2014 LSLCTT meeting. The feedback received from the group indicated that Alternative 5 is the most favorable of the three alternatives investigated. The basis for this opinion is:

- Increased resiliency achieved by having control structures on both the east and west sides of the I-29 road raise embankment.
- In comparison to Alternative 0, transfer of 4 impacted properties into the risk reduction area.
- Less length of impact to the existing Wild Rice River than Alternative 3.
- Less length of I-29 road raise embankment to deal with during construction and settlement mitigation.
- Favorability from the NDDOT perspective as it makes replacement of the existing I-29 bridges over the Wild Rice River independent of the flood risk reduction project.
- Favorable transportation safety due to a shorter length of I-29 grade raise.

5 CERTIFICATION

The preliminary analysis and conclusions provided are based on the limited data and project definition available at the time of this analysis. Using generally accepted engineering methods and practices, analyses have been performed using reasonable effort to characterize the site and proposed alternatives.

Certification:

I hereby certify that this memorandum was prepared by me or under my direct supervision and that I am a duly licensed Professional Engineer under the laws of the State of North Dakota.

C. Gregg Thielman

____

Gregg Thielman, HMG, LLC

PE #: 3777

Matthew R. Metzger

____

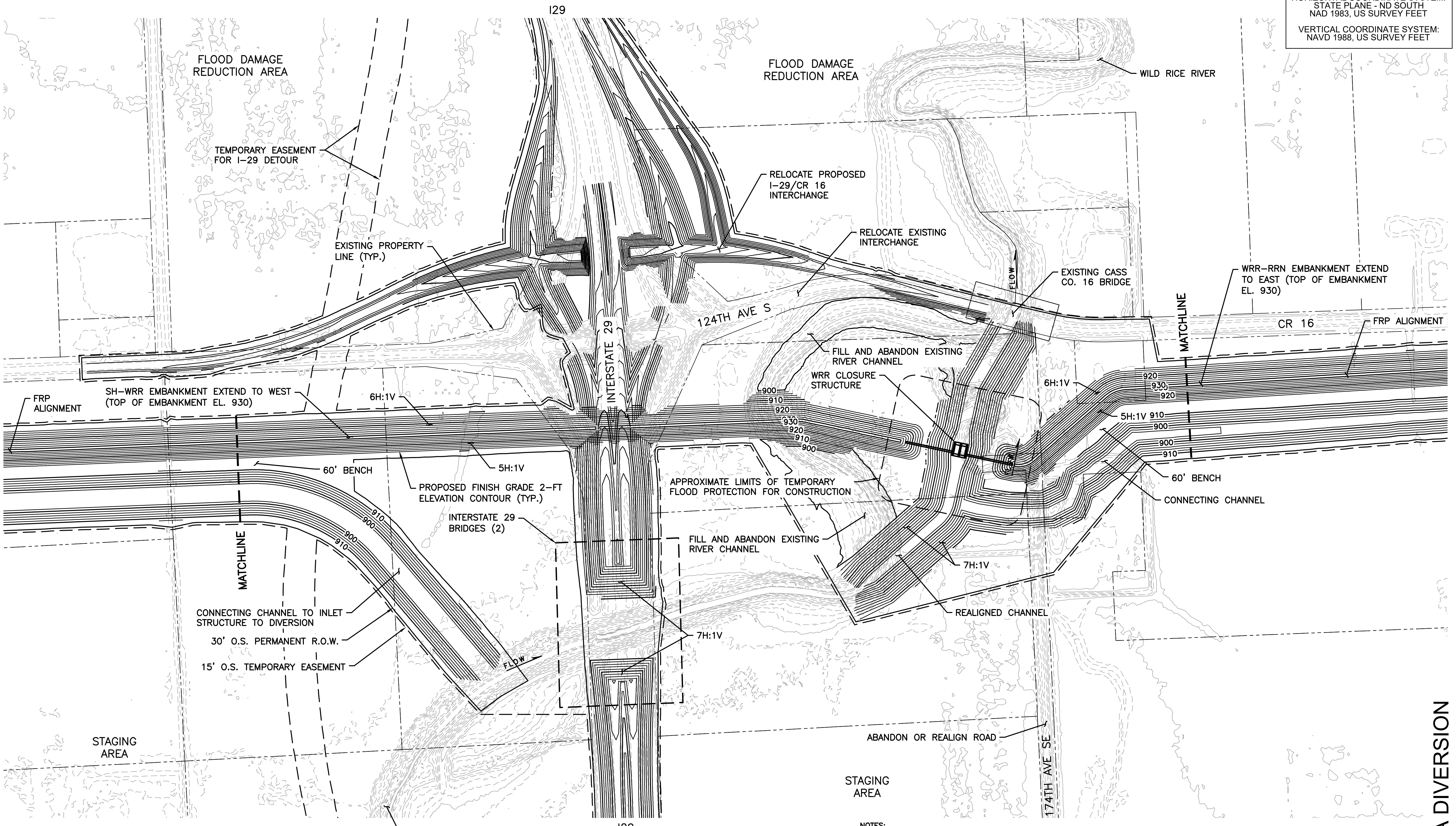
Matt Metzger, HMG, LLC

PE #: 9064

FIGURES

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET



PLAN VIEW

VE13A - WRR - ALTERNATE 0

300 0 300 600
SCALE IN FEET

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NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING AND COMPARISON ONLY. ADDITIONAL EMBANKMENT ALIGNMENT AND CONTROL STRUCTURE SITING MODIFICATIONS SHOULD BE CONSIDERED IN FUTURE DESIGN PHASES.
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4. EXCAVATION AND EMBANKMENT SLOPES AND GRADING OFFSETS SHOWN ARE PRELIMINARY, PENDING ADDITIONAL SITE SPECIFIC SOIL INVESTIGATIONS AND GEOTECHNICAL ANALYSIS.
5. REALIGNED CHANNEL RIPRAP, MAINTENANCE BUILDING, UTILITIES, ICE/DEBRIS CONTROL MEASURES, SCADA MONITORING POINTS AND ACCESS ROADS ARE A PART OF EACH STRUCTURE BASED ON THE SAME ASSUMPTIONS AS IN PHASE 4 BUT ARE NOT SHOWN; QUANTITIES ARE INCLUDED IN COST ESTIMATE AND ARE CONSIDERED COMPARABLE TO PHASE 4 QUANTITIES.
6. RIGHT OF WAY SHALL BE REVISED AS NEEDED DURING DETAILED DESIGN TO MEET USACE & CONSTRUCTIBILITY REQUIREMENTS AT THAT TIME.

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DATE: 7-25-2014
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CKD BY: MRM
SOLICITATION NO.: DIV. AUTH. TASK ORDER 14
SUBMITTED BY: HMG
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 6/17/2014
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: -

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - VE13 OPTION A
STRUCTURE ON WILD RICE RIVER
PLAN VIEW - ALTERNATE 0

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SHEET IDENTIFICATION
FIG 1

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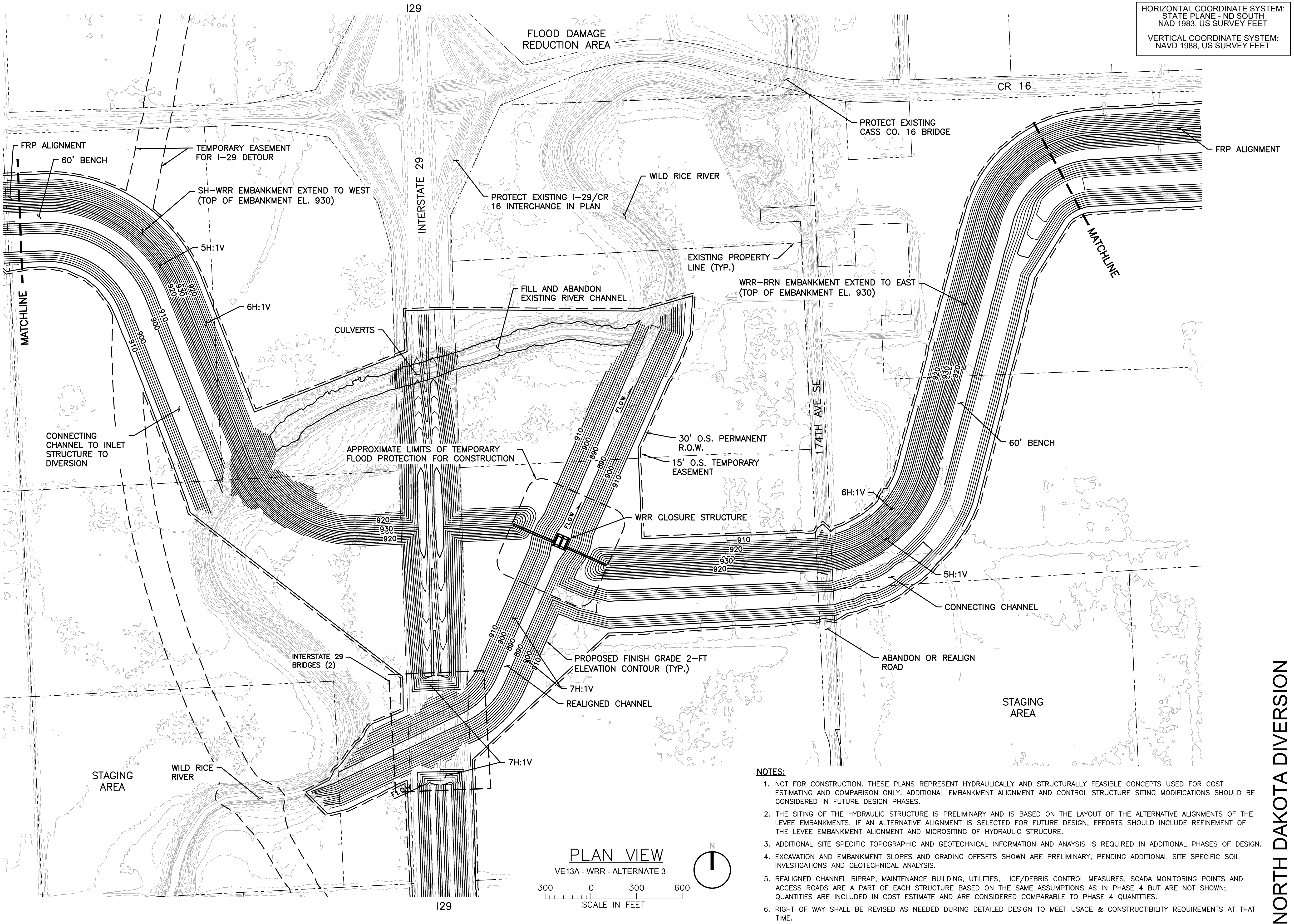

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FILE NAME: -
DATE: 7-25-2014
SOLICITATION NO.: DIV. AUTH. TASK ORDER 14
CONTRACT NO.: N/A
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CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - VE13 OPTION A
STRUCTURE ON WILD RICE RIVER
PLAN VIEW - ALTERNATE 3

SHEET IDENTIFICATION
FIG 2

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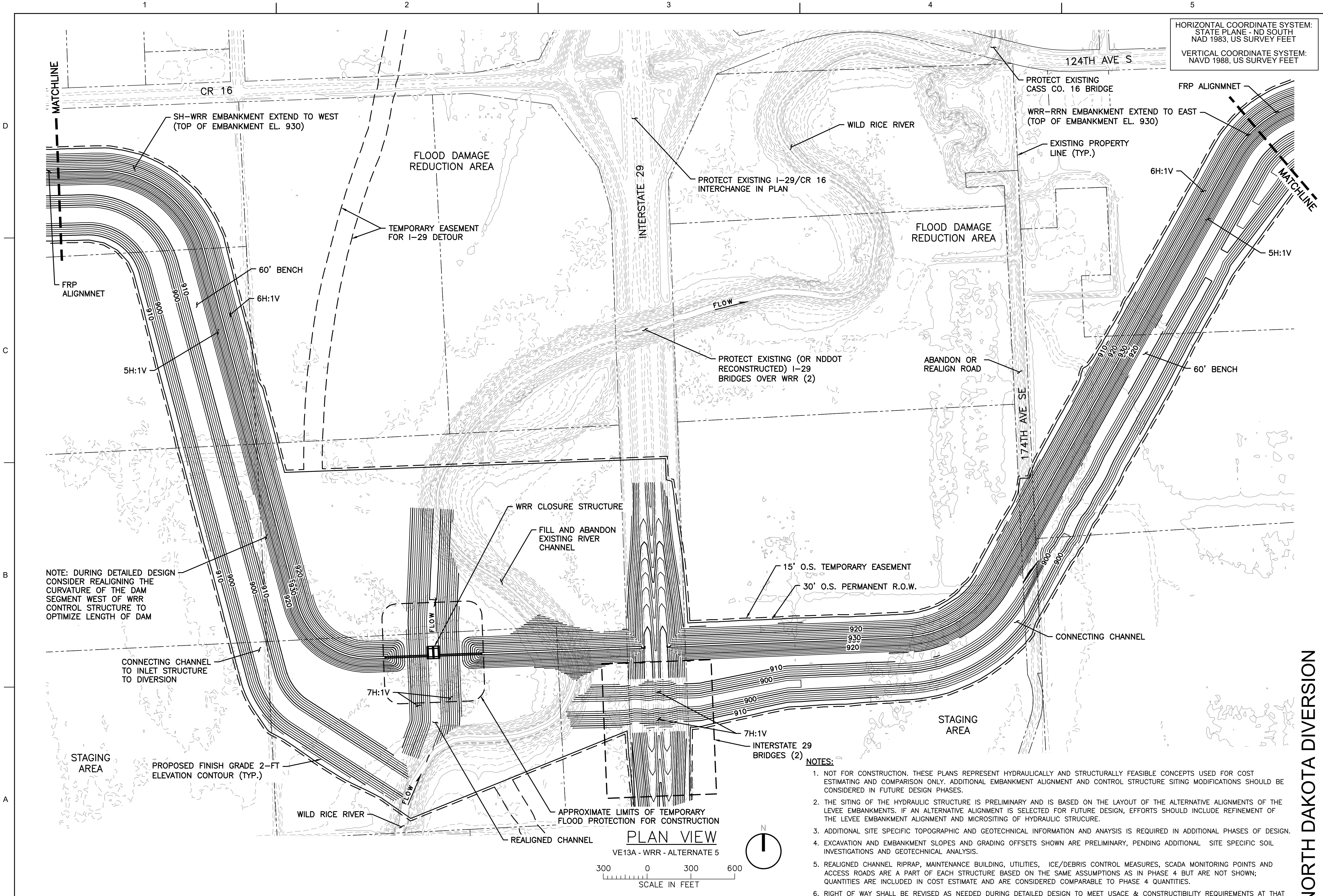

PLAN VIEW

VE13A - WRR - ALTERNATE 5

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FLOOD CONTROL - VE13 OPTION A
STRUCTURE ON WILD RICE RIVER
PLAN VIEW - ALTERNATE 5

SHEET
IDENTIFICATION
FIG 3

CADD USER: Matthew R. Metzger FILE: M:\DESIGN\34091004.13\34091004.13_C3D_DESIGN_ALT 5.DWG PLOT SCALE: 1:2 PLOT DATE: 7/25/2014 1:29 PM

STATE	PROJECT NO.	SECTION NO.	SHEET NO.
ND			

Note: 1) Top of bridge elevation based on Q500 Elevation plus 1 ft freeboard (Elev 929.53)
2) Top of levee elevation based on PMF elevation plus 5 ft freeboard (Elev 929.90)
3) Bridge width based on VE13A geometry
4) Bridge assumed to be 63IN beams with 8IN deck

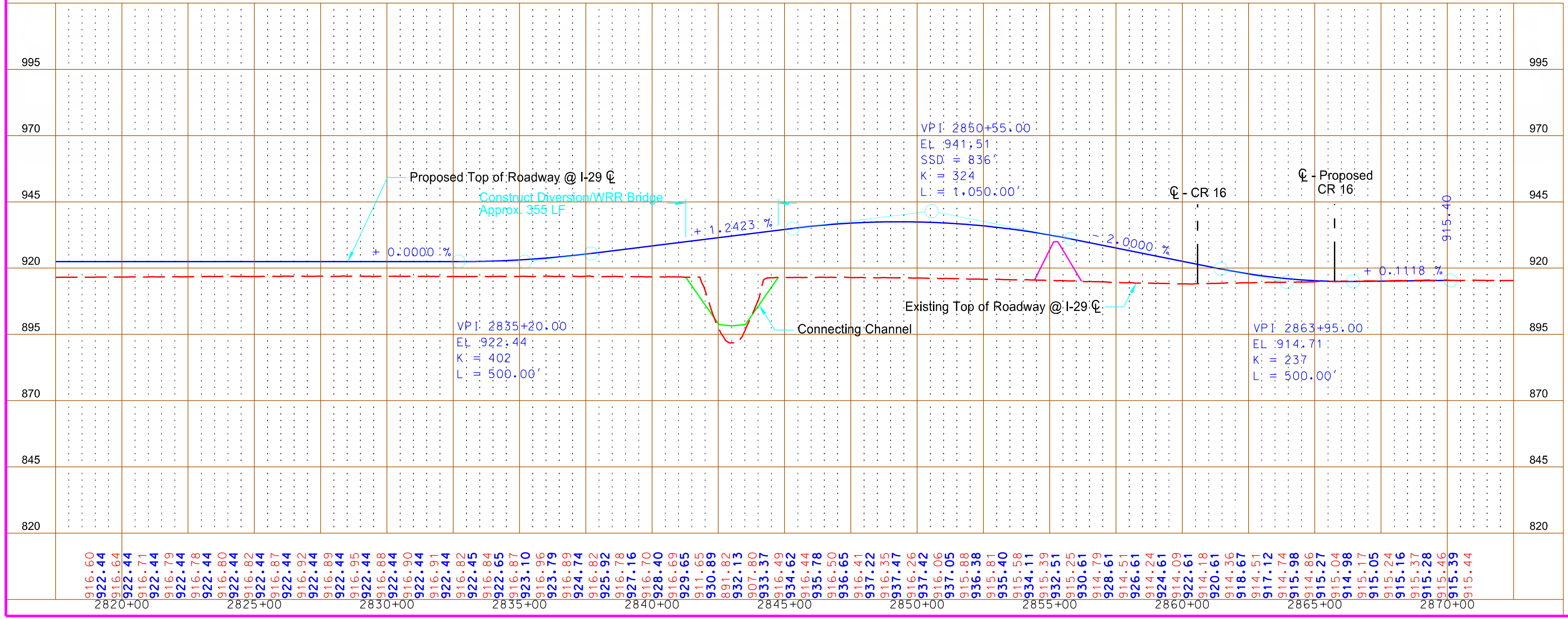

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Figure 4 - Alternative 0
Transportation Feature Summary

CR 16 Interchange Study

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STATE	PROJECT NO.	SECTION NO.	SHEET NO.
ND			

Note:
1) Top of bridge elevation based on Q500 Elevation plus 1 ft freeboard (Elev 929.53)
2) Top of levee elevation based on PMF elevation plus 5 ft freeboard (Elev 929.90)
3) Bridge width based on VE13A geometry
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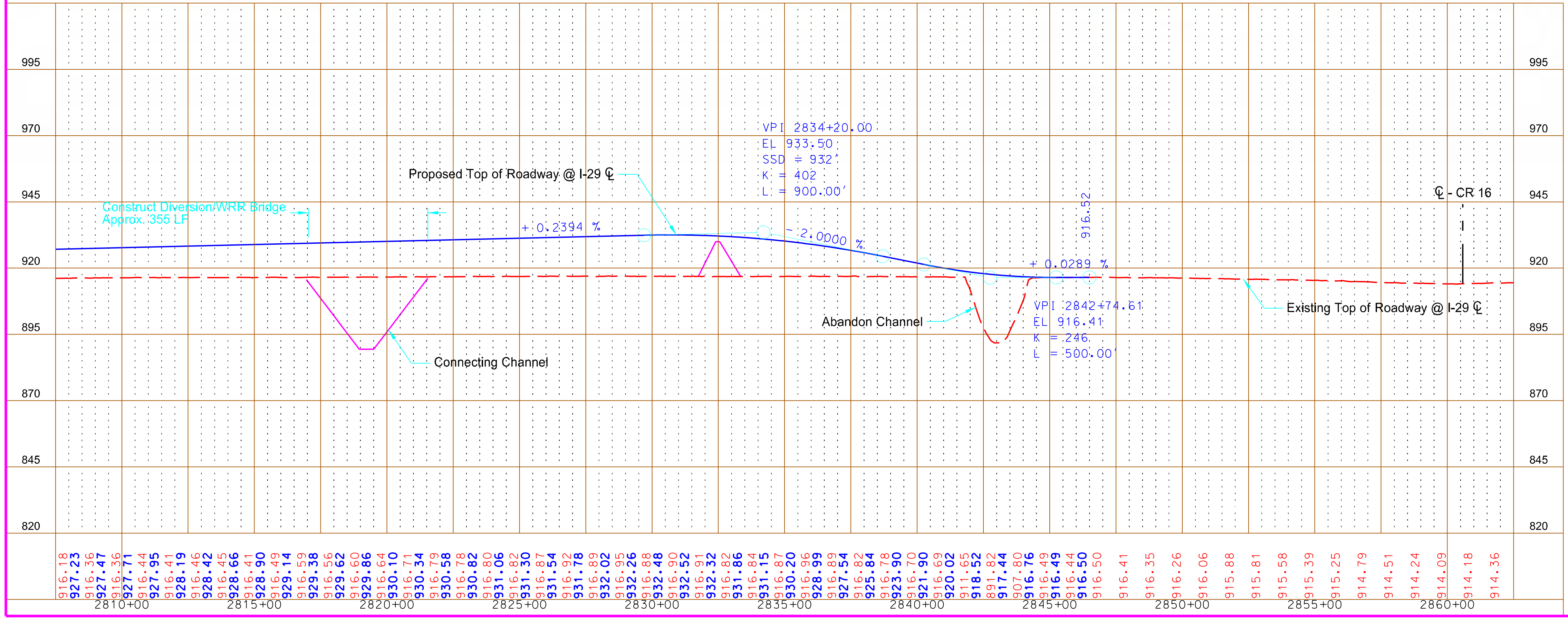

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PRELIMINARY

Figure 5 - Alternative 3
Transportation Feature Summary

CR 16 Interchange Study

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STATE	PROJECT NO.	SECTION NO.	SHEET NO.
ND			

Note: 1) Top of bridge elevation based on Q500 Elevation plus 1 ft freeboard (Elev 929.53)
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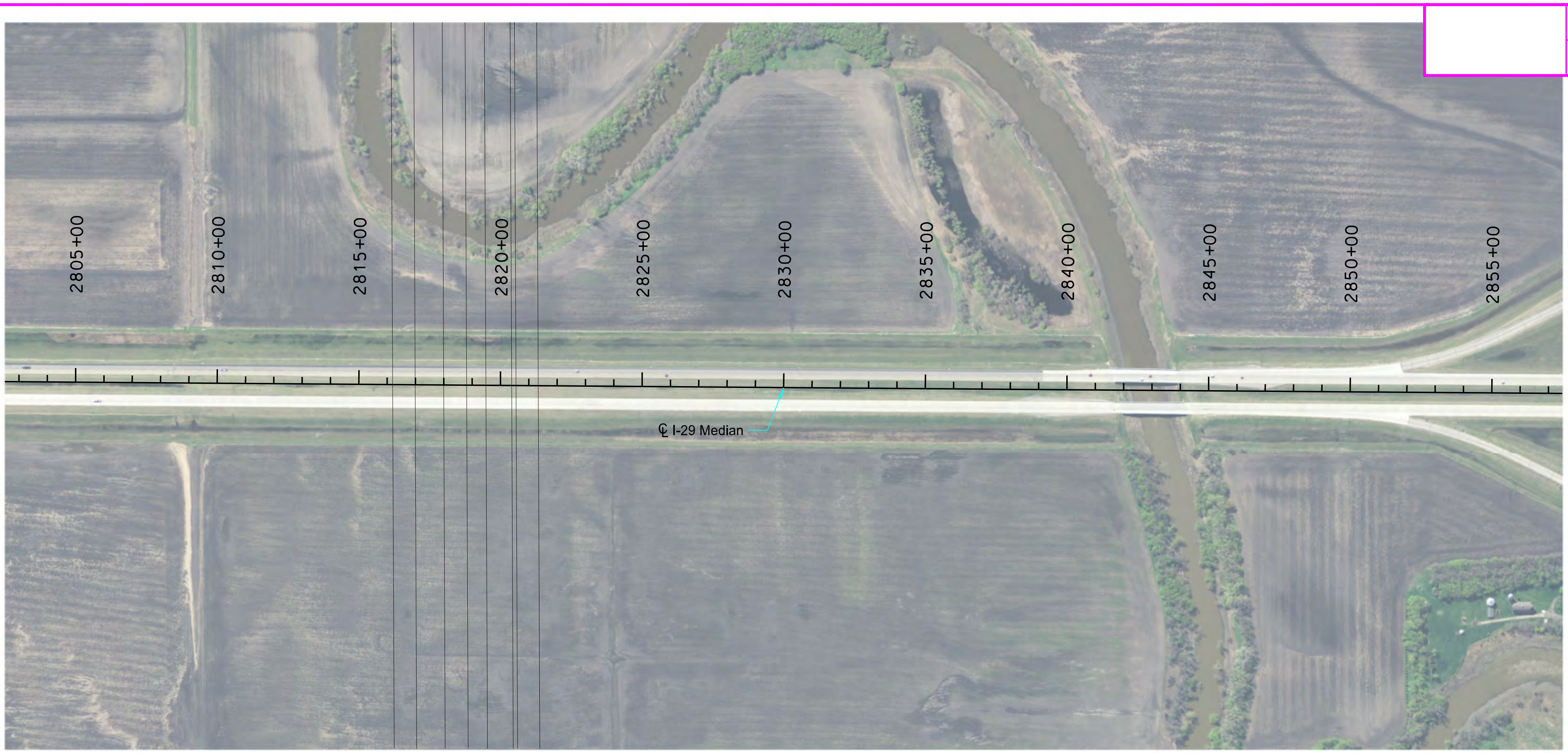

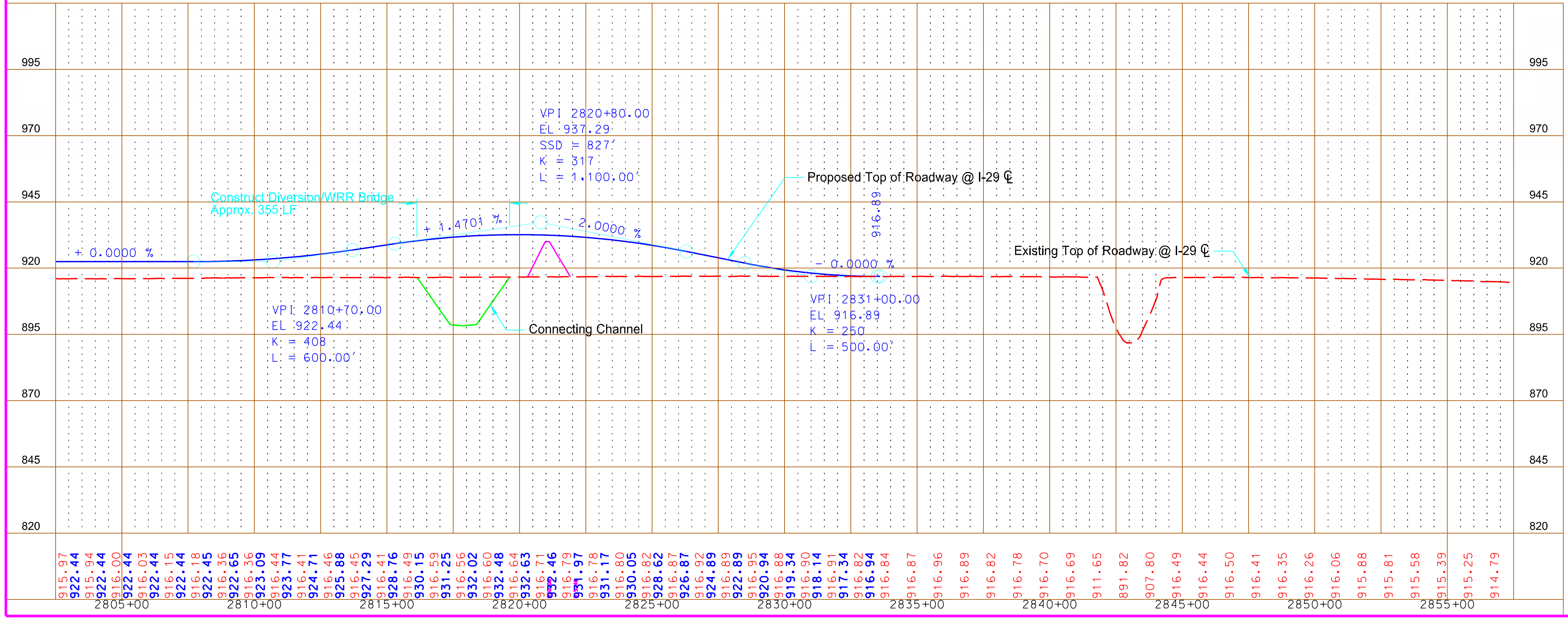

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Figure 6 - Alternative 5
Transportation Feature Summary

CR 16 Interchange Study

APPENDIX A – TASK ORDER 14, SUBTASK C, I-29 SOUTH CONCEPTUAL DESIGN OF INTERCHANGE; TECHNICAL MEMORANDUM – MICROSITING ALTERNATIVES SCREENING

Task Order 14, Subtask C
I-29 South Conceptual Design of Interchange

Technical Memorandum – Micrositing Alternatives Screening

August 16, 2013

This technical memorandum presents preliminary layouts for micrositing of the Wild Rice River (WRR) Control Structure and nearby I29 South interchange. The evaluation is in-process at the time of preparing this memo. This memo is intended to provide background information, to share initial layout concepts for comments from the design team, USACE and Local Sponsors, and to present the general methodology that will be used in the next steps of the micrositing evaluation. Feedback from the group will be incorporated into the next steps of the evaluation.

Background

The I-29 South Conceptual Design of Interchange study is included as part of Flood Diversion Authority Task Order 14. The main goal of the study is to evaluate the conceptual design of the Interstate 29 (I-29) and Cass County Highway 16 (CH16) Interchange, associated road raises, local drainage facilities, and the Embankment and Wild Rice River Crossing. Due to the proximity to the WRR Control Structure, an evaluation of the micrositing for the WRR Control Structure will also be performed. The VE-13A alignment was selected following the Post Feasibility Southern Alignment Alternatives (PFSAA) evaluation. At that time, micrositing was not performed for the I29 South interchange and WRR Control Structure. The purpose of this document is to evaluate the micrositing of these features and to recommend a layout for future value-based engineering design work.

Development of Alternatives

Layouts for different alternatives of the I-29 South interchange and WRR Control Structure have been developed using a set of design constraints and criteria developed by the design team. The design constraints and criteria are presented in **Exhibit 1** attached, and are presented to the main stakeholders (USACE and Local Sponsors) for review, comment and approval. The design constraints and criteria provide the framework for a screening-level comparison that is consistent with previous feasibility and post-feasibility evaluations of alternatives for this and other project features.

Layout concepts for a base case and six (6) alternatives were developed and presented to the main stakeholders at the July 25th LSLCTT meeting in Fargo. Conceptual figures of these layouts are presented in **Exhibit 2** attached.

Description of Alternatives

The base case is a layout initially established along the VE-13A alignment, slightly modified from that presented in the PFSAA report. A design meeting on July 25, 2013 identified feature modifications that were clearly preferable and those have been incorporated into the base case. The relative advantages and disadvantages of each alternative layout with respect to that of the base case will be developed as a next step in this assessment. **Table 1** of this memorandum presents a preliminary list of issues to consider for that assessment, including green and red shading to indicate pros and cons, respectively. It is worthwhile indicating that the qualifications in **Table 1** must be understood in relative terms; they are intended to highlight differences, not to necessarily imply that, for instance, a footprint change to the staging area of "Very Much" is actually significant.

Table 1 – Qualitative Description of WRR/I29-South Layout Alternatives and Comparison to Base Case

Parameter	**Alt. 0 (Base Case)**	**Alt. 1**	**Alt. 2**	**Alt. 3**	**Alt. 4**	**Alt. 5**	**Alt. 6**
WRR Control Structure Location	VE-13A Location	VE-13A Location	VE-13A Location	East of WRR, East of I29	West of WRR, West of I29	West of WRR, West of I29	West of I29, East of WRR
Interchange Relocation?	Yes	No	Yes	No	No	No	No
Embankment Length	Base Case	Much Longer	Longer	Much Longer	Longer	Much Longer	Much Longer
404B Impacts to Existing WRR Channel	Base Case (3200 LF)	Much Longer	Similar	Greater	Similar	Similar	Similar
No. of I29 Bridge Crossings	Base Case (1)	1	1	1	2	2	2
Footprint Change to Staging Area	Base Case	Some	Little	Much	Much	Very Much	Very Much

A key feature viewed as advantageous by both the geotechnical team and the transportation team is one I-29 bridge crossing. Two bridges introduces additional abutment, geotechnical and cost issues that can be avoided by using one bridge for both the Wild Rice River and the diversion channel connecting the staging area pool east and west of I29. Other parameters that are likely to factor into the qualitative assessment are additional properties in residential risk reduction area, erosion protection requirements, geotechnical considerations and access to the WRR Control Structure during flooding events.

Next Steps

Estimated construction costs and evaluation parameters will be determined for the base case. Alternatives will be evaluated against the base case using a pre-determined set of assessment criteria. If the working group provides feedback that result in additional alternatives, those layouts will be included in the evaluation. The assessment methodology will qualitatively evaluate and rank alternatives based on the following criteria:

- Estimated Construction Cost
- Property Impacts (Number of Residential Structures)
- Estimated Environmental Impacts: project footprint and 404B impacts
- Transportation Safety Considerations
- Constructability
- Embankment Length (a surrogate for risk)

One or two (2) preferred alternatives will then be selected and developed in greater detail. Conceptual site layouts and grading will be developed in CAD to illustrate orientation and footprints of project features and grading.

In addition to the factors listed above, the alternatives will be evaluated for geotechnical considerations. A preliminary geotechnical evaluation of alternatives will focus on a review of potential configurations for the Wild Rice channel realignment excavation, equalization/borrow ditch (what has been referred to in the past as Connecting Channel), dam, I-29 grade changes, bridge approach fills and abutments. The findings of the geotechnical evaluation will be used to further develop the micrositing layouts and identify advantages and disadvantages of the alternative layouts.

The geotechnical evaluation will focus on the following areas identified from the attached layout alternatives:

Geotech. Analysis	**Configuration**	**Description**	**Evaluation Method**	**Number of Analyses**
1	Alternative 0	Longitudinal I-29 through North abutment into diversion channel connecting staging area	Evaluate Approach Fill Height (xx feet)	1
2		Longitudinal I-29 through dam	Dam crest (xx feet TBD) , I-29 roadway (xx feet TBD)	1
3	Alternative 1 - 4	Evaluate appropriate offset for dam from diversion channel alignment assuming consistent channel configuration for flow capacity	Identify offset for fill heights ranging from 5 feet to 25 feet next to the typical channel (or an excavation for the WRR structure)	5

A technical memorandum will summarize the findings of the alternatives screening, the preferred alternative(s) going forward and provide useful information to use for background in future value engineering work for these project features.

Exhibit 1

WRR/I29 South Interchange Design Constraints and Criteria Summary

Task Order 14, Subtask C
I-29 South Conceptual Design of Interchange

Design Constraints and Criteria for Interchange and WRR Control Structure Siting

August 13, 2013

The I-29 South Conceptual Design of Interchange study included as part of Flood Diversion Authority Task Order 14 will evaluate the conceptual design of the Interstate 29 (I-29) and Cass County Highway 16 (CH16) Interchange, associated road raises, local drainage facilities, and the Embankment and Wild Rice River Crossing. Due to the interdependent nature of this area and the close proximity to the Wild Rice River (WRR) Control Structure, an evaluation of the siting for the WRR Control Structure will also be performed. The purpose of this document is to establish design constraints and criteria that can be used for the preliminary design of alternatives. It also establishes assessment factors that can be used to evaluate alternatives being considered. This evaluation is intended to also serve as background information for a future value engineering study of this part of the project.

Design constraints are critical items that must be adhered to for all alternatives under consideration. The following are identified design constraints:

- Maintain the Post Feasibility Southern Alignment Analysis (PFSAA) staging elevation (922.2) within +/- 0.1 ft for the 1% and 0.2% chance flood events.
- Maintain equal pools on both sides of I-29 for the 1% chance flood event.
- No new residential structures will be added to the upstream staging area.
- I-29 should cross the southern embankment at or near a right angle, per the preference of NDDOT and FHWA.
- I-29 shall be constructed at the 1% chance WSEL to edge-of-driving-lane or higher in the upstream staging area.
- I-29 shall be designed using applicable NDDOT and FHWA design guidelines.
- The RRN control structure location is fixed.
- Match the grade south of the I-29/CH16 interchange to protect the existing interchange for alternative(s) that involve relocating the WRR structure.
- If the interchange is relocated, provide vertical curves with a maximum slope of 2.5% and that meet the required clearances at the CH16 crossing.
- Assume CH16 is constructed over the top of I-29, per the NDDOT preference.
- Avoid new project features on any deed-restricted properties.
- A permanent easement of 30 ft, offset from the extents of the grading work, is assumed at each structure. A temporary construction easement of 15 feet, offset from the extents of the permanent easement, is assumed at each structure.
- Utilizing a "dam" cross section for the levee embankment. The top of dam elevation will be set at the PMF staging elevation plus 5 feet of freeboard, and the assumed side slopes will be 4H:1V for embankment heights under 20 feet and 5H:1V (upstream) and 6H:1V(downstream) for embankment heights of 20 feet and greater as set forth in the PFSAA.
- Assume the channel west of the WRR control structure is the same cross section as presented in the PFSAA and provides a local drainage connection to the WRR. Detailed earthwork balance and determination of this cross section for embankment borrow is not part of this study, but is

recommended as part of a separate effort. A shrinkage factor of 25%-35% is recommended based on experience in other projects using Red River valley clays for levees.

- Separate Fish passage system is not required at this location.
- Appropriate geotechnical offsets shall be maintained for determining excavation and embankment alignments.
- Project features shall be located such that geotechnical considerations are met for settlement, approach fills, connecting channel/borrow trench excavation slope stability and cross section(s), etc.
- Project features shall be staged, sequenced and constructed such that geotechnical considerations are met for settlement, approach fills, cross section(s), etc. of both permanent and temporary features.
- Maintain traffic on I-29 during construction.
- Maintain conveyance on the WRR during construction.
- Provide hydraulic conveyance under I-29 to connect the staging area across the I-29 embankment. Detailed analysis of these flows, 2-D velocity modeling and the hydraulic design of this feature is outside the scope of this study, but is recommended as part of a separate study.
- Assume flows through the WRR Structure are the same as those assumed in the PFSAA.

Design criteria will be used to develop and compare alternatives. The following are identified design criteria:

- Match the length of river impacted (404B) to avoid large increase in impacts to the OHWM.
- Maintain the current I-29 alignment, if possible.
- Maintain the current I-29 right-of-way (ROW) extents, if possible.
- Minimize the length of staging area Embankment as measured between CH81 and CH17.
- Avoid sharp bends (greater than 45 degrees) in the embankment and channel, if possible.
- Provide acceptable construction staging, bridge(s) over the WRR and detouring during construction.
- Adequately design for the hydraulics, geotechnical considerations and erosion potential at the merging of the WRR and connecting channel. It is preferable if there is only 1 bridge over both the WRR and the constructed channel.
- Avoid creating conditions for scour and erosion. Detailed analysis of two-way flow under an I-29 bridge over both the WRR and constructed channel is not part of this study, but is recommended as part of a separate effort.
- Requirements for constructing I-29 on a “dam”.
- Preferably replace the I-29 bridge over the WRR.
- Preferably protect the CH16 bridge over the WRR.
- Use the same criteria for embankment as the Post-Feasibility Southern Alignment Analysis (PFSAA) with the exception of freeboard which is described above under Design Constraints.
- Need to maintain conveyance (channel) from the WRR to inlet.
- No need to maintain conveyance (channel) from the RRN to WRR.
- Roadway and bridge design should also follow NDDOT and FHWA design standards.
- The access road to the WRR structure should tie into CH16, not I-29.
- Satisfactory sizing of the connecting channel as borrow source for levee/dam embankment.
- Hydraulic structures will be constructed off-channel “in the dry”.
- A minimum buffer of 300 feet is included between the proposed gated structure and the existing river banks. This assumption was applied to the gated structures, not the wingwalls.
- The centerline radii of the constructed channels shall be a minimum radius of 3 times the water surface top width in the constructed channel.
- Preferably align the WRR channel perpendicular to the dam embankment.

The Post-Feasibility Southern Alignment Analysis (PFSAA), which evaluated and compared several alternative alignments to the FEIS alignment established assessment factors for the evaluation of alternatives. Similarly, **Assessment Factors** have been established for this study of the I-29 South conceptual design:

1. Construction Costs.
2. Property Impacts (Number of Residential Structures).
3. Environmental Impacts: project footprint and 404B impacts.
4. Transportation Safety Considerations.
5. Constructability.
6. Embankment Length.

Exhibit 2

WRR/I29 South Interchange Preliminary Micrositing Layout Figures

Diversion Channel Alignment Alternatives
CR16 Options 07/25/2013

Alternative 0

—	CR16 Realignment
■	Embankment
🟩	Diversion Channel
—	WRR Channel Realignment
—	Estimated River OHWM Impact
▨	Bridge
☐	VE13A Alignment

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Diversion Channel Alignment Alternatives
CR16 Options 07/25/2013

Alternative 1

■	Embankment
🟩	Diversion Channel
━	WRR Channel Realignment
━	Estimated River OHWM Impact
▨	Bridge
☐	VE13A Alignment

Diversion Channel Alignment Alternatives
CR16 Options 07/25/2013

Alternative 2

- CR16 Realignment
- Embankment
- Diversion Channel
- WRR Channel Realignment
- Estimated River OHWM Impact
- ▨ Bridge
- ☐ VE13A Alignment

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Wild Rice River
25TH ST S
36TH ST S
Protect Existing Interchange
CH16 Bridge (No Modification)
124TH AVE S (CH16)
Wild Rice River
29
29
Wild Rice River Control Structure
174TH AVE SE
I-29 Combined WRR & Diversion Bridge
Staging Area
I29
I29
49TH ST SE
Flow
49TH S
HMG
Feet
0
262.5
525
1,050
Diversion Channel Alignment Alternatives
CR16 Options 07/25/2013
Alternative 3
Embankment
Diversion Channel
WRR Channel Realignment
Estimated River OHWM Impact
Bridge
VE13A Alignment

Diversion Channel Alignment Alternatives
CR16 Options 07/25/2013

Alternative 4

- ■ Embankment
- 🟩 Diversion Channel
- ▬ WRR Channel Realignment
- ▬ Estimated River OHWM Impact
- ▨ Bridge
- ☐ VE13A Alignment

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36TH ST S
25TH ST S
Wild Rice River
Protect Existing Interchange
CH16 Bridge (No Modification)
124TH AVE S (CH16)
I-29 Bridge
Wild Rice River
29
29
174TH AVE SE
Wild Rice River Control Structure
Staging Area
I29
I29
I-29 Bridge
49TH ST SE
49TH S
Flow
HMG
Feet
0
262.5
525
1,050
Diversion Channel Alignment Alternatives
CR16 Options 07/25/2013
Alternative 5
Embankment
Diversion Channel
WRR Channel Realignment
Estimated River OHWM Impact
Bridge
VE13A Alignment
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25TH ST S
Wild Rice River
36TH ST S
Protect Existing Interchange
CH16 Bridge (No Modification)
124TH AVE S (CH16)
I-29 Bridge
Wild Rice River
29
29
Wild Rice River Control Structure
174TH AVE SE
I-29 Bridge
I29
I29
Staging Area
49TH ST SE
Flow
49TH ST S
HMG
Feet
0
262.5
525
1,050
Diversion Channel Alignment Alternatives
CR16 Options 07/25/2013
Alternative 6
Embankment
Diversion Channel
WRR Channel Realignment
Estimated River OHWM Impact
Bridge
VE13A Alignment
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APPENDIX B – TECHNICAL MEMORANDUM: PROPOSED STUDY OF GEOTECHNICAL CONSIDERATIONS – FM DIVERSION POST-FEASIBILITY ALTERNATIVES ASSESSMENT: WILD RICE CONTROL STRUCTURE / CR16 INTERCHANGE STUDY

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Appendix B
Technical Memorandum

To: Mr. Gregg Thielmann – Houston-Moore Group
From: Aaron T. Grosser, PE, and Kristin Alstadt
Subject: Proposed Study of Geotechnical Considerations – FM Diversion Post-Feasibility Alternatives Assessment: Wild Rice Hydraulic Structure and I-29/CR16 Interchange Micrositing
Date: July 15, 2014
Project: 34091004.13, Task Order 14, FY2013
Revision: 2

Barr Engineering Company (Barr) has performed preliminary slope stability and settlement analyses for Task Order 14 of the Fargo-Moorhead Flood Diversion Project. This memo presents the results of these preliminary analyses. The stability analysis was performed to evaluate geotechnical issues and provide a basic framework for the alternatives evaluation for the location of the Wild Rice River (WRR) Control Structure. These possible configurations are applicable to the future I-29 grade changes for the bridge approach fills and include evaluation of stability longitudinally through the abutments. The following table presents the preliminary stability analyses that were performed. **Figure 1** presents the locations where these conditions may apply.

Table 1. Summary of Geotechnical Analyses

Analysis	Configuration	Description	Evaluation Method
1	Alternative 5 (applies to Alt 0 and Alt 3)	Longitudinal I-29 Southbound through the north abutment into the channel	Evaluate approach fill height, design fill offset, and channel stability
2	Alternative 5 (applies to Alt 0 and Alt 3)	Staging effects of the I-29 bridge construction related to the existing Wild Rice River	Evaluate influence of the I-29 roadway on the Wild Rice River stability. Evaluated stability of the I-29 roadway.
3	Alternative 5 (applies to Alt 0 and Alt 3)	Evaluate appropriate offset for dam from Wild Rice River channel re-alignment assuming consistent channel configuration for flow capacity	Evaluate approach dam design height and re-aligned Wild Rice River channel stability

The following presents the stability analyses that were performed, results of the analyses, and recommendations based on the results. A preliminary settlement analysis was performed to determine the anticipated settlement expected to occur due to the addition of the I-29 abutment fill. Ground improvement methods using wick drain design is also presented. All recommendations are intended to be used for planning purposes.

1.0 Geotechnical Data

As part of the slope stability analysis effort, a soil boring (14-207M) was performed by InterState Drilling, LLC, to: (1) determine the stratigraphy in this area, (2) collect soil samples for laboratory testing, and (3) collect in-situ testing data. The boring log is provided in **Attachment A**.

Laboratory testing procedures are described in more detail below.

1.1 Index Properties

Index property testing was mainly performed on disturbed samples and included water content, unit weight, liquid limit, plastic limit, and grain size. The tests were performed following the appropriate ASTM standard. The results of this testing helped identify characteristics of the soils and define the stratigraphy. This characterization allowed identification of the different soil formations at the site including: Sherack, Oxidized Brenna, Brenna, Argusville, and Glacial Till. The index property testing results are displayed on the 14-207M boring log included in **Attachment A**.

1.2 Shear Strength and Consolidation Tests

The laboratory testing results were used to determine the shear strength of the five different soil formations in this area. The shear strength testing included isotropically consolidated-undrained triaxial compression testing with pore-water pressure measurements (R-Bar tests), unconsolidated-undrained triaxial compression testing, consolidation testing, and a proctor test. Some limited index property testing was also performed on undisturbed samples. All laboratory testing results are included in **Attachment A.**

The laboratory testing performed on samples from Boring 14-207M were compared to recommended strength values provided by the USACE[1]. The results of the comparisons between Wild Rice River site-specific strength data and established USACE parameters are summarized below:

- The location of the Wild Rice River structure is near the southern extent of Lake Agassiz. For this reason the Lower Lake Agassiz Clays (highly interbedded Argusville and Brenna formations based on field observation and laboratory results) were analyzed as one material type for effective stress stability analyses. Further discussion of the Lower Lake Agassiz Clays is provided in Design Documentation Report (DR), Oxbow, Hickson, Bakke Ring Levee Attachment D-1, Geotechnical Engineering Parameters dated 3 January 2014(Attachment D-1).

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[1] Fargo-Moorhead Metropolitan Area Flood Risk Management Project – Diversion Channel and Low-Flow Design. Memorandum for Record. CEMVP-EC-D. April 24, 2012.

- Isotropically consolidated-undrained triaxial compression testing with pore-water pressure measurements (R-Bar test) was performed on one Sherack sample at three confining pressures. The results of the drained shear strength values at 15% strain (ultimate strength) are displayed in **Figure 2** along with previously tested results from Oxbow (OHB) samples and tests performed along the entire Red River Valley (FMMFS). The site-specific data test results plot along the established USACE bi-linear parameter line which strength envelope was used for stability modeling. Peak drained strengths for Sherack were also evaluated and are plotted on **Figure 3** along with previously tested results from Oxbow (OHB) samples and tests performed along the entire Red River Valley (FMMFS). The site-specific data test results plot slightly above the established USACE bi-linear parameter line strength envelope which was used for stability modeling of the I-29 abutment raise. This is a conservative approach based on the data collected.

- One unconsolidated-undrained shear strength test performed on an undisturbed sample of Sherack collected from Boring 14-207M was plotted with previously tested results from Oxbow (OHB) samples and tests performed along the entire Red River Valley (FMMFS) as shown in **Figures 4** and **5** for ultimate and peak strengths, respectively. Undrained shear strength testing results for the Wild Rice River sample plotted slightly above the established USACE ultimate strength of 900 psf (**Figure 4**) and slightly below the USACE peak strength of 1400 psf (**Figure 5**). The previously established USACE parameters were applied to the models as representative for Sherack undrained strengths.

- Isotropically consolidated-undrained triaxial compression testing with pore-water pressure measurements (R-Bar tests) were performed on samples from the Oxidized Brenna, Brenna, and Argusville Formations, each at three confining pressures. These strengths were plotted together to determine one drained strength envelope that would be applied to all three layers in the model due to their similar strength values resulting from the laboratory testing. The results of the drained shear strength values at 15% strain (ultimate) and peak strengths are displayed in **Figures 6** and **7** along with previously tested results from Oxbow (OHB) samples and tests performed along the entire Red River Valley (FMMFS). The drained strength design envelopes applied to the models is a non-linear failure envelope for both ultimate (**Figure 6**) and peak (**Figure 7**) strengths. The Wild Rice River site-specific drained strengths used in the models are higher than the established USACE strengths and previously tested Oxbow strengths resulting in upper-bound values.

- Six unconsolidated-undrained shear strength tests were performed on undisturbed samples of Oxidized Brenna, Brenna, and Argusville collected from Boring 14-207M. The results were plotted with previously tested samples from Oxbow (OHB) and along the entire Red River Valley (FMMFS) as shown in **Figures 8** and **9** for ultimate and peak strengths, respectively. Undrained shear strength testing results on Wild Rice River samples all plotted well above the established USACE parameters for all three formations. For modeling purposes, the previously established USACE parameters for undrained Oxidized Brenna, Brenna, and Argusville strengths were applied since they did not impact the modeling results and met the factor of safety requirements. This is a conservative approach based on the data collected.

- All three geotechnical analyses involved the stability of an embankment consisting of fill material. The established USACE Levee Fill material strengths were used to represent this material. The fill permeability was assumed to be similar to the Sherack Formation since it is the source of the fill material. The unit weight of the fill material was based on one compaction test performed on Wild Rice River Sherack material from a depth of 1 to 5 feet. The results are plotted on **Figure 10** along with previously performed proctors in the area. The average value of all the proctors shown resulted in a unit weight of 115 pcf which was also the value of the site-specific proctor justifying the use of this unit weight in the models for all fill material.

1.3 Stability Model Input Parameters

The stratigraphy along the Wild Rice River consists of five units and a compacted material (levee fill) as shown in **Table 2** below based on the Boring 14-207M. These layers are also shown in the model output figures in **Attachment B**. Model input parameters for seepage of the five units are the recommended values provided by the USACE[2]. The permeability values used in the SEEP/W analysis are included below in **Table 2**. The permeability of the compacted embankment material was assumed to be the same as the Sherack Formation having no anisotropy (k_y/k_x ratio = 1).

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[2] Fargo-Moorhead Metropolitan Area Flood Risk Management Project – Diversion Channel and Low-Flow Design. Memorandum for Record. CEMVP-EC-D. April 24, 2012.

Table 2. Material Permeability Properties Summary

Material	Material Model Type	Sample Material	Horizontal Permeability	
			k_y/k_x ratio	k_x [ft/day]
Levee Fill	Sat / Unsaturated	Silty Clay	1	1.13E-02
Sherack	Sat / Unsaturated	Silty Clay	0.25	1.13E-02
OX Brenna	Sat / Unsaturated	Silty Clay	1	1.40E-03
Brenna	Saturated Only	N/A	1	2.80E-04
Argusville	Saturated Only	N/A	1	2.80E-04
Glacial Till	Saturated Only	N/A	0.25	5.70E-02

All of the material parameters were confirmed with site-specific laboratory testing. When applicable, the undrained and drained shear strength envelopes were selected such that approximately one-third of the data points were below the design envelope while two-thirds of the data points were above the design envelope. **Tables 3** and **4** summarize the unit weight and strength properties used in the SLOPE/W analysis. The unit weights are the established USACE parameters, except for the site-specific unit weight of the compacted fill.

Table 3. Material Strength Properties Summary

Material	Unit Weight	Shear Strength Parameters			
		Drained (ESSA)		Undrained (USSA)	
	γ_{sat}	φ' [deg.] and c' [psf]		c [psf]	
	[pcf]	Ultimate	Peak	Ultimate	Peak
Levee Fill	115	c' = 150, phi' = 24 at 1500 psf, phi' = 11	c' = 150, phi' = 28 at 1500 psf, phi' = 21	900	1400
Sherack	115	c' = 0, phi' = 28 at 2000 psf, phi' = 11	c' = 0, phi' = 30 at 2000 psf, phi' = 25	900	1400
Oxidized Brenna	108	See curvilinear envelope in Table 4		900	1000
Brenna	106			575	650
Argusville	110			575 +10psf/ft	825
Glacial Till	123	c' = 225, phi' = 22	c' = 225, phi' = 25	1900	2200

Table 4. Curvilinear Properties Summary

Oxidized Brenna, Brenna, and Argusville Non-linear Failure Envelope			
Ultimate (15%) Strain*		Peak Stress^	
Effective Normal Stress	Shear Stress	Effective Normal Stress	Shear Stress
σ' [psf]	τ' [psf]	σ' [psf]	τ' [psf]
0	0	0	0
1000	600	500	380
2000	1000	1500	950
3000	1350	2500	1350
4000	1700	3500	1720
5000	2000	4500	2100

* See Figure 6
^ See Figure 7

2.0 Stability Analysis

The slope stability analysis was conducted using SLOPE/W, part of the GeoStudio 2012 Version 8.1 software package. SLOPE/W uses the limit equilibrium theory to compute the factor of safety of earth and rock slopes. In the limit equilibrium approach, the geologic material is assumed to be at the state of limiting equilibrium and a factor of safety is computed. Spencer's method was used to calculate the factor of safety of the channel slope in this stability analysis using a 2-foot minimum slip surface depth. This method is considered an adequate limit equilibrium method because it provides a factor of safety based on both force and moment equilibrium.

In SLOPE/W, the critical failure surface was modeled using the entry and exit method. This allows the location of the trial slip surfaces to be chosen manually, or where they will enter and exit the ground surface, with a chosen number of entry and exit points. Once the critical slip surface is found, the technique optimizes the solution of the circular surface, yielding the lowest factor of safety. The pore pressures used in the SLOPE/W model were computed by a SEEP/W analysis. This analysis used steady-state seepage assuming that the channel bottom is completely empty, which represents the most conservative case in terms of slope stability. The groundwater boundary condition was set at a total head of 10 feet below ground surface along the vertical extent as suggested by USACE guidelines for the project which closely matches the depth to groundwater reported in Boring 14-207M of 11 feet below ground surface.

Two types of stability analyses were performed: the Undrained Strength Stability Analysis (USSA) and the Effective Stress Stability Analysis (ESSA). The USSA was performed to analyze the case in which loading or unloading is applied rapidly and excess pore-water pressures do not have time to dissipate during shearing. This approach is often referred to as the end-of-construction case. The ESSA was performed to account for much slower loading or unloading, or no external loading, in which the drained shear strength of the materials is mobilized and no excess pore-water pressures are allowed to develop.

For typical end-of-construction cases, the minimum recommended factor of safety for levees, embankments, and dams is 1.30 according to United States Army Corps of Engineers (USACE) standard EM 1110-2-1913, Table 6-1b (USACE, 2003) and EM 1110-2-2300[3]. Significant amounts of strength data were collected for this project unlike typical projects where smaller datasets with greater variability are encountered allowing for better evaluation of design strength parameters. Previous experience in the Red River Valley indicates that the long-term, drained condition typically controls the design of a project due to the low drained shear strength of the Brenna Formation. Therefore, the USACE selected the target factor of safety values of 1.4 for the long-term or drained conditions for levees and embankments. These coincide with the required minimum factor of safety values for levee stability and were selected to reduce the potential that the diversion channel slopes would fail and result in the implementation of a difficult and expensive fix. For long-term conditions, slopes of a dam and structures nearby are recommended to meet a drained factor of safety value of 1.5 (EM 1110-2-2300).

2.1 Stability Model Results

The following is a summary of the findings of the geotechnical analysis for the Wild Rice River micro-siting alternatives. It should be noted that these analyses do not take into account the effect of previous failures that have occurred along the Wild Rice River. A detailed field reconnaissance has not been performed but is recommended to evaluate any existing scarps and perform a back-analysis to determine the effects of previous failures that have occurred in the area.

2.1.1 Analysis 1

Analysis 1 evaluated the longitudinal stability of I-29 through the north abutment into the diversion channel incorporating the approach fill height for Alternative 5, which would also apply to Alternatives 0 and 3. The analysis consisted of a channel opening with 7H:1V slopes and a 100-foot bottom. The north abutment is currently offset a distance of 46.5 feet from the channel crest consisting of compacted

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[3] Engineering Manual 1110-2-2300. *Engineering and Design: General Design and Construction Considerations for Earth and Rock-Fill Dams*. Department of the Army Corps of Engineers. July 30, 2004.

excavated material with 5H:1V side slopes. The analysis evaluated stability under drained (ESSA) and undrained (USSA) conditions having strengths based on ultimate strengths.

The current channel slopes of 7H:1V meet the minimum recommended factor of safety value of 1.3 for USSA conditions and the typical target factor of safety value of 1.4 for the long-term or drained condition. The undrained analysis was the controlling condition. All failure surfaces resulted in global failures with the slip surface entering through the abutment and exiting at the bottom of the channel.

Based on this analysis, it appears that the north abutment into the diversion channel slopes is stable under the proposed conditions. This analysis is based on mostly USACE values whose strengths are lower than those predicted for the site. Model results are summarized in **Table 5** and the model outputs are provided in **Attachment B**.

Table 5. Analysis 1 Modeling Results

Factor of Safety Values			
Conditions	**Left Bank**	**Right Bank**	**Required**
ESSA	2.29*	2.46	1.40
USSA	1.47	1.55	1.30

* Condition resulted in a shallow failure occurring in the Brenna Formation where the water table enters the channel resulting in a factor of safety of 2.05

2.1.2 Analysis 2

Analysis 2 evaluated the existing Wild Rice River channel and whether any stability effects were a result of the elevation changes to the I-29 roadway and excavations adjacent to the roadway during construction of the bridges, construction staging, and Wild Rice River structure as part of Alternative 0, 3, and 5. The computed factor of safety values for Analysis 2 are based on peak material strengths. The recommended factor of safety value is 1.3 for USSA conditions and for long-term conditions, slopes of the dam and structures nearby, such as the highway embankment, are recommended to meet an ESSA factor of safety value of 1.5.

The analysis first evaluated the right bank of the existing Wild Rice River condition. The modeled river bank is stable under drained and undrained conditions; however, there does appear to be apparent historical failures just downstream of the evaluated section. The same cross-section was evaluated with the proposed I-29 bridge elevation increase. The I-29 raise (approximately 250 feet away from the Wild Rice River) appears to have little to no effect on the Wild Rice River stability and resulted in factor of safety values above the recommended. A third stability of a global failure entering through the I-29 embankment and exiting through the existing Wild Rice River resulted in factor of safety values well above the recommended.

Based on this analysis, drained and undrained conditions for the Wild Rice River channel and I-29 raise meet the recommended factor of safety values. Results for Analysis 2 are summarized in **Table 6** and model outputs from this analysis are provided in **Attachment B**. Potential corrective actions for the I-29 bridge raise include flatter slopes or ground improvement methods (discussed in greater detail in Section 4.0).

Table 6. Analysis 2 Modeling Results

Conditions	Factor of Safety Values			
	Wild Rice River Channel	Global Site Stability	I-29 Abutment Raise	Required
ESSA	2.59	4.20	2.20	1.50
USSA	2.09	2.23	1.87	1.30

2.1.3 Analysis 3

Analysis 3 evaluated the appropriate offset for the proposed dam from the re-aligned Wild Rice River channel. The proposed dam height is 20 feet next to the re-aligned Wild Rice River channel having 7H:1V slopes and a 50-foot channel bottom. The dam slopes on the right side of the alignment are 7H:1V and there is no offset from the crest of the re-aligned channel to the toe of the dam. The dam on the left side of the alignment has slopes of 6H:1V whose offset distance was analyzed (refer to **Figure 1**). The analysis evaluated stability under drained (ESSA) and undrained (USSA) conditions using ultimate strengths.

The analysis resulted in two possible failure conditions. If a surcharge load (in this case the dam) was applied at the crest of the channel with no offset, the slip surface would enter through the crest of the dam and exit through the channel bottom requiring a factor of safety value of 1.5 or greater. However, as the dam offset was increased, the suggested failure surface entry location changed from the dam crest to the top of the channel with the exit location in the channel bottom. Once this slip surface was identified, the height of the dam or the offset of the dam from the channel had no effect on the channel factor of safety value as the slip surface was only concerned with the channel stability requiring a factor of safety value of 1.4 or greater.

Based on the above requirements, a channel with 7H:1V slopes meets the recommended factor of safety values with a dam having no offset (slip surface through the dam crest and channel bottom). The undrained condition resulted in low factor of safety values and was the controlling condition. All undrained failure surfaces were global failures with the slip surface entering through the dam crest and exiting at the bottom of the re-aligned channel. The ESSA condition failure surfaces were very shallow and occurred in the Brenna Formation where the water table enters the re-aligned channel.

A summary of the model results are provided in **Table 7** and the model outputs from this analysis are provided in **Attachment B**.

Table 7. Analysis 3 Modeling Results

Conditions	Factor of Safety Values		
	Left Bank	Right Bank	Required
ESSA	2.11*	2.22*	1.50
USSA	1.30	1.40	1.30

* Condition resulted in a shallow failure occurring in the Brenna Formation where the water table enters the channel resulting in a factor of safety of 1.97

3.0 Settlement Analysis

A settlement analysis was performed for Task Order 14 of the Fargo-Moorhead Flood Diversion Project. The analysis provides the anticipated settlement expected to occur due to the addition of I-29 abutment fill. The method used to calculate settlement was based on one-dimensional primary consolidation theory for an embankment load (Poulos and Davis Method) taking into account settlement due to long term secondary compression but does not consider immediate (elastic) settlement. Secondary compression settlement is anticipated to occur at a constant effective stress in saturated cohesive soils due to plastic adjustment of soil fabrics. The secondary settlement assumes that the material is completely saturated and compressibility of water and soil grains is negligible. For this analysis the soil surcharge load assumes that all settlement has occurred for the current I-29 abutment fill and thus settlement would only result from the additional fill required to meet the design height and volume. The analysis was therefore performed for a 150-foot-wide I-29 embankment crest width with side slopes of 3H:1V and maximum fill to be placed 15 feet above the current maximum I-29 existing elevation. It was also assumed that no consolidation settlement would occur above the water table in unsaturated soils. The analysis was performed in accordance with guidelines outlined within the USACE Engineering Manual (EM) 1110-1-1904, "Settlement Analysis"[4].

---

[4] Engineering Manual 110-1-1904. *Engineering and Design: Settlement Analysis*. Department of the Army Corps of Engineers. September 30, 1990.

3.1 Geotechnical Data

The laboratory testing performed on five samples from Boring 14-207M and on two previously performed borings (SB-17 and SB-18) drilled in close proximity to the area of this study (NTI, 2008[5]) were compared to recommended shear strength values provided by the USACE[6] to verify that the available USACE data along the entire diversion channel alignment was applicable to this region.

3.1.1 Soil Parameters

Each soil layer was evaluated for settlement in 1-foot increments. The middle depth of each increment was used in calculating the average effective vertical stress before the abutment fill is constructed (σ_o'), the pore water pressure (u), and the average additional effective stress due to the construction of the abutment ($\Delta\sigma'$). The unit weights for each soil layer are based on site-specific laboratory testing parameters. Soil layer thicknesses are based on Boring 14-207M and soil design parameters are provided in **Attachment C**. The consolidation settlement was assumed to only occur below the water table in the Sherack, Oxidized Brenna, Brenna, and Argusville Formations. No settlement was assumed to occur in the stiff glacial till.

3.1.1.1 Overconsolidation Ratio

The overconsolidation ratio (OCR) values were determined from seven laboratory consolidation tests that were performed on soils in the region. The overconsolidation ratio was plotted versus depth to confirm that the trend of the OCR values decrease with depth as shown in **Figure 11**. A trendline was fitted to the USACE dataset to provide a relationship of OCR with depth and this was used in settlement calculations. The lowest OCR value in the Argusville Formation and Weathered Till was 1.05 based on USACE data showing that the soils are normally to moderately overconsolidated. Values from the seven consolidation tests performed in the area plot around the trendline indicating that the USACE dataset fits well with the site-specific values.

3.1.1.2 Index Properties

Moisture contents, void ratios, dry densities, compression index, and recompression index values are based the consolidation laboratory results performed on samples collected from Boring 14-207M.

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[5] Wild Rice River, South Side Flood Control. Stability Evaluation of Conceptual Diversion Channel and Levee Construction. Northern Technologies, Inc. (NTI), 2008.

[6] Fargo-Moorhead Metropolitan Area Flood Risk Management Project – Diversion Channel and Low-Flow Design. Memorandum for Record. CEMVP-EC-D. April 24, 2012.

3.1.2 Groundwater Level

Groundwater was encountered at 11 feet during drilling of the 14-207M boring performed in this region. Based on previous work in the area for the project, the groundwater was assumed in the calculations to be 10 feet below existing ground surface.

3.2 Settlement Results

Preliminary time-rate of settlement has been addressed in this preliminary estimate and will be necessary once the development of staging plans and road design is begun so that effects of settlement of fills over time are addressed and mitigated, for example, with wick drains or other ground improvement methods.

The settlement in each soil layer was computed and the total primary settlement was calculated for various approach fill heights, ranging from 1 to 15 feet, presented in 1-foot height changes. This approach was taken to account for the maximum additional fill height that will be added at the existing I-29 location. The decreasing fill heights from 1 to 15 feet will apply to the North and South portions of the highway abutment approach. Results of the primary settlement for each fill height are provided in **Table 8**.

Table 8. Results of Primary Settlement Analysis

Approach Fill Height [feet]	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Primary Settlement [inches]	0.9	1.7	2.5	3.3	4.1	4.8	5.5	6.1	6.8	7.4	8.0	8.6	9.1	9.7	10.2

Since only saturated soils were assumed to consolidate, 5 feet of Sherack Formation was evaluated for 15 feet of fill having approximately 1.1 inches of settlement. The Brenna Formation and Oxidized Brenna Formation were observed to have settlements of around 2.5 and 2.2 inches for a 15-foot high abutment and 4.4 inches of settlement in the Argusville layer resulting in a total settlement of 10.2 inches. The large amount of settlement in the Argusville Formation is attributed to its greater thickness than the above layers. Note that the time to end of primary consolidation for the Oxidized Brenna, Brenna, and Argusville [referred to as the Lower Lake Agassiz Clay] takes much longer to occur (~46 years) compared to the Sherack Formation (<1 year). Consolidation settlement generally varied in an almost linear relationship with embankment height. Depending on height, the native soils along I-29 and the Wild Rice River could experience consolidation settlement ranging from less than 1 inch up to 10.2 inches for a 15-foot approach fill height. The results of these analyses should be used as a guide during design to determine the amount of "overbuild" of the engineered fills (roads, dams, levees) that will be required to maintain the minimum fill height elevations. For example, in the approach alignment, if 15 feet of fill is placed, the anticipated settlement is about 10.2 inches. Therefore the fill height should be constructed at least 10.2 inches higher than designed.

Secondary consolidation analysis resulted in an additional 0.9 inches of settlement anticipated to occur in one log cycle (10 x time to end of primary settlement). Therefore, for a 15-foot abutment height, the total primary and secondary consolidation is expected to be about 11.1 inches. Additional settlement values for each individual soil layer and abutment heights are provided in **Attachment C**.

4.0 Wick Drain Design

Barr has performed a preliminary evaluation for the applicability of vertical drains for Task Order 14 of the Fargo-Moorhead Flood Diversion Project in order to accelerate consolidation of the foundation soils under an applied load. Surcharge loads and wick drains decrease the settlement time and accelerate the rate of strength gain of in-situ foundation soils. The analysis is based on the anticipated settlement results presented earlier in this memo due to the additional I-29 abutment fill that will range from 1 to 15 feet. The method used to calculate settlement was based on the Federal Highway Administration Report No. FHWA/RD-86/168, Prefabricated Vertical Drains, Volume 1, Engineering Guidelines, dated 1986, and the Naval Facilities Engineering Command Soil Mechanics Design Manual 7.01, dated 1986.

For this analysis the soil surcharge load assumes that all settlement has occurred for the current I-29 abutment fill and thus settlement would only result from the additional fill required to meet the design height and volume and any surcharge load that will be applied and then later removed. The analysis was performed for a 150-foot-wide I-29 embankment crest width with side slopes of 3H:1V and maximum fill to be placed 15 feet above the current maximum I-29 elevation. The Sherack Formation was evaluated having a 2-way drainage path. The Oxidized Brenna, Brenna, and Argusville Formations were considered to be one layer referred to as the Lower Lake Agassiz Clay evaluated having a 2-way drainage path at the contact with the Sherack Formation and Unit "A" Till. Assumptions for the preliminary analysis include:

- Radial drainage only
- No backpressure from horizontal drainage
- Soil parameters are constant in time and position
- No drain resistance considered
- No disturbance considered

- Embankment and surcharge loadings occur instantaneously for purposes of settlement calculations
- Wicks drains will be placed in a triangular array
- Wick drain equivalent diameter of 2.1 inches.
- The horizontal coefficient of consolidation (c_h) is based on a weighted average value that includes layer-specific anisotropy. Anisotropy value of 3 was applied to the Sherack Formation and a value of 1.5 was applied to the Lower Lake Agassiz Clay layers. Applying a weighted average based on the layer thicknesses resulted in a c_h value of 0.142 ft^2/day.

Calculations were performed for a total duration of 6 months (180 days), assumed to be the construction duration for this project. Two fill heights were evaluated, 0 to 7 feet and 7 to 15 feet, as the approach fill height will range from 0 to 15 feet. Vertical wick drain spacings from 4 to 10 feet were evaluated in 1-foot increments to determine the appropriate surcharge heights that would result in the most cost-effective combination of surcharge and wick spacing.

4.1 Geotechnical Data

Geotechnical parameters for the wick drain analysis coincide with the parameters previously presented for the settlement analysis including moisture contents, compression indices, unit weights, and void ratio values. A summary of the design parameters are provided in **Attachment D**. The soil layer thicknesses are based on Boring 14-207M. Each soil layer was evaluated for settlement and time-rate consolidation in 1-foot increments. The unit weight of the embankment fill was 115 pcf based on compaction testing results on Sherack Formation fill.

4.1.1 Groundwater Level

Groundwater was encountered at 11 feet during drilling of the 14-207M boring performed in this region. Based on previous work in the area for the project, the groundwater was assumed in the calculations to be 10 feet below existing ground surface.

4.2 Wick Drain Results

The wick drain design surcharge thickness and drain spacings were based on the additional fill height that will be added to the existing I-29 location ranging from 1 to 15 feet having an embankment crest width of 150 feet and side slopes of 3H:1V. Results of the wick drain design analysis for the I-29 raise near the Wild Rice River structure are described below. Results assumed a triangular wick drain pattern over a construction period of 6 months. Time-rate of consolidation assumes that 99% of primary settlement has occurred, or less than 1 inch of settlement will occur after the end of construction. Two fill heights were evaluated: 0 to 7 feet and 7 to 15 feet. Wick drain lengths are assumed to extend to the top of the till, approximately 73 feet below ground surface.

For an embankment height increase of 0 to 7 feet, various surcharge heights and wick drain spacings were evaluated. If only a surcharge load is to be applied to obtain the desired settlement, a surcharge height of greater than 20 feet would need to be applied for 6 months. To decrease the amount of surcharge required, a 6-foot surcharge with wick drains spaced at 7 feet was chosen for cost comparison purposes. An additional cost optimization can be performed for final design. **Table 9** summarizes the results of the wick drain design.

Table 9. Wick Drain Design Results for a 0 to 7-foot Embankment

Wick Spacing [feet]	Surcharge Height [feet]
10	17.0
9	13.3
8	8.5
7	**6.0**
6	4.0
5	2.3
4	1.5

For an embankment height increase of 7 to 15 feet, various surcharge heights and wick drain spacings were evaluated. If only a surcharge load is to be applied to obtain the desired settlement, a surcharge height of greater than 25 feet would need to be applied for 6 months. To decrease the amount of surcharge required, an 11-foot surcharge with wick drains spaced at 7 feet was chosen for cost comparison purposes. An additional cost optimization can be performed for final design. **Table 10** summarizes the results of the wick drain design.

Table 10. Wick Drain Design Results for a 7 to 15-foot Embankment

Wick Spacing [feet]	Surcharge Height [feet]
10	25.0
9	20.0
8	16.0
7	**11.0**
6	7.0
5	4.0
4	3.0

Based on the preliminary wick design results, the I-29 bridge abutment was modeled with the anticipated 11 feet of surcharge to check slope stability. The results are summarized in **Table 11** indicating that the I-29 embankment will be stable with up to 11 feet of surcharge load for a total I-29 embankment height of 35 feet (10 feet of existing foundation and 15 feet of embankment raise) under drained and undrained conditions.

Table 11. Surcharge Stability Modeling Results

	Factor of Safety Values	
Conditions	I-29 Abutment Raise and Surcharge	Required
ESSA	2.16	1.50
USSA	1.31	1.30

Wick drain design parameters and results are provided in **Attachment D**.

5.0 Recommendations

Analysis 1 evaluated the longitudinal stability of I-29 through the north abutment into the diversion channel incorporating the approach fill and a bridge opening with 7H:1V slopes and a 100-foot bottom. The current bridge opening slopes of 7H:1V resulted in factor of safety values for drained and undrained conditions above the recommended values and therefore suggest the proposed channel dimensions and fill height will be stable under assumed conditions.

Analysis 2 evaluated the existing Wild Rice River channel and stability effects as a result of the elevation changes to the I-29 roadway during construction and staging. Preliminary modeling suggests that the I-29 raise (approximately 250 feet away from the Wild Rice River) appears to have no effect on the Wild Rice River stability. If stability issues are anticipated on the I-29 bridge raise, potential corrective actions include flatter slopes or ground improvement methods.

Analysis 3 evaluated the appropriate offset for the dam from the re-aligned Wild Rice River channel. If a surcharge load (dam) is applied at the top of the channel and there is no offset, the channel and dam appear to be stable in proposed conditions meeting the required drained factor of safety for dams of 1.5 and undrained factor of safety value of 1.3.

Primary and secondary consolidation generally varied in a linear relationship with dam height. Depending on height, the native soils along I-29 and the Wild Rice River could experience primary consolidation settlement ranging from less than one inch up to approximately 11 inches for a taller dam or abutment fill height of 15 feet.

The wick drain design surcharge thickness and drain spacings were based on the additional fill height that will be added to the existing I-29 location ranging from one to 15 feet. For an embankment height increase of 0 to 7 feet, a surcharge load of 6 feet can be applied for 6 months with wick drains spaced at 7 feet. For an embankment height increase of 7 to 15 feet, a surcharge load of 11 feet can be applied for 6 months with wick drains spaced 7 feet apart.

6.0 Limitations of Analysis

The preliminary analysis and conclusions provided are based on the limited dataset available at the time of this analysis. Using generally accepted engineering methods and practices, analyses have been performed using reasonable effort to characterize the site. However, the analyses represent a large area, and variations in stratigraphy, strength, and groundwater conditions may occur.

Kristin Alstadt

July 15, 2014

Kristin N. Alstadt

Date

Certifications

I hereby certify that this memorandum was prepared by me or under my direct supervision and that I am a duly licensed Professional Engineer under the laws of the State of North Dakota.

July 15, 2014

Aaron T. Grosser

Date

PE #: 6221

Figures

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

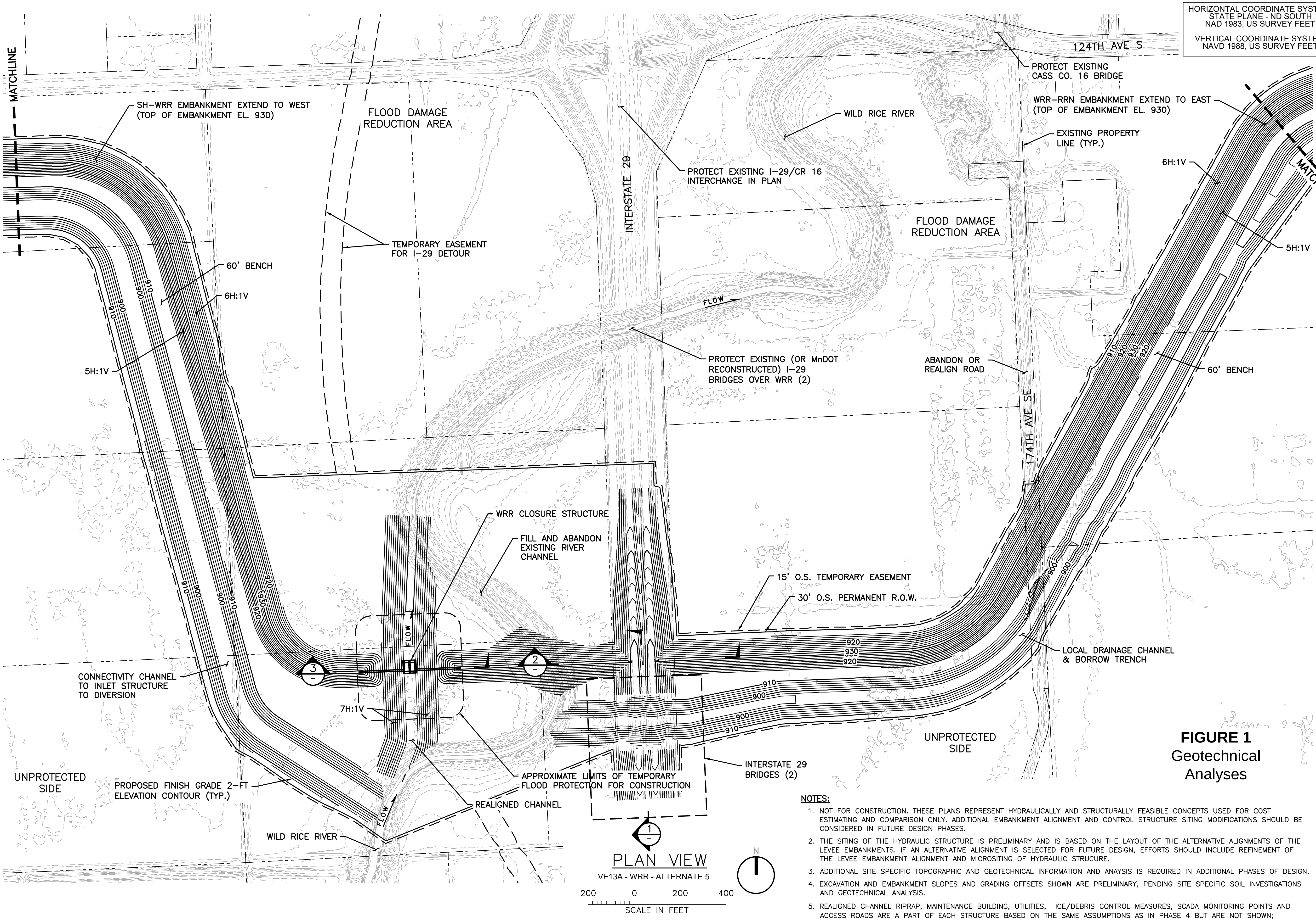


FIGURE 1
Geotechnical
Analyses

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING AND COMPARISON ONLY. ADDITIONAL EMBANKMENT ALIGNMENT AND CONTROL STRUCTURE SITING MODIFICATIONS SHOULD BE CONSIDERED IN FUTURE DESIGN PHASES.
2. THE SITING OF THE HYDRAULIC STRUCTURE IS PRELIMINARY AND IS BASED ON THE LAYOUT OF THE ALTERNATIVE ALIGNMENTS OF THE LEVEE EMBANKMENTS. IF AN ALTERNATIVE ALIGNMENT IS SELECTED FOR FUTURE DESIGN, EFFORTS SHOULD INCLUDE REFINEMENT OF THE LEVEE EMBANKMENT ALIGNMENT AND MICROSITING OF HYDRAULIC STRUCURE.
3. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC AND GEOTECHNICAL INFORMATION AND ANAYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
4. EXCAVATION AND EMBANKMENT SLOPES AND GRADING OFFSETS SHOWN ARE PRELIMINARY, PENDING SITE SPECIFIC SOIL INVESTIGATIONS AND GEOTECHNICAL ANALYSIS.
5. REALIGNED CHANNEL RIPRAP, MAINTENANCE BUILDING, UTILITIES, ICE/DEBRIS CONTROL MEASURES, SCADA MONITORING POINTS AND ACCESS ROADS ARE A PART OF EACH STRUCTURE BASED ON THE SAME ASSUMPTIONS AS IN PHASE 4 BUT ARE NOT SHOWN; QUANTITIES ARE INCLUDED IN COST ESTIMATE AND ARE CONSIDERED COMPARABLE TO PHASE 4 QUANTITIES.

US Army Corps
of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR.
-	CONCEPT - NOT FOR CONSTRUCTION	-	-

MARK	DESCRIPTION	DATE	APPR.

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: -
DWN BY: -
CKD BY: -
DATE: -
SUBMITTED BY: N/A
SOLICITATION NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: -
CONTRACT NO.: N/A
SIZE: ANSI D
FILE NAME: -
FILE NUMBER: N/A

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - VE13 OPTION A
STRUCTURE ON WILD RICE RIVER
PLAN VIEW - GEOTECH ANALYSIS SECTIONS

NORTH DAKOTA DIVERSION

SHEET
IDENTIFICATION
FIGURE 1

CADD USER: Max J. Jouppe FILE: M:\DESIGN\34091004.13\34091004.13_GEOTECH_FIGURE_1.DWG PLOT SCALE: 1:2 PLOT DATE: 5/14/2014 9:32 AM

Figure 2

Sherack Drained Shear Strength @ 15% Strain

Mohr-Coulomb Effective Stress Plot

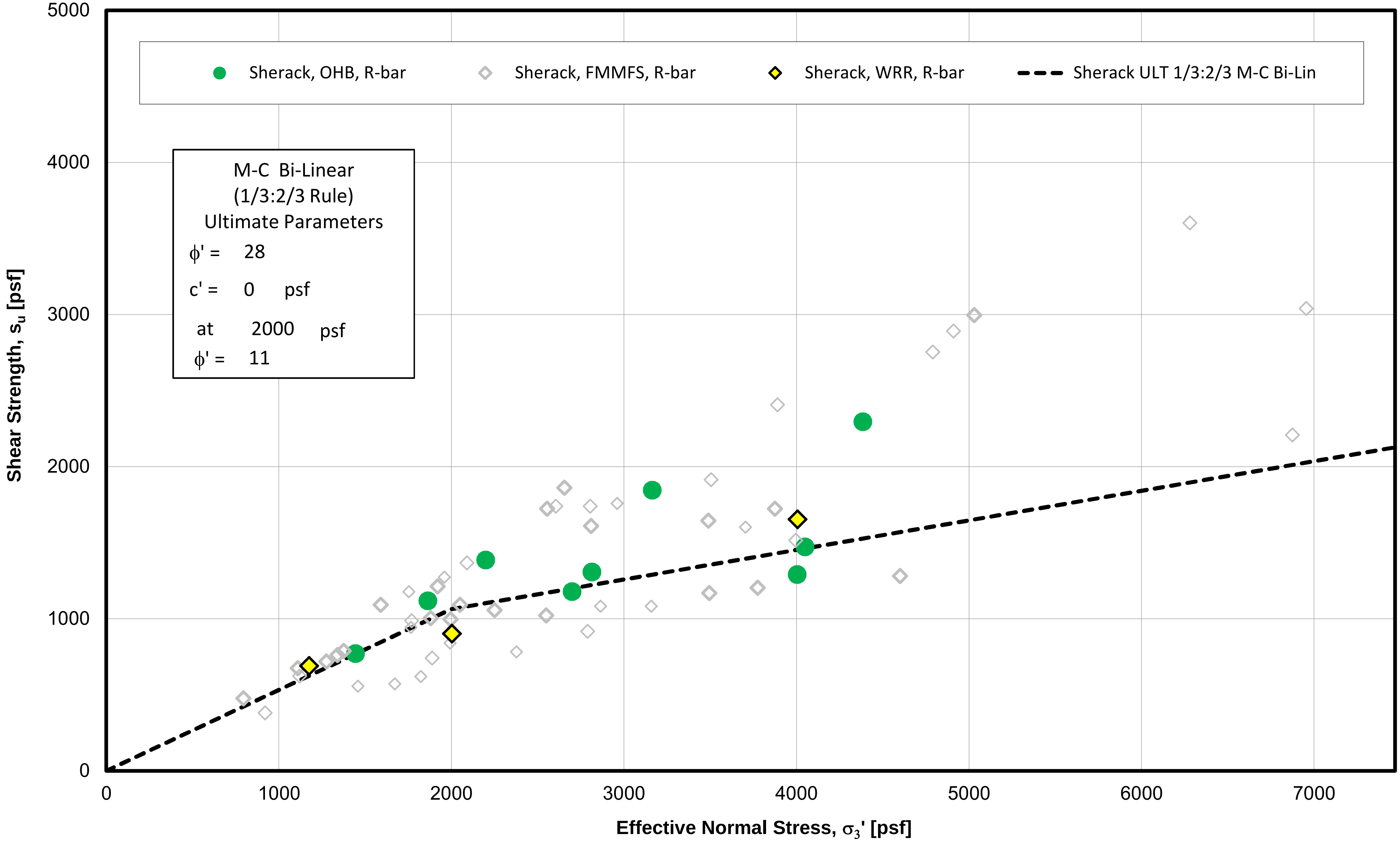


Figure 3

Sherack Drained Shear Strength @ Peak Stress

Mohr-Coulomb Effective Stress Plot

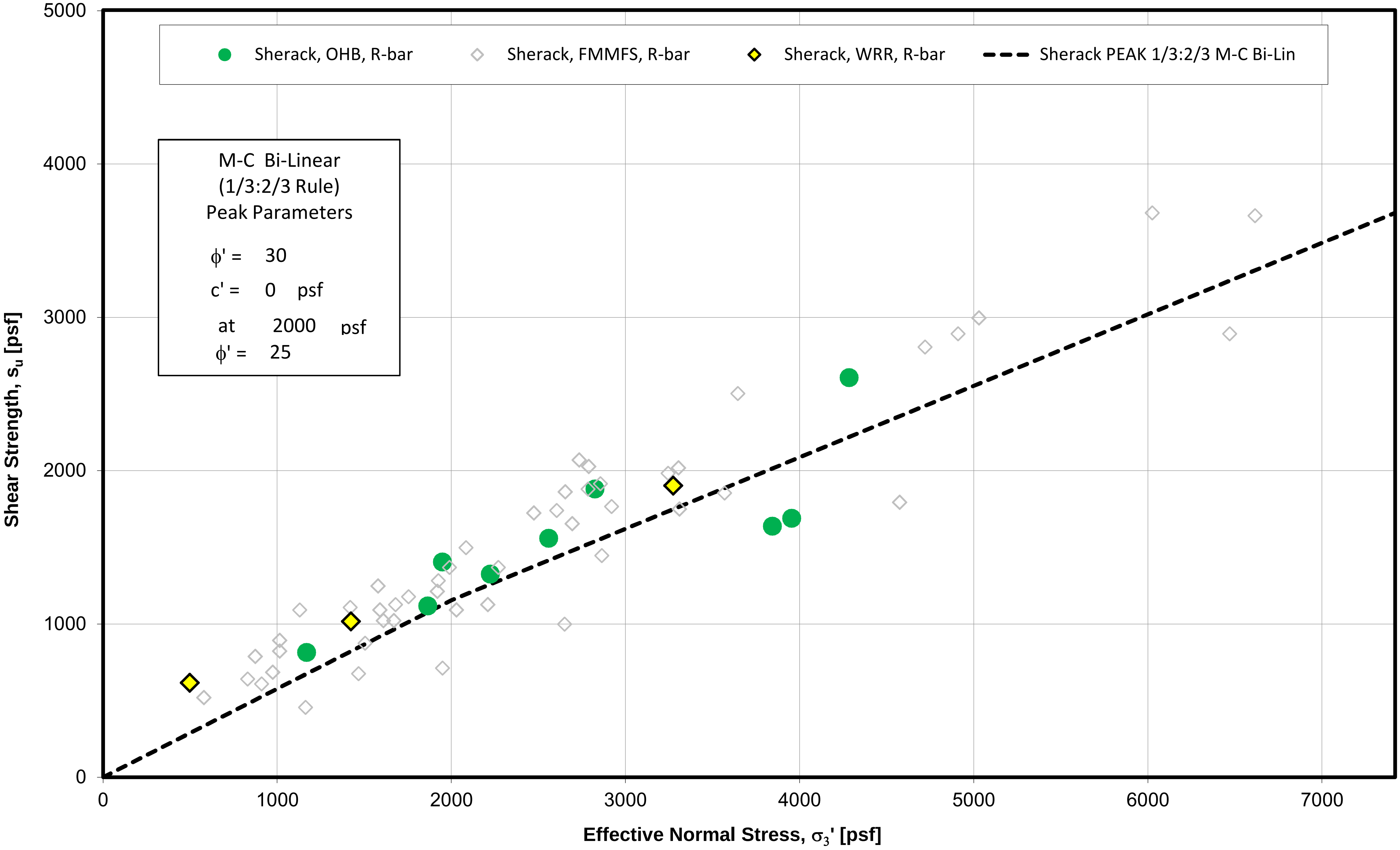
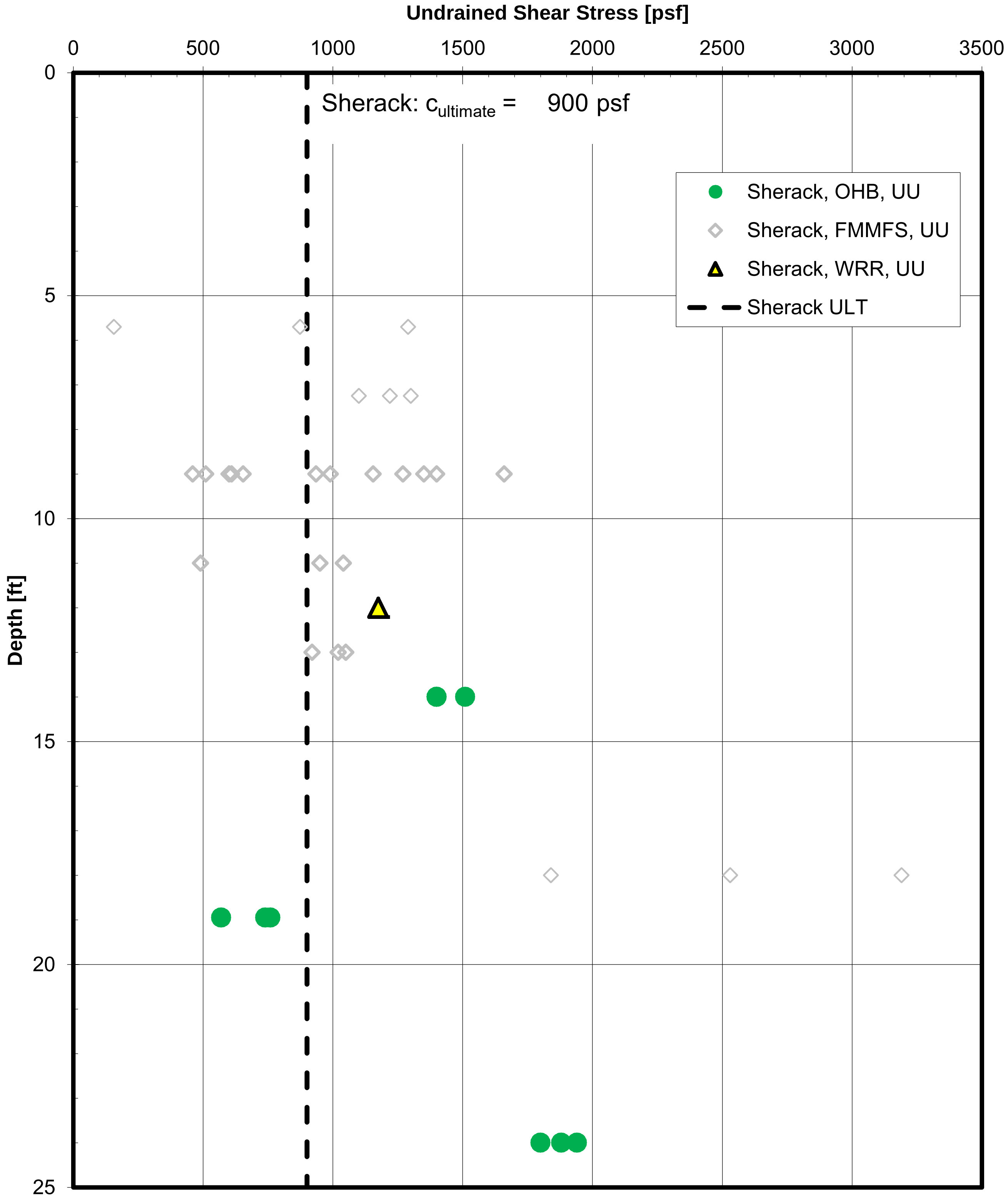


Figure 4

Sherack Formation

Undrained Shear Strength (UU) Data @ 15% Strain



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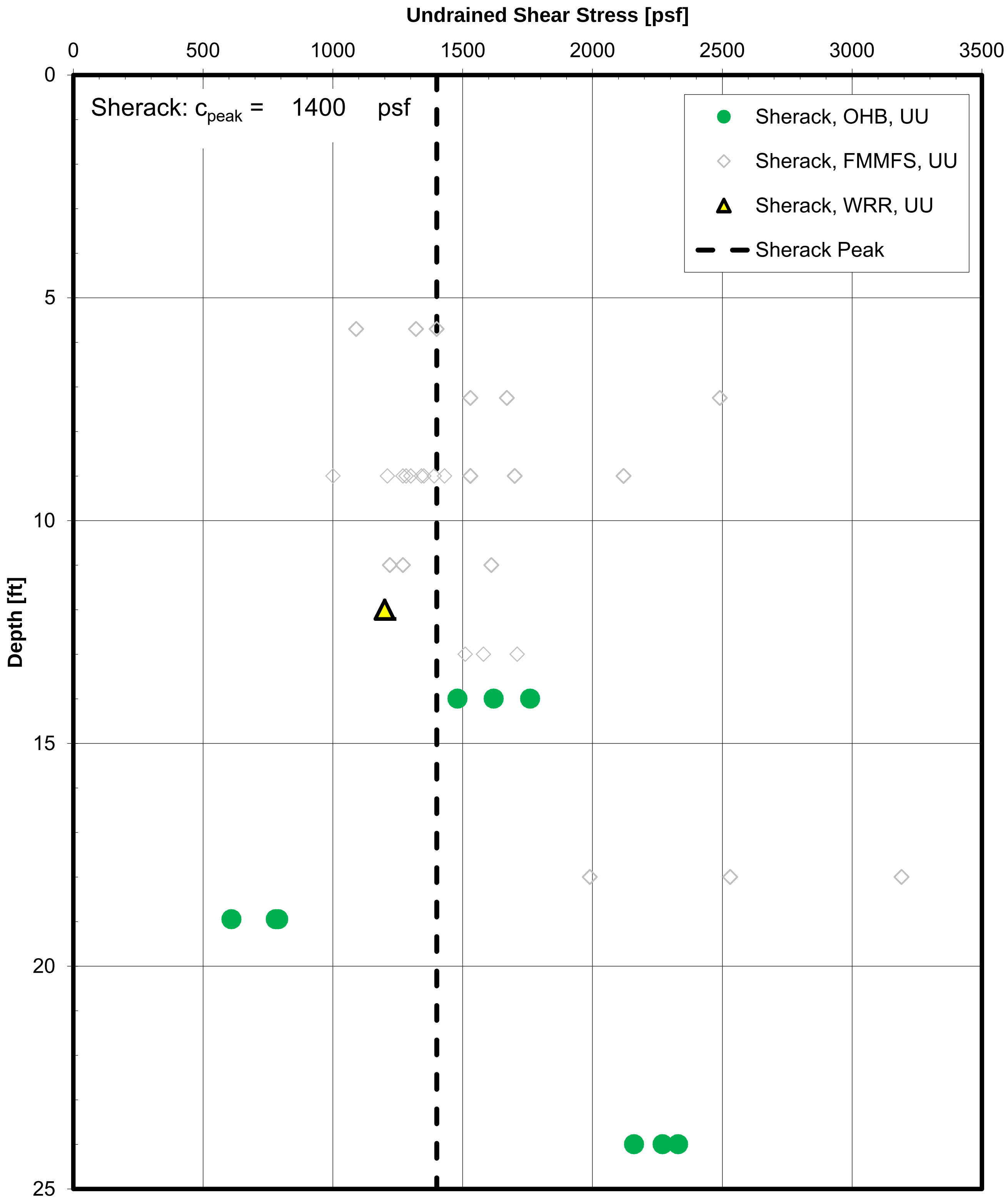
5/16/2014

Figure 5

Sherack Formation

Peak Undrained Shear Strength (UU) Data



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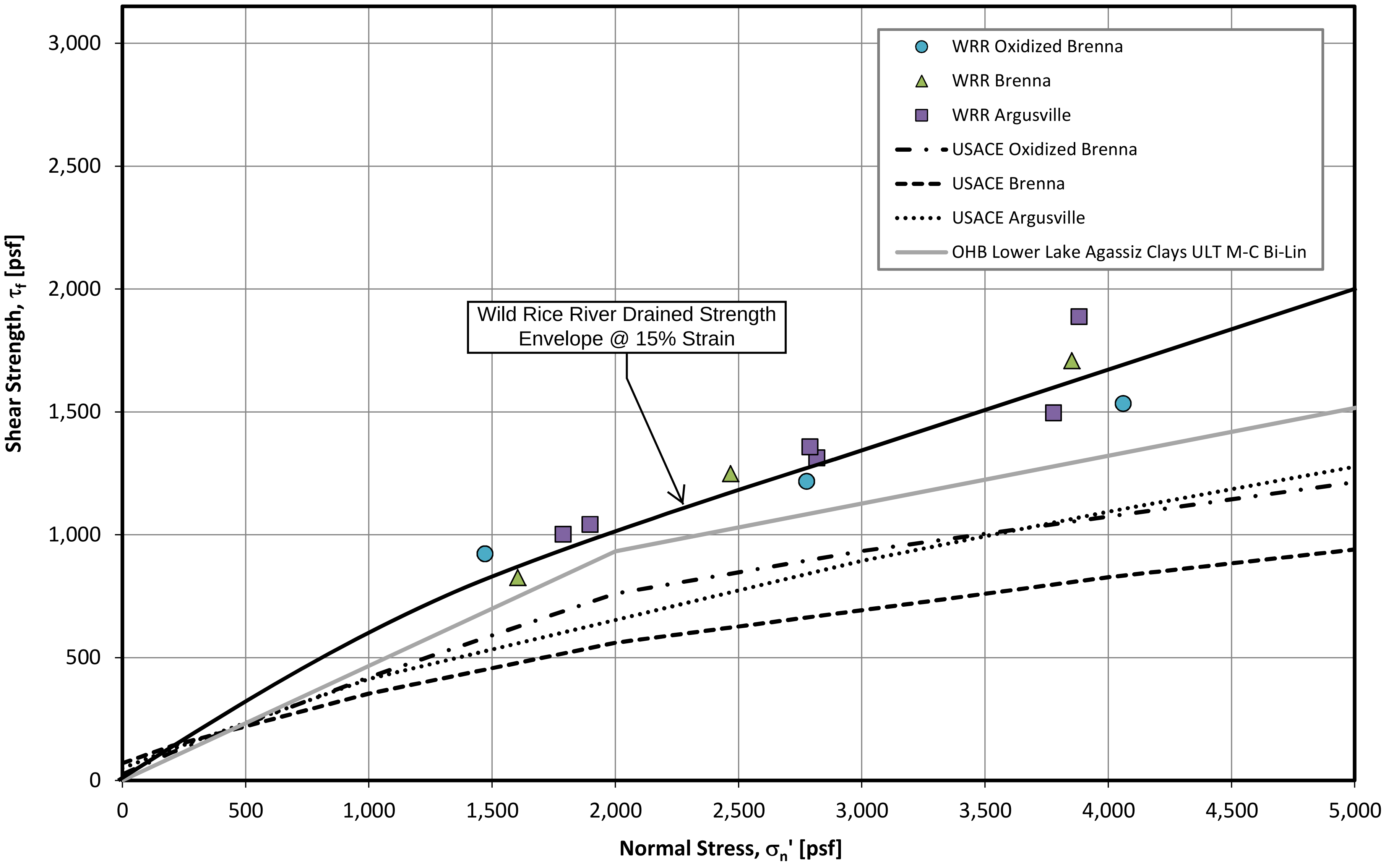
5/14/2014

FIGURE 6

Oxidized Brenna, Brenna, and Argusville Drained Shear Strength @15% Strain

Mohr-Coulomb Effective Stress Plot



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5/30/2014

FIGURE 7

Oxidized Brenna, Brenna, and Argusville Drained Peak Shear Strength

Mohr-Coulomb Effective Stress Plot

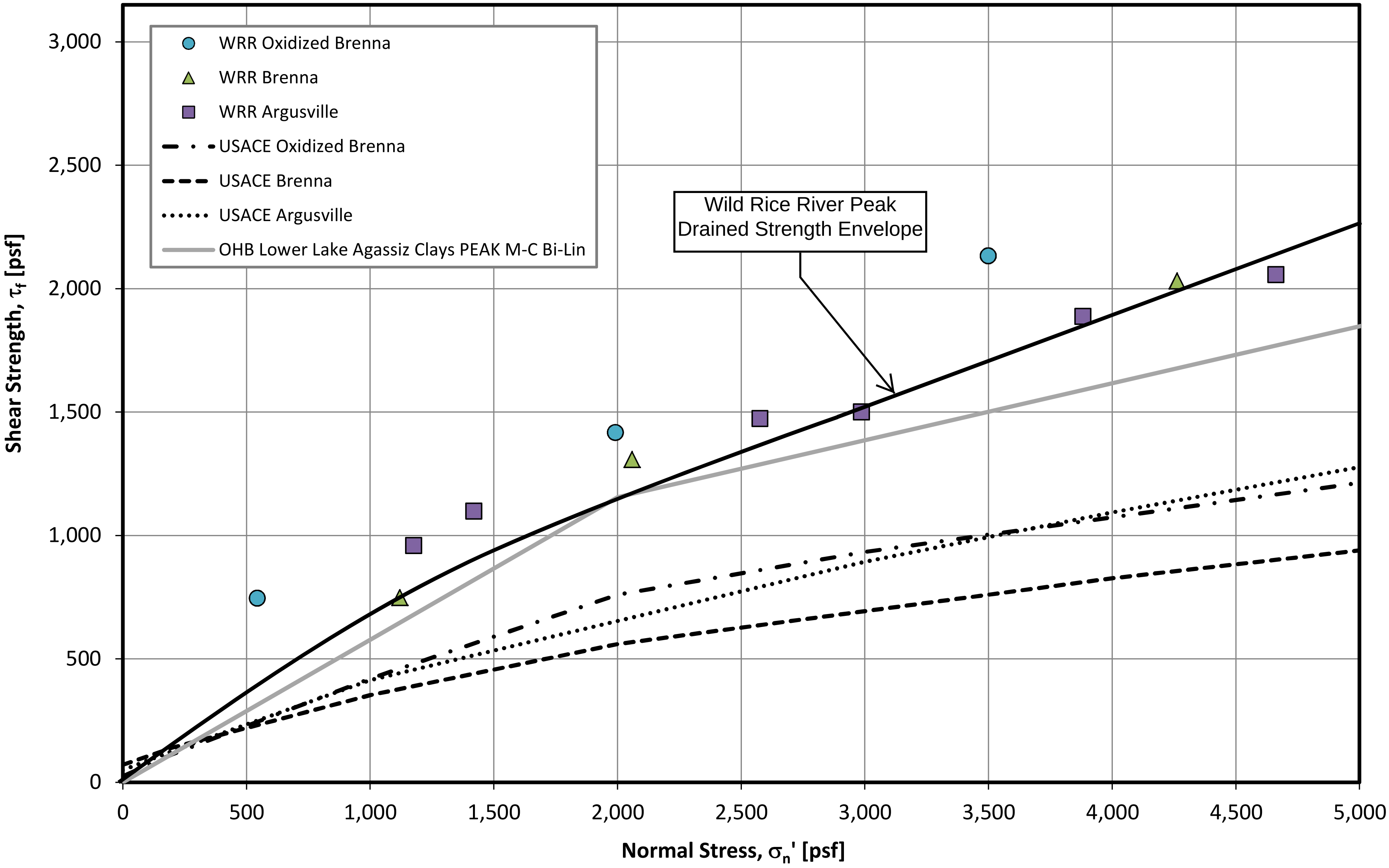
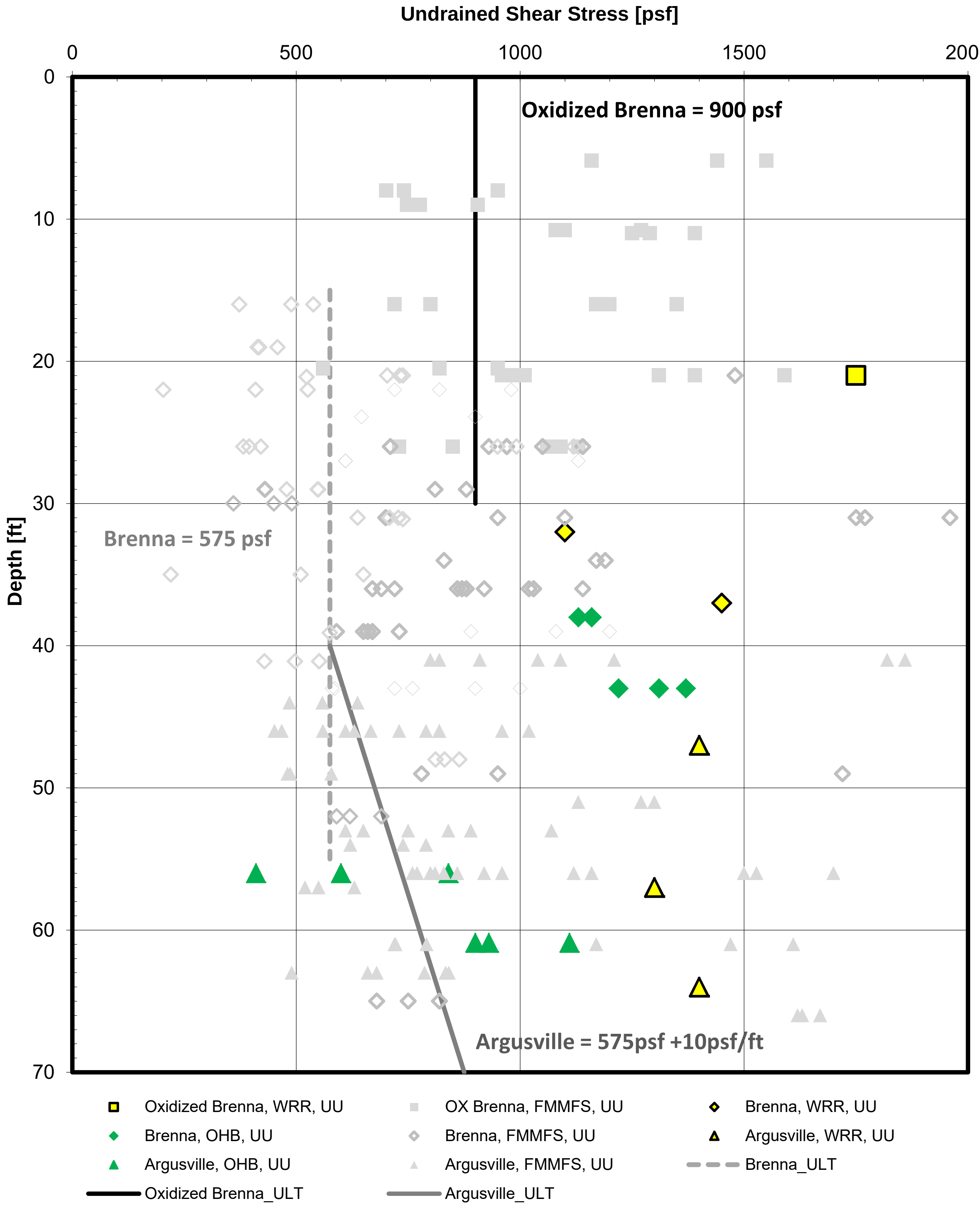


Figure 8

Oxidized Brenna, Brenna, and Argusville

Undrained Shear Strength (UU) Data @ 15% Strain



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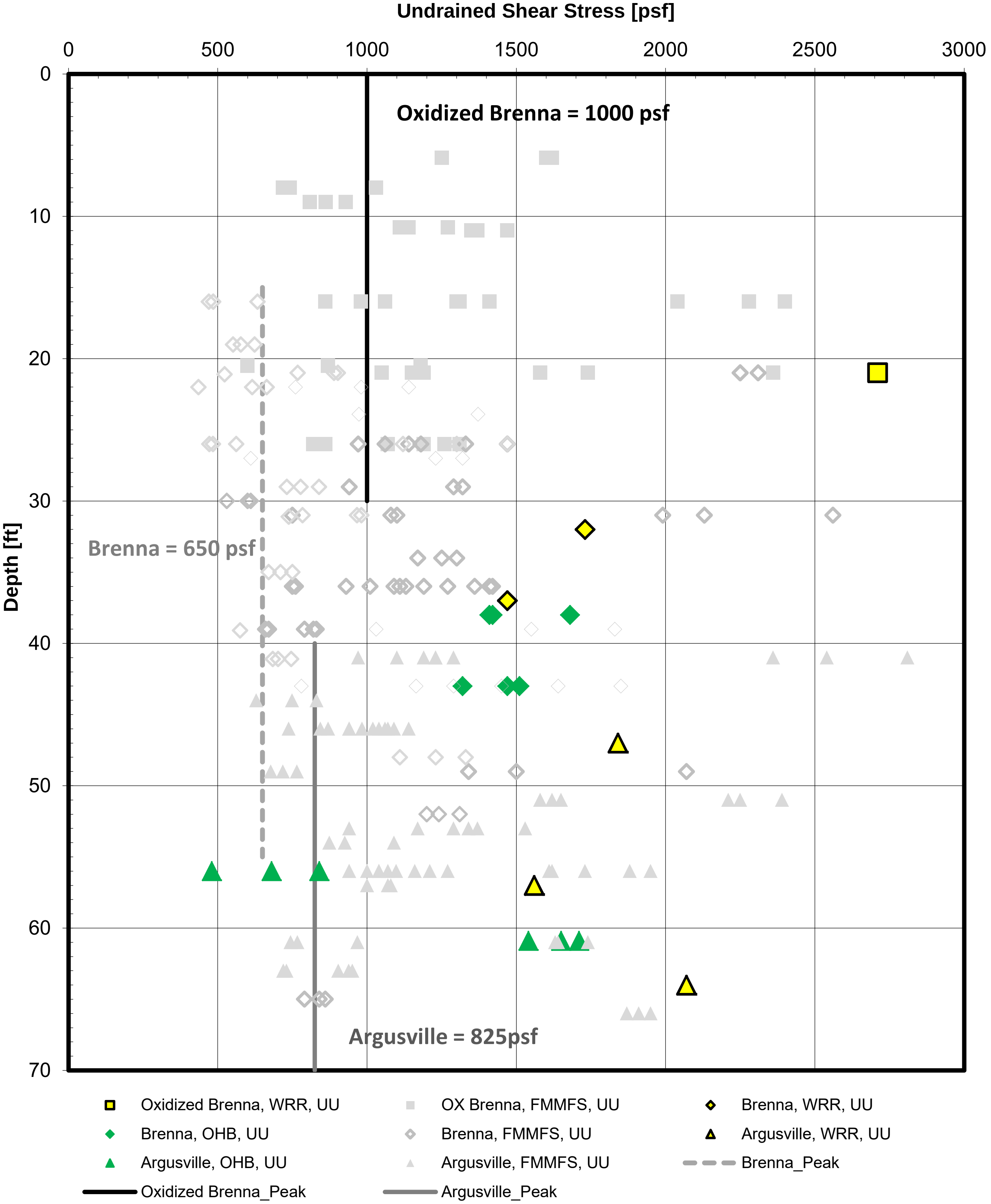
5/30/2014

Figure 9

Oxidized Brenna, Brenna, and Argusville

Peak Undrained Shear Strength (UU) Data



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5/30/2014

Figure 10

Sherack Unit Weights

100% Compaction Based on Standard Proctor Results

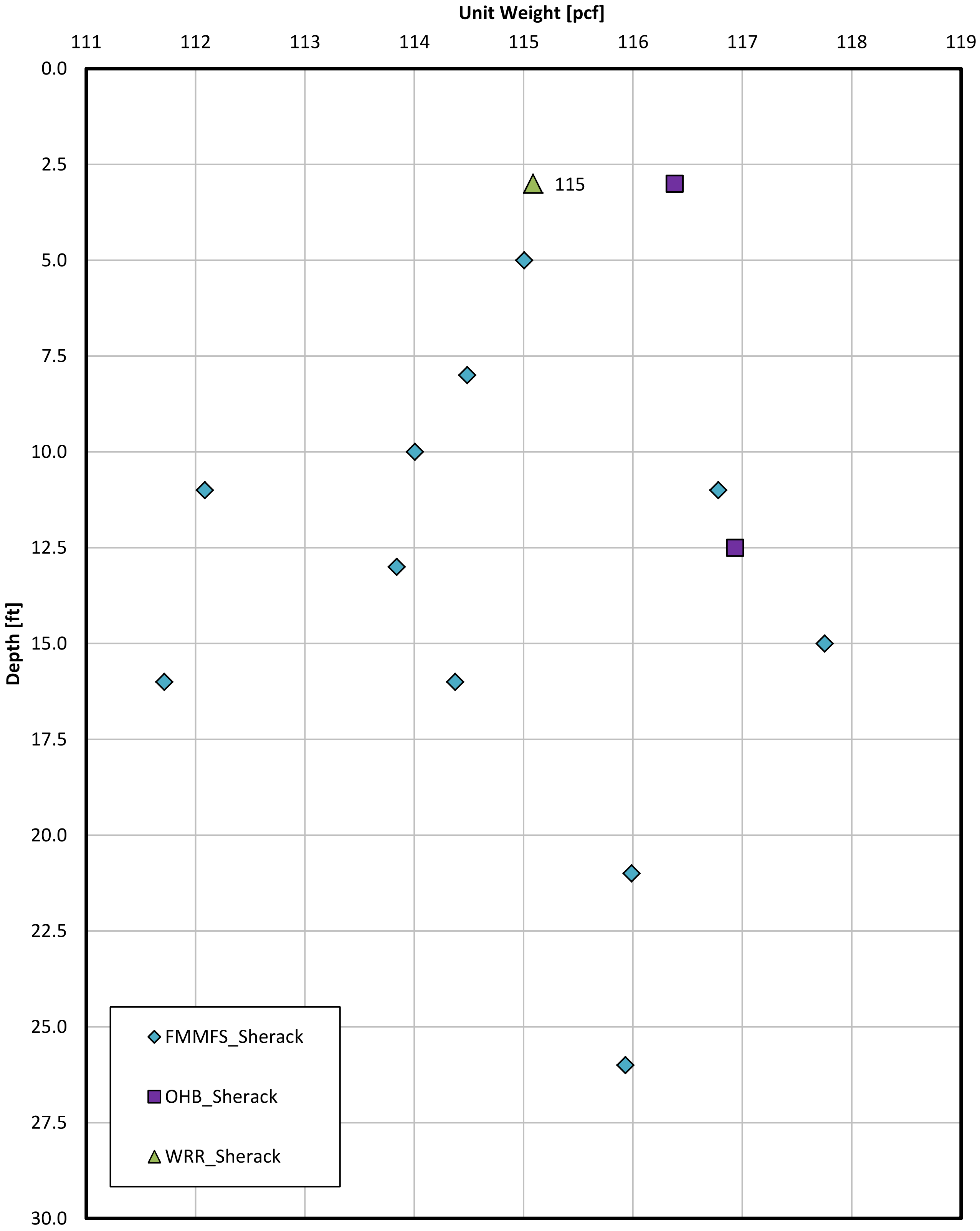
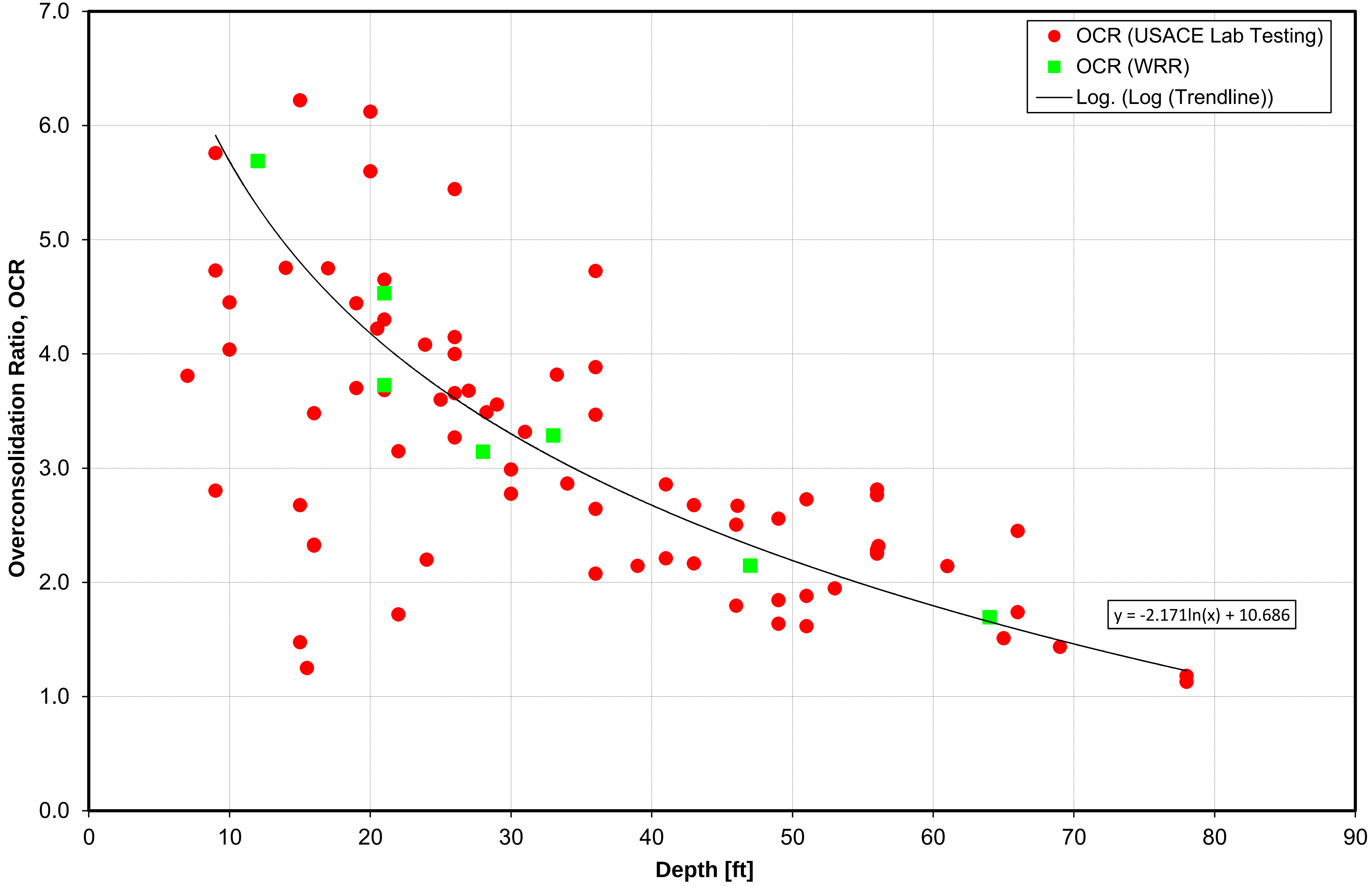


FIGURE 11

Overconsolidation Ratio vs Depth

Fargo-Moorhead Metropolitan Feasibility Study



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Attachment A

Boring Log and Laboratory Testing Results

Barr Engineering Company
4700 West 77th St. Suite 200
Minneapolis, MN 55435
Telephone: 952-832-2600

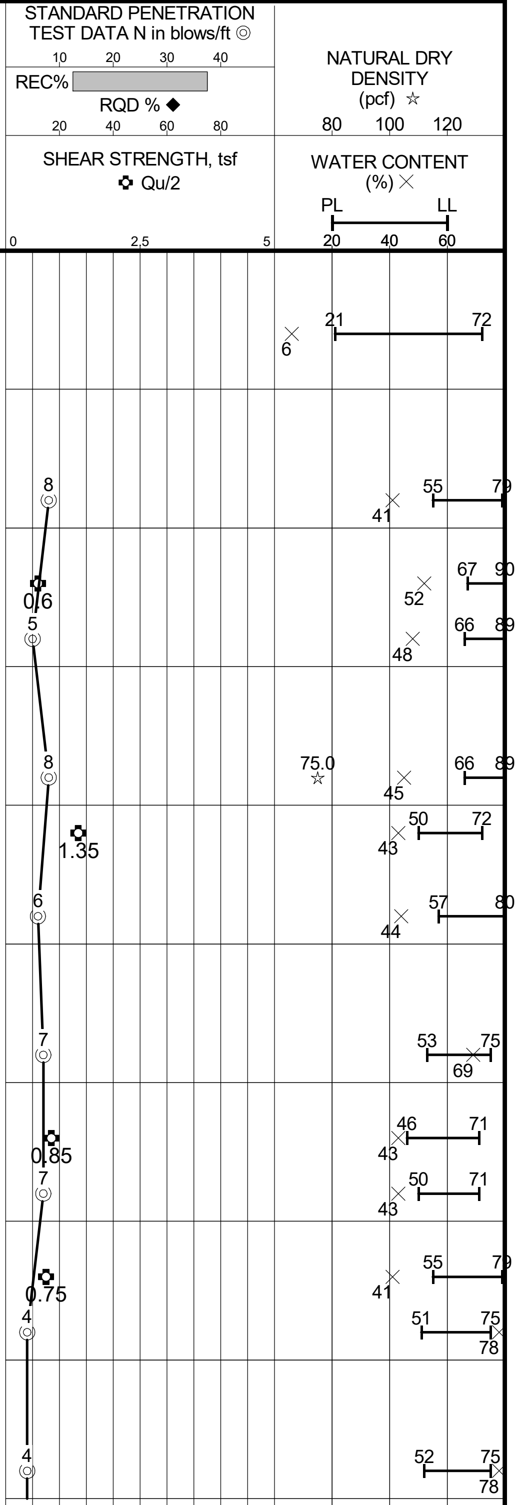
LOG OF BORING 14-207M

Sheet 1 of 2

Project: Wild Rice River Hydraulic Structure
Job No.: 34091004.13, Task Order 14, FY2013
Location: Wild Rice River / I-29 & CR16 Interchange
Coordinates: UTM 14 N:5174755m, E:665699m
Datum: UTM - NAD83

Surface Elevation: 913.8 ft
Drilling Method: HSA
Sampling Method: Split-spoon / 5" Thin-wall
Completion Depth: 86.3 ft

Elevation, feet	Depth, feet	MATERIAL DESCRIPTION	Sample No.	% Recovery	SPT, N value or RQD %
		Surface Elev.: 913.8 ft			
	0	FAT CLAY WITH SILT (CH): yellowish brown; moist; medium stiff to stiff; high plasticity; laminated in zones; 80% clay, 20% silt, trace gypsum; [SHERACK].			
910					
	5				
905			1		8
	10				
▼					
900		898.8 ft	2		5
	15	FAT CLAY (CH): yellowish brown; wet; medium stiff to stiff; high plasticity; laminated in zones; forms "slickensides" in zones; oxidation staining; trace silt; [OXIDIZED BRENNA]. 15.0ft			
895			3		8
	20				
890		888.8 ft	4		6
	25	FAT CLAY (CH): gray/yellow/brown mottling to gray; wet; medium stiff to stiff; high plasticity; laminated in zones; forms "slickensides" in zones; oxidation staining; trace silt becoming siltier with depth; [BRENNA]. 25.0ft			
885			5		7
	30				
880			6		7
	35				
		875.8 ft			
875		FAT CLAY WITH SILT (CH): gray; wet; soft to medium stiff; high plasticity; sticky in zones; gritty texture; silty appearance when sheared; few silt; trace fine gravel; [ARGUSVILLE]. 38.0ft	7		4
	40				
870			8		4
	45				



Continued Next Page

Date Boring Started: 2/17/14
Date Boring Completed: 2/18/14
Logged By: Grant Riddick
Drilling Contractor: Interstate Drilling LLC
Drill Rig: Diedrich D-50

Water Levels (ft)
▼ At Time of Drilling 11.0

Remarks: Coordinates are approximate

Weather:

M:\GINTPROJECTS\WILDRICERIVER-I29 ROAD BORING.GPJ BARRLIBRARY.GLB BOREHOLE LOG REPORT BARR TEMPLATE.GDT

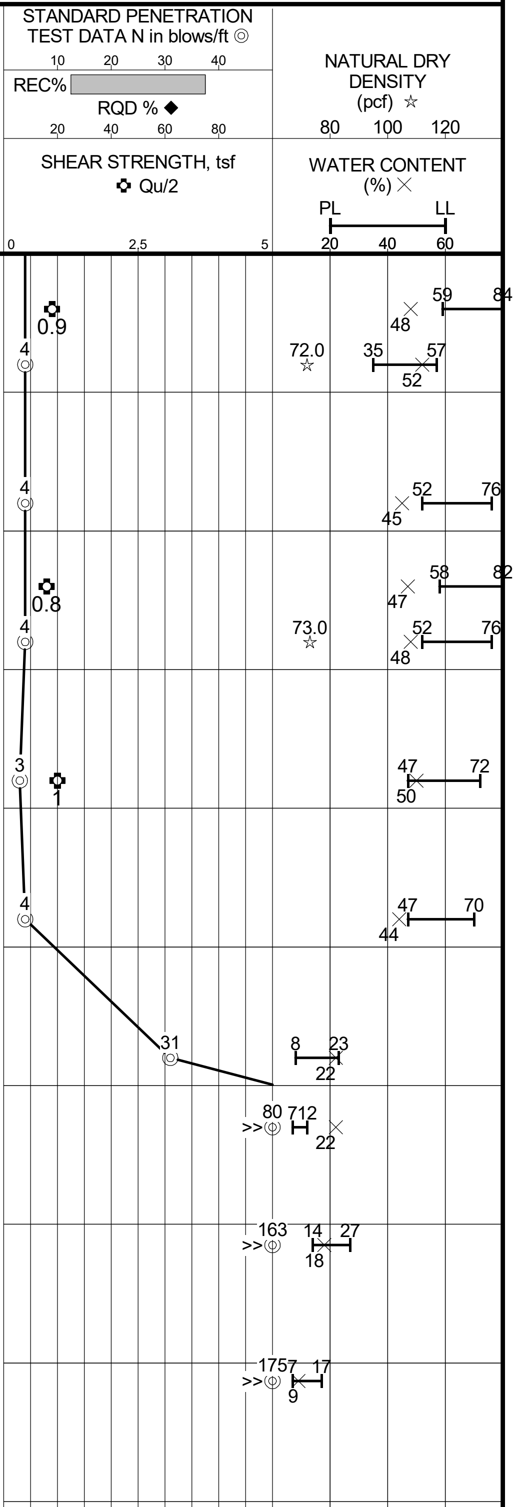
Barr Engineering Company
4700 West 77th St. Suite 200
Minneapolis, MN 55435
Telephone: 952-832-2600

LOG OF BORING 14-207M

Sheet 2 of 2

Project:	Wild Rice River Hydraulic Structure	Surface Elevation:	913.8 ft
Job No.:	34091004.13, Task Order 14, FY2013	Drilling Method:	HSA
Location:	Wild Rice River / I-29 & CR16 Interchange	Sampling Method:	Split-spoon / 5" Thin-wall
Coordinates:	UTM 14 N:5174755m, E:665699m	Completion Depth:	86.3 ft
Datum:	UTM - NAD83		

Elevation, feet	Depth, feet	MATERIAL DESCRIPTION		Graphic Log	Samples	Sample No.	% Recovery	SPT, N value or RQD %
	45	FAT CLAY WITH SILT (CH): gray; wet; soft to medium stiff; high plasticity; sticky in zones; gritty texture; silty appearance when sheared; few silt; trace fine gravel; [ARGUSVILLE]. *(Continued)*						
865						9		4
	50							
860						10		4
	55							
855						11		4
	60							
850						12		3
	65							
845						13		4
	70	840.8 ft						
840		CLAYEY SAND (SC): gray; wet; medium dense; no plasticity; [OUTWASH].	73.0ft			14		31
	75	838.4 ft						
		SILT WITH CLAY AND GRAVEL (ML): gray; moist to wet; dense; little to no plasticity; [UNIT "A" TILL].	75.4ft			15		80
835								
	80					16		163
830								
	85	827.5 ft				17		175
		Bottom of Boring at 86.3 feet	86.3ft					



Date Boring Started:	2/17/14	Water Levels (ft)	
Date Boring Completed:	2/18/14	▽ At Time of Drilling	11.0
Logged By:	Grant Riddick		
Drilling Contractor:	Interstate Drilling LLC		
Drill Rig:	Diedrich D-50		

Remarks: Coordinates are approximate

Weather:

Water Content Test Summary (ASTM:D2216)

Project: Wild Rice River Structure Job: 9288

Client Barr Engineering Company Date: 3/20/2014

Sample Information & Classification

Boring #	14-207M	14-207M	14-207M	14-207M	14-207M	14-207M	14-207M	14-207M
Sample #	1	2	4	5	6	7	8	10
Depth (ft)	8-8.5	13-13.5	23-23.5	28-28.5	33-33.5	38-38.5	43-43.5	53-53.5
Type or BPF	Jar	Jar	Jar	Jar	Jar	Jar	Jar	Jar
Material Classification	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)
Water Content (%)	40.8	47.5	43.8	38.7	42.5	48.3	46.9	46.4

Sample Information & Classification

Boring #	14-207M	14-207M	14-207M	14-207M	14-207M	14-207M		
Sample #	12	13	14	15	16	17		
Depth (ft)	63-63.5	67-67.5	73-73.5	76-76.5	80-80.5	85.5-86		
Type or BPF	Jar	Jar	Jar	Jar	Jar	Jar		
Material Classification	Fat Clay (CH)	Fat Clay (CH)	Clayey Sand (SC)	Lean Clay with sand (CL/CL-ML)	Sandy Lean Clay with gravel (CL)	Sandy Silty Clay (CL-ML/CL)		
Water Content (%)	49.7	44.1	21.8	21.5	18.4	8.7		

Sample Information & Classification

Boring #	14-207MU	14-207MU	14-207MU	14-207MU	14-207MU	14-207MU	14-207MU	
Sample #	1	2	3	4	5	6	7	
Depth (ft)	11-13	20.2-22.2	31-33	36-38	46-48	56-58	63.2-65.2	
Type or BPF	5T	5T	5T	5T	5T	5T	5T	
Material Classification	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	
Water Content (%)	52.0	43.0	42.7	41.4	47.7	46.6	43.7	

Sample Information & Classification

Boring #								
Sample #								
Depth (ft)								
Type or BPF								
Material Classification								
Water Content (%)								

Laboratory Test Summary

Project: **Wild Rice River Structure** **Job:** **9288**

Client: **Barr Engineering Company** **Date:** **3/20/14**

Sample Information & Classification

Boring #	14-207M	14-207M	14-207M				
Sample	3	9	11				
Depth (ft)	18-18.5	48-48.5	58-58.5				
Type or BPF	Jar	Jar	Jar				
Classification	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)				

Water Content, Dry Density

Water Content (%)	44.5	51.5	47.7				
Dry Density (pcf)	75.3	72.1	73.3				

Sample Information & Classification

Boring #							
Sample							
Depth (ft)							
Type or BPF							
Classification							

Water Content, Dry Density

Water Content (%)							
Dry Density (pcf)							

Sample Information & Classification

Boring #							
Sample							
Depth (ft)							
Type or BPF							
Classification							

Water Content, Dry Density

Water Content (%)							
Dry Density (pcf)							

Laboratory Test Summary

Project: Wild Rice River Structure Job: 9287
Client: Barr Engineering Company Date: 3/19/2014

Sample Information & Classification

Boring #	14-207MU	14-207MU	14-207MU	14-207MU	14-207MU	14-207MU	14-207MU	
Sample #	1	2	3	4	5	6	7	
Depth (ft)	11-13	20.2-22.2	31-33	36-38	46-48	56-58	63.2-65.2	
Type or BPF	5T	5T	5T	5T	5T	5T	5T	
Material Classification	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	

Atterberg Limits

Liquid Limit (%)	89.5	71.9	71.4	78.6	84.0	82.0	76.4	
Plastic Limit (%)	22.4	21.8	25.0	23.8	24.8	23.8	23.7	
Plasticity Index (%)	67.1	50.1	46.4	54.8	59.2	58.2	52.7	

Sample Information & Classification

Boring #	14-207M	14-207M	14-207M	14-207M	14-207M	14-207M	14-207M	14-207M
Sample #	1	2	3	4	5	6	7	8
Depth (ft)	8-8.5	13-13.5	18-18.5	23-23.5	28-28.5	33-33.5	38-38.5	43-43.5
Type or BPF	Jar	Jar	Jar	Jar	Jar	Jar	Jar	Jar
Material Classification	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)

Atterberg Limits

Liquid Limit (%)	78.5	88.7	89.3	80.0	75.3	71.3	74.9	75.1
Plastic Limit (%)	23.7	22.6	22.9	23.3	22.2	21.3	23.8	23.3
Plasticity Index (%)	54.8	66.1	66.4	56.7	53.1	50.0	51.1	51.8

Laboratory Test Summary

Project: Wild Rice River Structure Job: 9287

Client: Barr Engineering Company Date: 3/19/2014

Sample Information & Classification

Boring #	14-207M	14-207M	14-207M	14-207M	14-207M	14-207M	14-207M	14-207M
Sample #	9	10	11	12	13	14	15	16
Depth (ft)	48-48.5	53-53.5	58-58.5	63-63.5	67-67.5	73-73.5	76-76.5	80-80.5
Type or BPF	Jar	Jar	Jar	Jar	Jar	Jar	Jar	Jar
Material Classification	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Fat Clay (CH)	Clayey Sand (SC)	Lean Clay with sand (CL/CL-ML)	Sandy Lean Clay with gravel (CL)

Atterberg Limits

Liquid Limit (%)	57.0	76.4	76.2	71.9	70.4	22.9	26.5	27.4
Plastic Limit (%)	21.9	24.5	24.3	25.1	23.2	14.5	19.1	13.3
Plasticity Index (%)	35.1	51.9	51.9	46.8	47.2	8.4	7.4	14.1

Sample Information & Classification

Boring #	14-207M							
Sample #	17							
Depth (ft)	85.5-86							
Type or BPF	Jar							
Material Classification	Sandy Silty Clay with gravel (CL-ML/CL)							

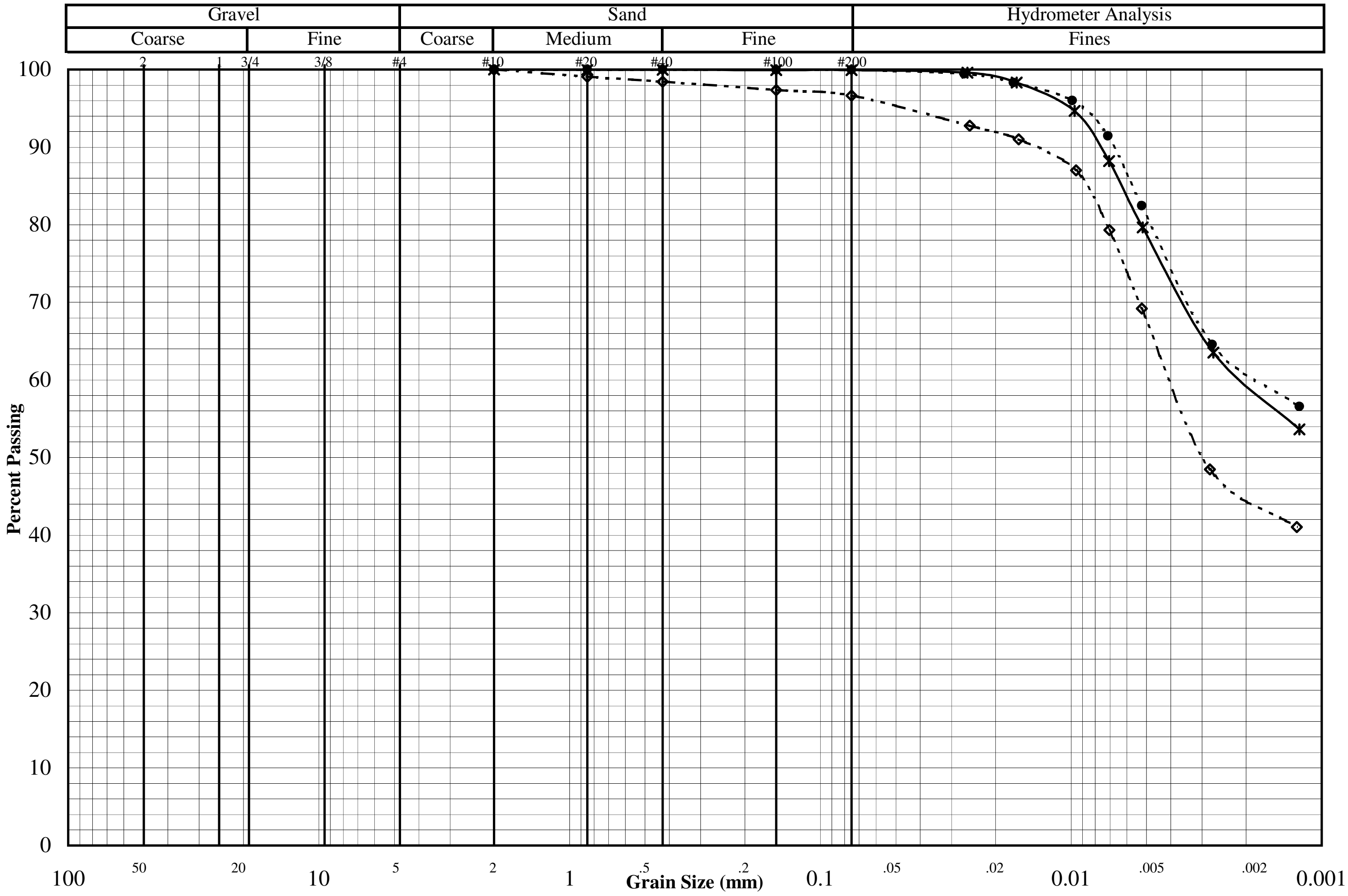
Atterberg Limits

Liquid Limit (%)	17.3							
Plastic Limit (%)	10.8							
Plasticity Index (%)	6.5							

Grain Size Distribution ASTM D422

Project:	Wild Rice River Structure	Job No. :	**9288**
		Test Date:	3/25/14
Reported To:	Barr Engineering Company	Report Date:	3/27/14

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
∗	14-207MU	2	20.2-22.2	5T	Fat Clay (CH)
●	14-207MU	3	31-33	5T	Fat Clay (CH)
◇	14-207MU	5	46-48	5T	Fat Clay (CH)



Other Tests

	∗	●	◇
Liquid Limit			
Plastic Limit			
Plasticity Index			
Water Content			
Dry Density (pcf)			
Specific Gravity	2.73*	2.73*	2.73*
Porosity			
Organic Content			
pH			
Shrinkage Limit			
Penetrometer			
Qu (psf)			

(* = assumed)

Percent Passing

	∗	●	◇
Mass (g)	93.1	99.5	97.1
2"			
1.5"			
1"			
3/4"			
3/8"			
#4			
#10	100.0	100.0	100.0
#20	100.0	100.0	99.1
#40	100.0	100.0	98.4
#100	99.9	100.0	97.4
#200	99.9	100.0	96.6

	∗	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

2401 W 66th Street

SOIL ENGINEERING TESTING, INC.

Richfield, Minnesota 55423-2031

Grain Size Distribution ASTM D422

Job No. : **9288**

Project:	Wild Rice River Structure	Test Date:	3/25/14
Reported To:	Barr Engineering Company	Report Date:	3/27/14

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
Spec 1	14-207MU	2	20.2-22.2	5T	Fat Clay (CH)
Spec 2	14-207MU	3	31-33	5T	Fat Clay (CH)
Spec 3	14-207MU	5	46-48	5T	Fat Clay (CH)

Sieve Data

Specimen 1		Specimen 2		Specimen 3	
Sieve	% Passing	Sieve	% Passing	Sieve	% Passing
2"		2"		2"	
1.5"		1.5"		1.5"	
1"		1"		1"	
3/4"		3/4"		3/4"	
3/8"		3/8"		3/8"	
#4		#4		#4	
#10	100.0	#10	100.0	#10	100.0
#20	100.0	#20	100.0	#20	99.1
#40	100.0	#40	100.0	#40	98.4
#100	99.9	#100	100.0	#100	97.4
#200	99.9	#200	100.0	#200	96.6

Hydrometer Data

Specimen 1		Specimen 2		Specimen 3	
Diameter (mm)	% Passing	Diameter	% Passing	Diameter	% Passing
0.026	99.6	0.027	99.4	0.025	92.7
0.016	98.3	0.017	98.3	0.016	91.0
0.010	94.7	0.010	96.0	0.010	87.0
0.007	88.2	0.007	91.4	0.007	79.3
0.005	79.7	0.005	82.4	0.005	69.2
0.003	63.5	0.003	64.6	0.003	48.5
0.001	53.6	0.001	56.6	0.001	41.0

Remarks

Specimen 1	Specimen 2	Specimen 3

SOIL ENGINEERING TESTING, INC.

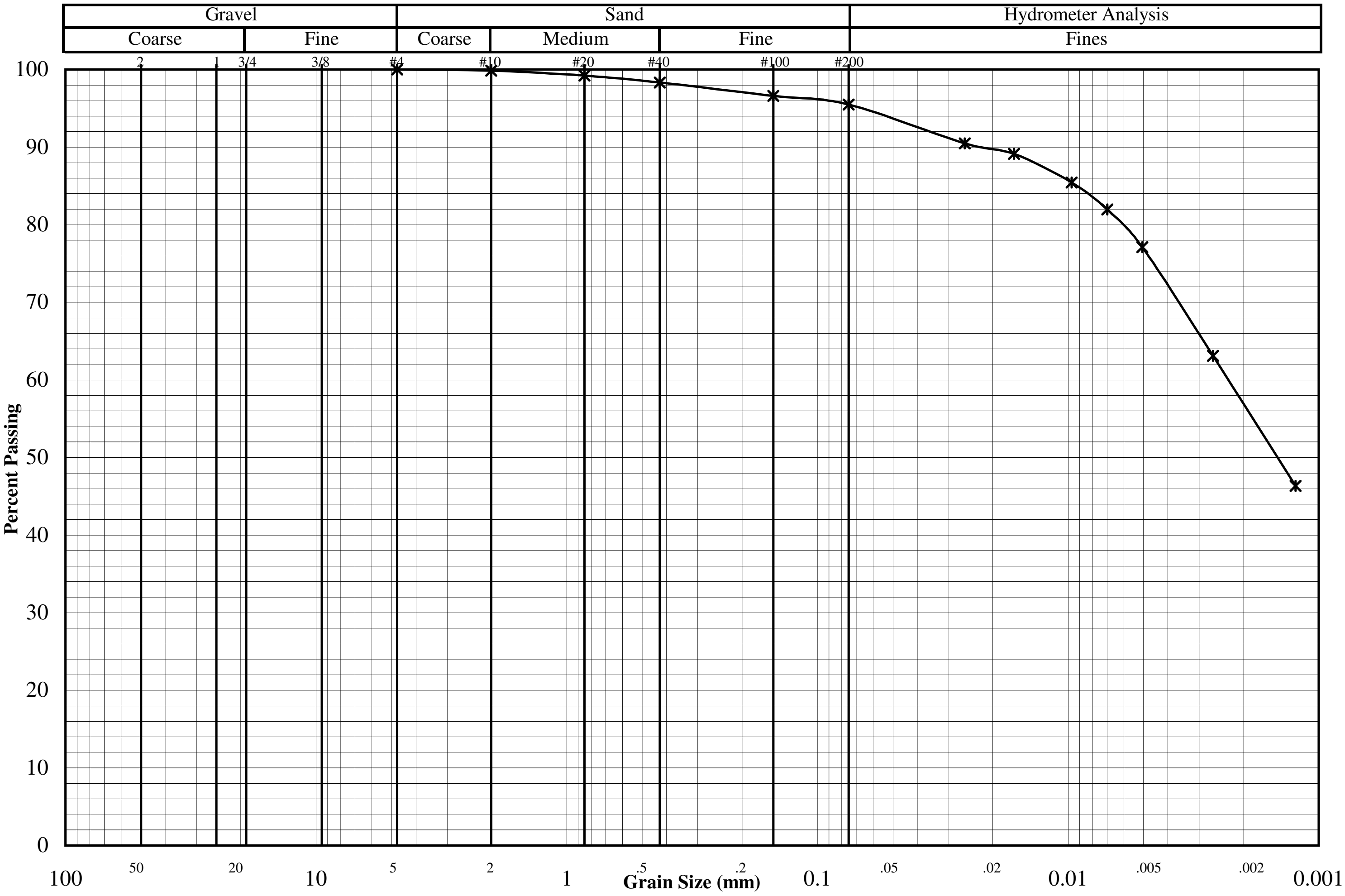
2401 West 66th Street

Richfield, MN 55423

Grain Size Distribution ASTM D422

Project:	Wild Rice River Structure	Job No. :	**9288**
		Test Date:	3/25/14
Reported To:	Barr Engineering Company	Report Date:	3/27/14

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
✱	14-207MU	7	63.2-65.2	5T	Fat Clay (CH)
●					
◇					



Other Tests

	✱	●	◇
Liquid Limit			
Plastic Limit			
Plasticity Index			
Water Content			
Dry Density (pcf)			
Specific Gravity	2.75*		
Porosity			
Organic Content			
pH			
Shrinkage Limit			
Penetrometer			
Qu (psf)			

(* = assumed)

Percent Passing

	✱	●	◇
Mass (g)	91.7		
2"			
1.5"			
1"			
3/4"			
3/8"			
#4	100.0		
#10	99.8		
#20	99.2		
#40	98.3		
#100	96.6		
#200	95.5		

	✱	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

SOIL ENGINEERING TESTING, INC.

2401 W 66th Street

Richfield, Minnesota 55423-2031

Grain Size Distribution ASTM D422

Job No. : **9288**

Project:	Wild Rice River Structure	Test Date:	3/25/14
Reported To:	Barr Engineering Company	Report Date:	3/27/14

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
Spec 1	14-207MU	7	63.2-65.2	5T	Fat Clay (CH)
Spec 2					
Spec 3					

Sieve Data

Specimen 1		Specimen 2		Specimen 3	
Sieve	% Passing	Sieve	% Passing	Sieve	% Passing
2"		2"		2"	
1.5"		1.5"		1.5"	
1"		1"		1"	
3/4"		3/4"		3/4"	
3/8"		3/8"		3/8"	
#4	100.0	#4		#4	
#10	99.8	#10		#10	
#20	99.2	#20		#20	
#40	98.3	#40		#40	
#100	96.6	#100		#100	
#200	95.5	#200		#200	

Hydrometer Data

Specimen 1		Specimen 2		Specimen 3	
Diameter (mm)	% Passing	Diameter	% Passing	Diameter	% Passing
0.026	90.5				
0.016	89.1				
0.010	85.4				
0.007	82.0				
0.005	77.1				
0.003	63.1				
0.001	46.4				

Remarks

Specimen 1	Specimen 2	Specimen 3

SOIL ENGINEERING TESTING, INC.

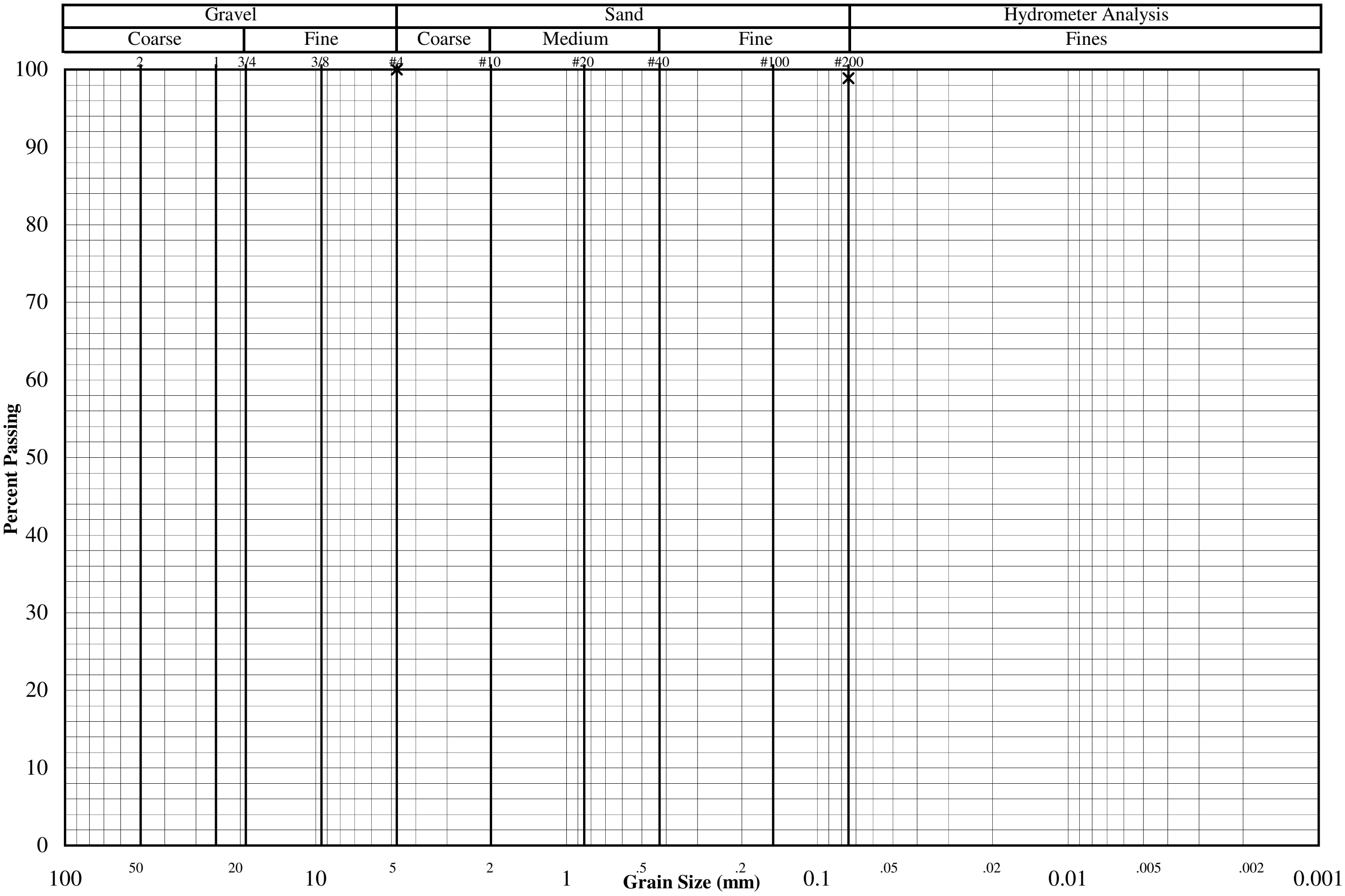
2401 West 66th Street Richfield, MN 55423

Grain Size Distribution ASTM D422

Job No. : **9288-A**

Project:	Wild Rice River Structure	Test Date:	4/23/14
Reported To:	Barr Engineering Company	Report Date:	4/25/14

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
∗	14-207M	Bag 1 of 1	1-5	Bag	Fat Clay (CH)
●					
◇					



Other Tests

	∗	●	◇
Liquid Limit	71.5		
Plastic Limit	20.8		
Plasticity Index	50.7		
Water Content	5.6		
Dry Density (pcf)			
Specific Gravity			
Porosity			
Organic Content			
pH			
Shrinkage Limit			
Penetrometer			
Qu (psf)			

(* = assumed)

Percent Passing

	∗	●	◇
Mass (g)	204.1		
2"			
1.5"			
1"			
3/4"			
3/8"			
#4	100.0		
#10			
#20			
#40			
#100			
#200	98.9		

	∗	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

2401 W 66th Street

SOIL ENGINEERING TESTING, INC.

Richfield, Minnesota 55423-2031

Moisture Density Curve ASTM: D698, Method B

Project: **Wild Rice River Structure** Date: **4/25/14**

Client: **Barr Engineering Company** Job No. **9288-A**

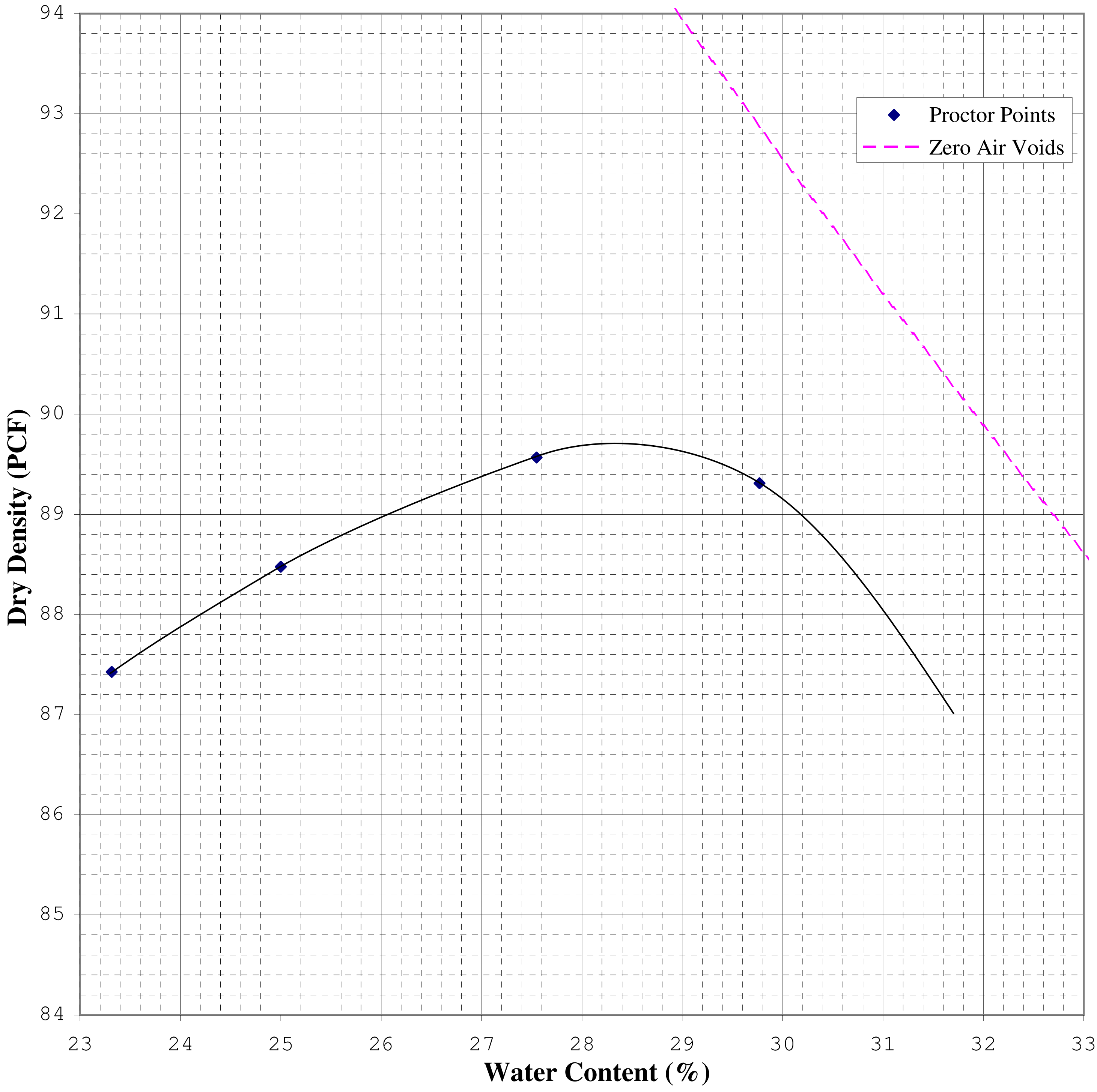
Boring No. **14-207M** Sample: Depth(ft): **1-5** Location:

Soil Type: **Fat Clay (CH)**

As Received W.C. (%): **5.6** LL: **71.5** PL: **20.8** PI: **50.7** Specific Gravity: **2.67** *Assumed

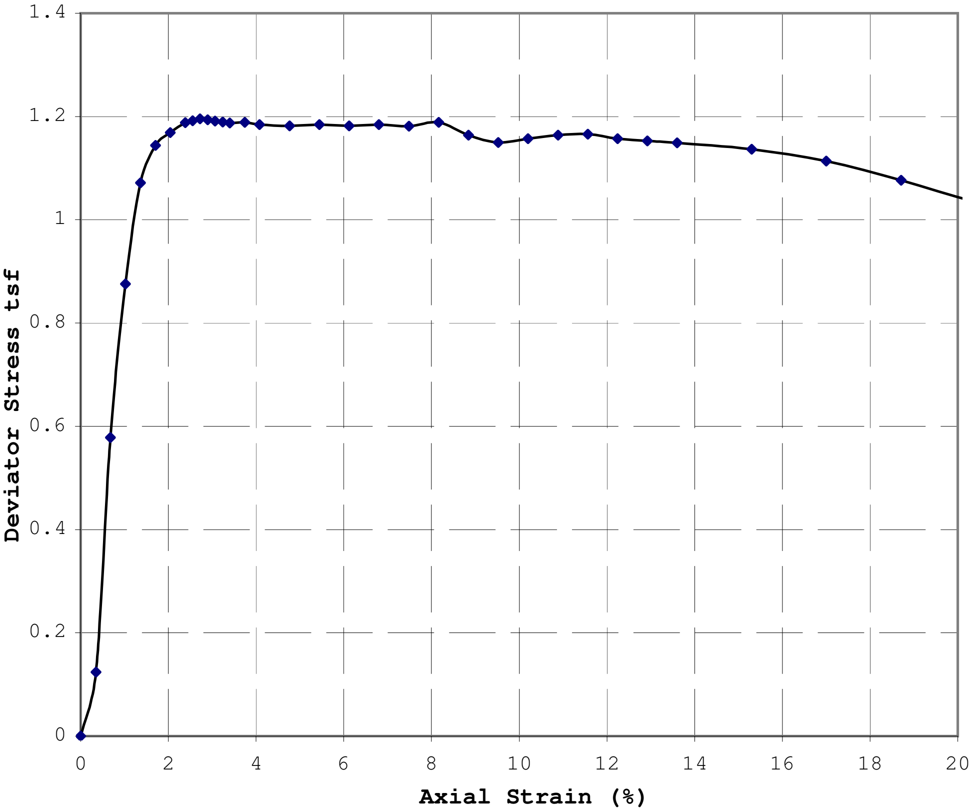
◆ △ ◆ △

Maximum Dry Density (pcf): **89.7** ______ Opt. Water Content (%): **28.3** ______



Triaxial U-U Stress/Strain Curves (ASTM:D2850)

Project: Wild Rice River Structure
Job: 9288
Client: Barr Engineering Company
Date: 4/4/14
Remarks: Specimens trimmed to given sizes; Allowed to adjust under applied confining pressures for about 10 minutes.



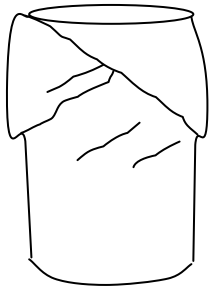
Boring: 14-207MU Depth: 11-13
Sample #: 1
Soil Type: Fat Clay w/laminations of silt (CH)

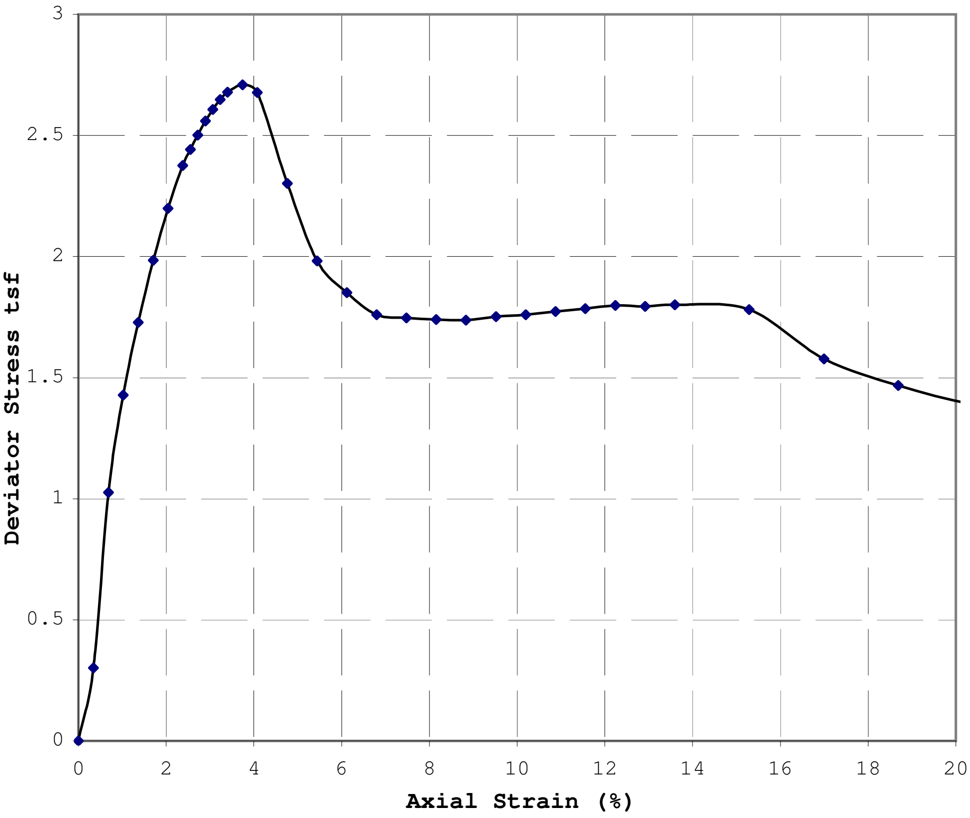
Strain Rate (in/min): 0.030
Sample Type: 5T
Dia. (in) 1.44 Ht. (in) 2.94
Height to Diameter Ratio: 2.04

Max Deviator Stress: 1.20 tsf
Strain at Failure (%): 2.7
Confining Pressure: 0.8 tsf
W.C. (%) 46.4
Yd (pcf): 74.6

LL: 89.5
PL: 22.4
PI: 67.1

Sketch of Specimen After Failure





Boring: 14-207MU Depth: 20.2-22.2
Sample #: 2
Soil Type: Fat Clay (CH)

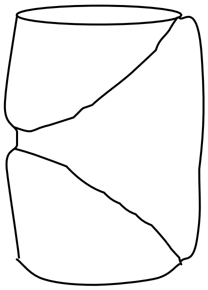
Strain Rate (in/min): 0.030
Sample Type: 5T

Dia. (in): 1.44 Ht. (in): 2.94
Height to Diameter Ratio: 2.04

Max Deviator Stress: 2.71 tsf
Strain at Failure (%): 3.7
Confining Pressure: 1.0 tsf
W.C. (%): 42.7
Yd (pcf): 78.4

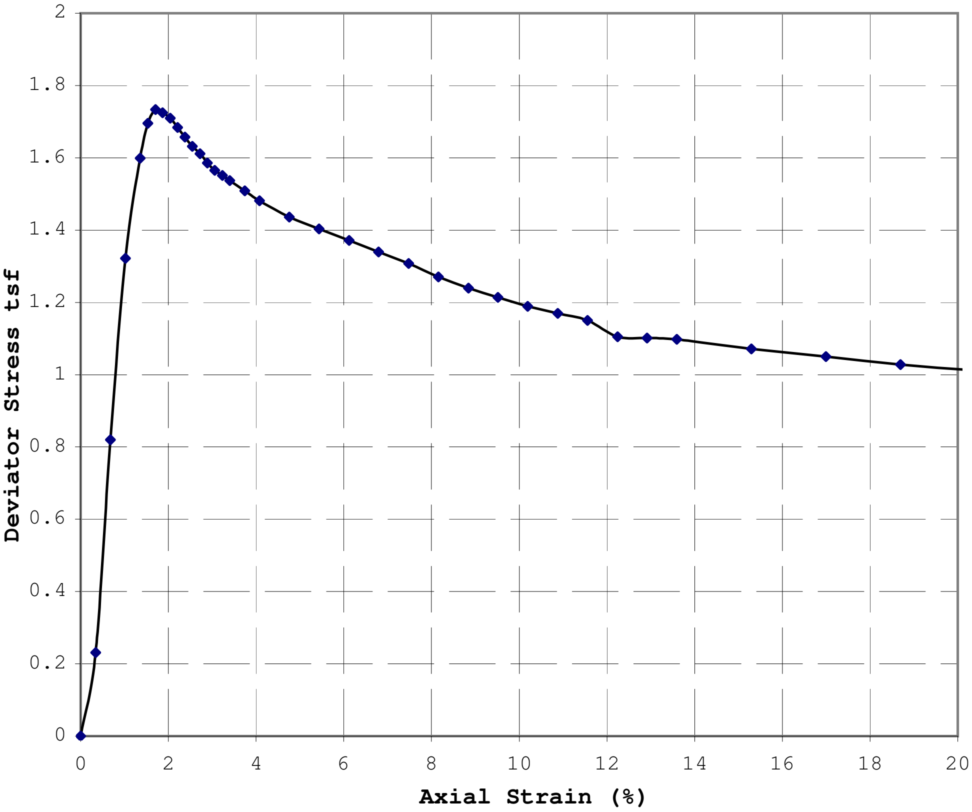
LL: 71.9
PL: 21.8
PI: 50.1

Sketch of Specimen After Failure



Triaxial U-U Stress/Strain Curves (ASTM:D2850)

Project: Wild Rice River Structure
Job: 9288
Client: Barr Engineering Company
Date: 4/4/14
Remarks: Specimens trimmed to given sizes; Allowed to adjust under applied confining pressures for about 10 minutes.



Boring: 14-207MU
Depth: 31-33
Sample #: 3
Soil Type: Fat Clay w/a couple small patches of silt (CH)

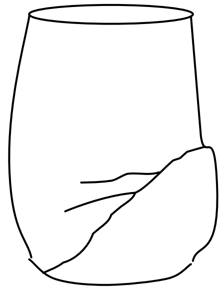
Strain Rate (in/min): 0.030
Sample Type: 5T
Dia. (in) 1.44
Ht. (in) 2.94
Height to Diameter Ratio: 2.04

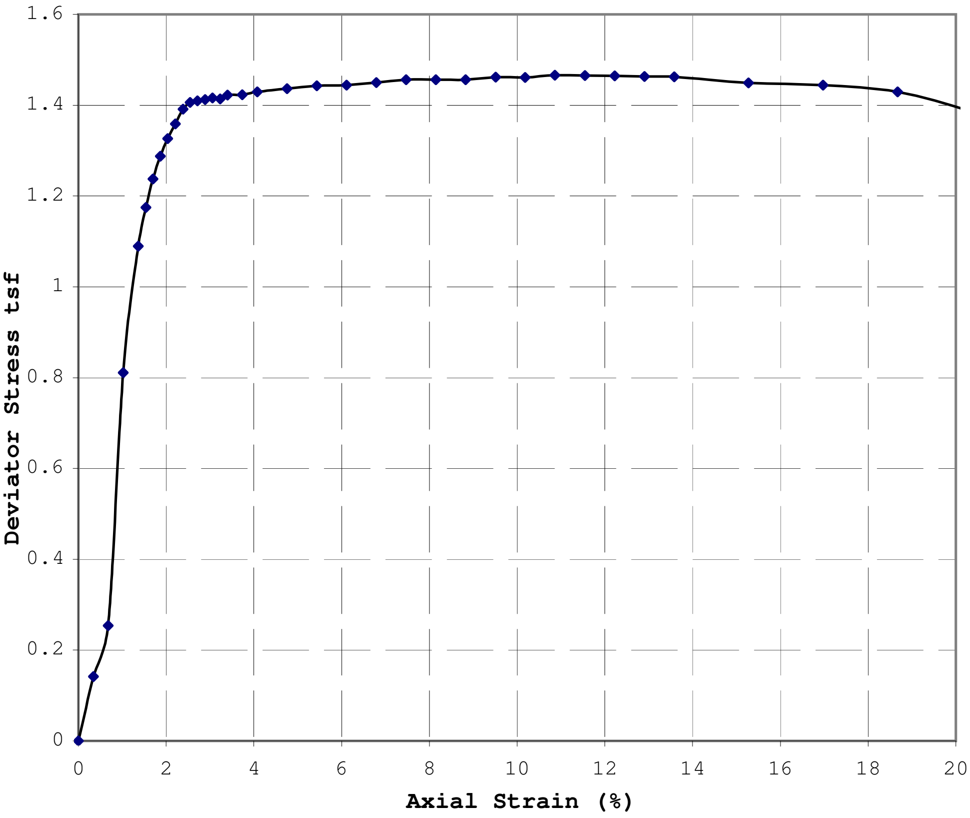
Max Deviator Stress: 1.73 tsf
Strain at Failure (%): 1.7
Confining Pressure: 1.0 tsf

W.C. (%) 45.4
Yd (pcf): 75.6

LL: 71.4
PL: 25
PI: 46.4

Sketch of Specimen After Failure





Boring: 14-207MU
Depth: 36-38
Sample #: 4
Soil Type: Fat Clay w/laminations of silt (CH)

Strain Rate (in/min): 0.030
Sample Type: 5T

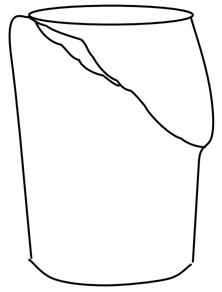
Dia. (in): 1.44
Ht. (in): 2.95
Height to Diameter Ratio: 2.04

Max Deviator Stress: 1.47 tsf
Strain at Failure (%): 10.9
Confining Pressure: 1.3 tsf

W.C. (%): 40.6
Yd (pcf): 80.2

LL: 78.6
PL: 23.8
PI: 54.8

Sketch of Specimen After Failure

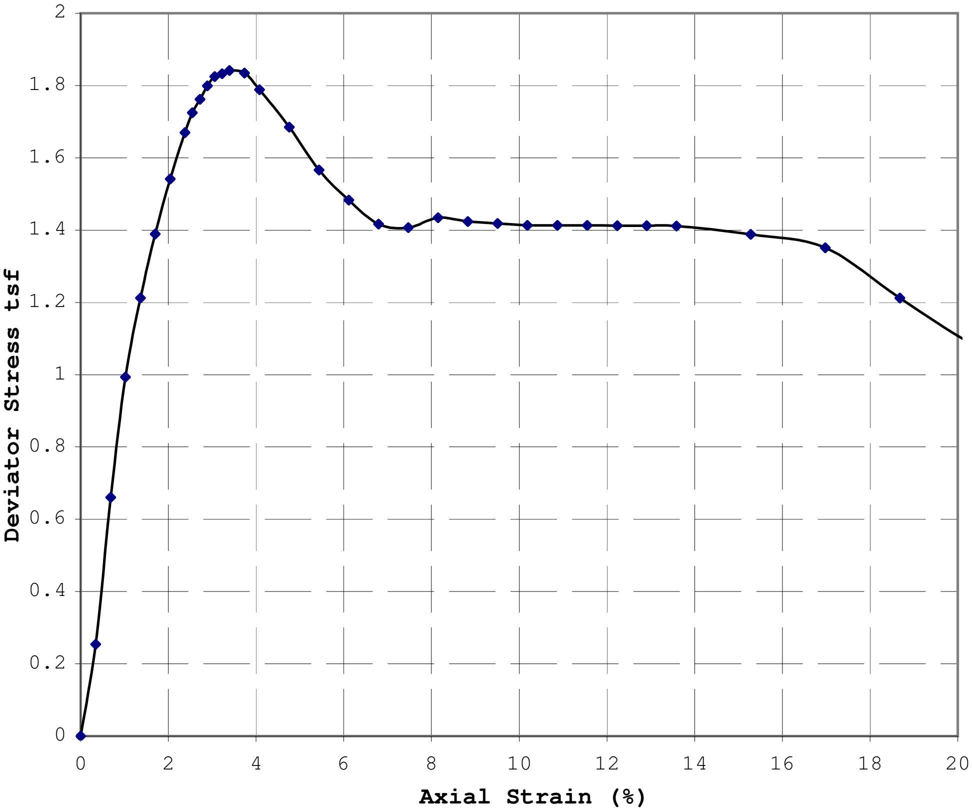


SOIL ENGINEERING TESTING, INC.

2401 West 66th Street
Richfield, MN 55423

Triaxial U-U Stress/Strain Curves (ASTM:D2850)

Project: Wild Rice River Structure Job: 9288
Client: Barr Engineering Company Date: 4/4/14
Remarks: Specimens trimmed to given sizes; Allowed to adjust under applied confining pressures for about 10 minutes.



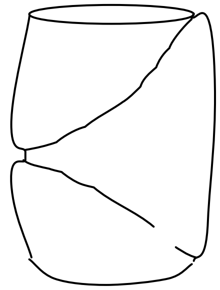
Boring: 14-207MU Depth: 46-48
Sample #: 5
Soil Type: Fat Clay (CH)

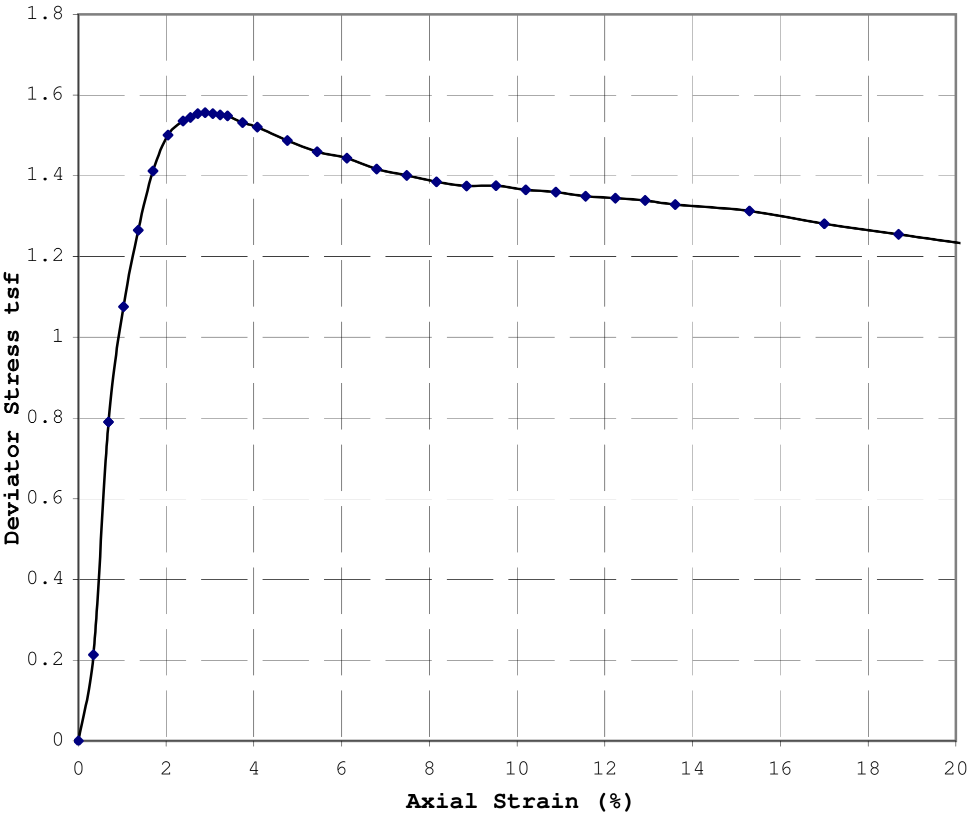
Strain Rate (in/min): 0.030
Sample Type: 5T
Dia. (in) 1.44 Ht. (in) 2.95
Height to Diameter Ratio: 2.04

Max Deviator Stress: 1.84 tsf
Strain at Failure (%): 3.4
Confining Pressure: 1.5 tsf
W.C. (%) 44.9
Yd (pcf): 75.6

LL: 84
PL: 24.8
PI: 59.2

Sketch of Specimen After Failure





Boring: 14-207MU Depth: 56-58
Sample #: 6
Soil Type: Fat Clay w/a trace of gravel (CH)

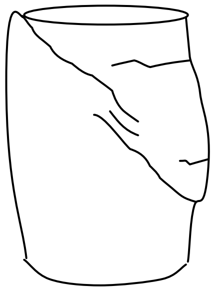
Strain Rate (in/min): 0.030
Sample Type: 5T

Dia. (in): 1.44 Ht. (in): 2.94
Height to Diameter Ratio: 2.04

Max Deviator Stress: 1.56 tsf
Strain at Failure (%): 2.9
Confining Pressure: 1.8 tsf
W.C. (%): 45.1
Yd (pcf): 75.6

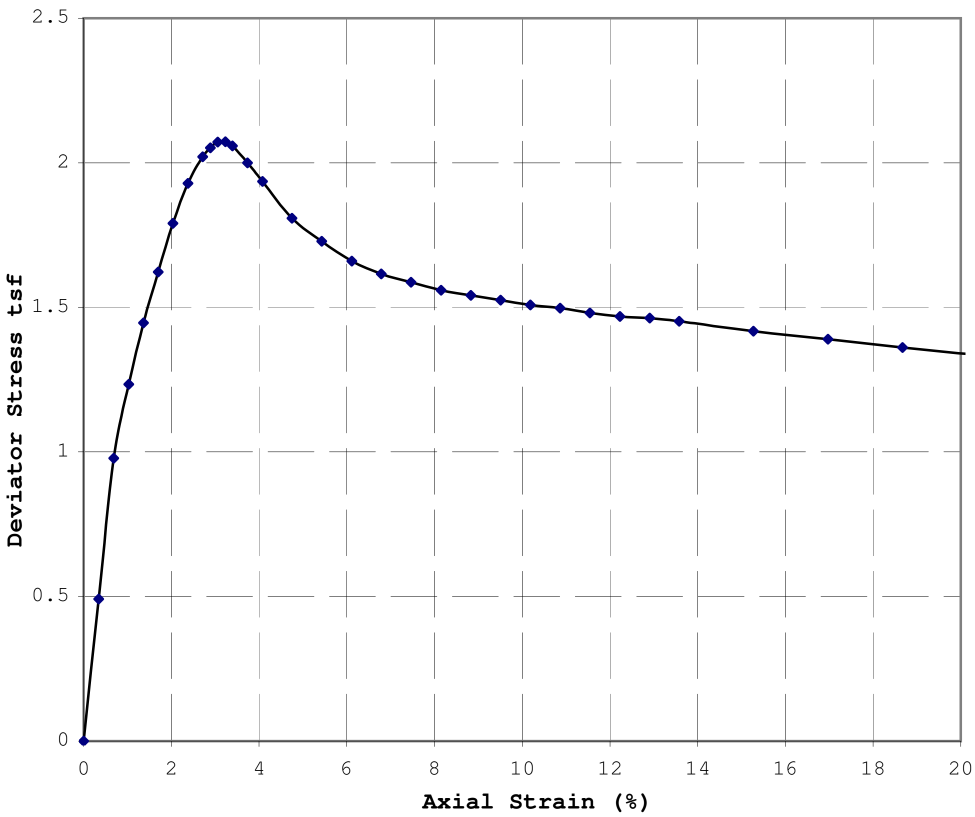
LL: 82
PL: 23.8
PI: 58.2

Sketch of Specimen After Failure



Triaxial U-U Stress/Strain Curves (ASTM:D2850)

Project: Wild Rice River Structure
Job: 9288
Client: Barr Engineering Company
Date: 4/4/14
Remarks: Specimens trimmed to given sizes; Allowed to adjust under applied confining pressures for about 10 minutes.



Boring: 14-207MU
Depth: 63.2-65.2
Sample #: 7
Soil Type: Fat Clay (CH)

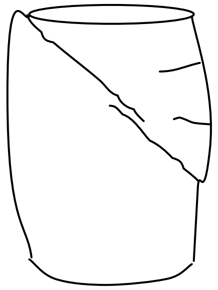
Strain Rate (in/min): 0.030
Sample Type: 5T
Dia. (in) 1.44
Ht. (in) 2.95
Height to Diameter Ratio: 2.04

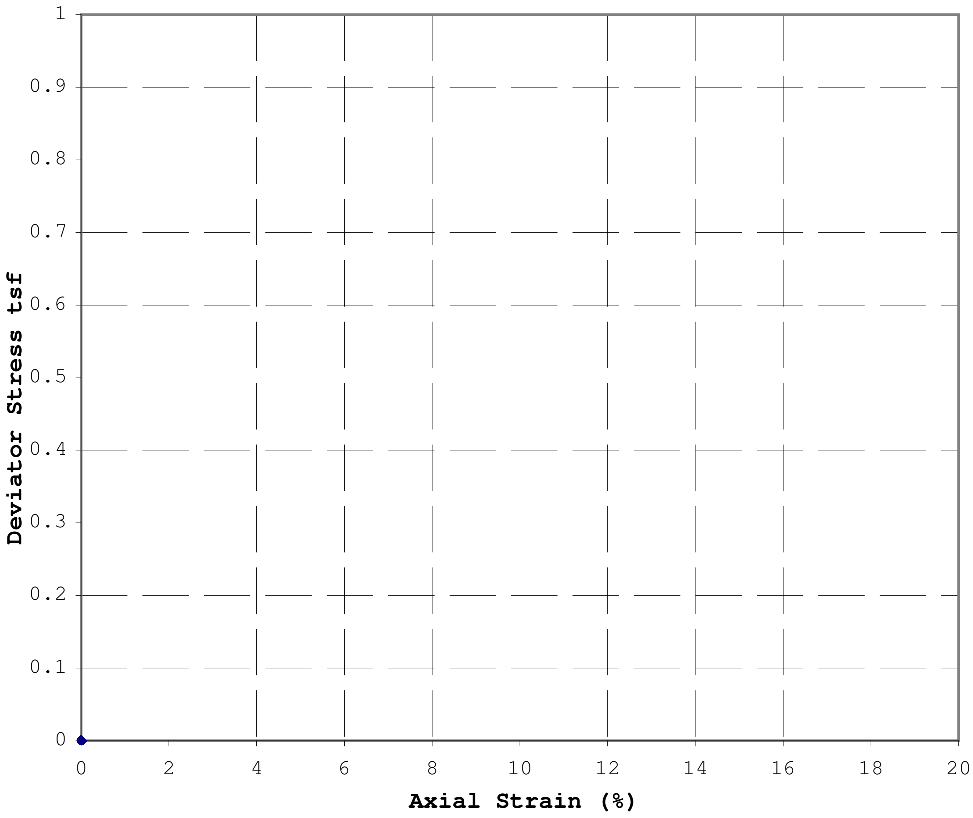
Max Deviator Stress: 2.07 tsf
Strain at Failure (%): 3.2
Confining Pressure: 2.0 tsf

W.C. (%) 45.0
Yd (pcf): 75.1

LL: 76.4
PL: 23.7
PI: 52.7

Sketch of Specimen After Failure





Boring:
Depth:
Sample #:
Soil Type:

Strain Rate (in/min):
Sample Type:

Dia. (in):
Ht. (in):
Height to Diameter Ratio:

Max Deviator Stress: tsf
Strain at Failure (%):
Confining Pressure: tsf

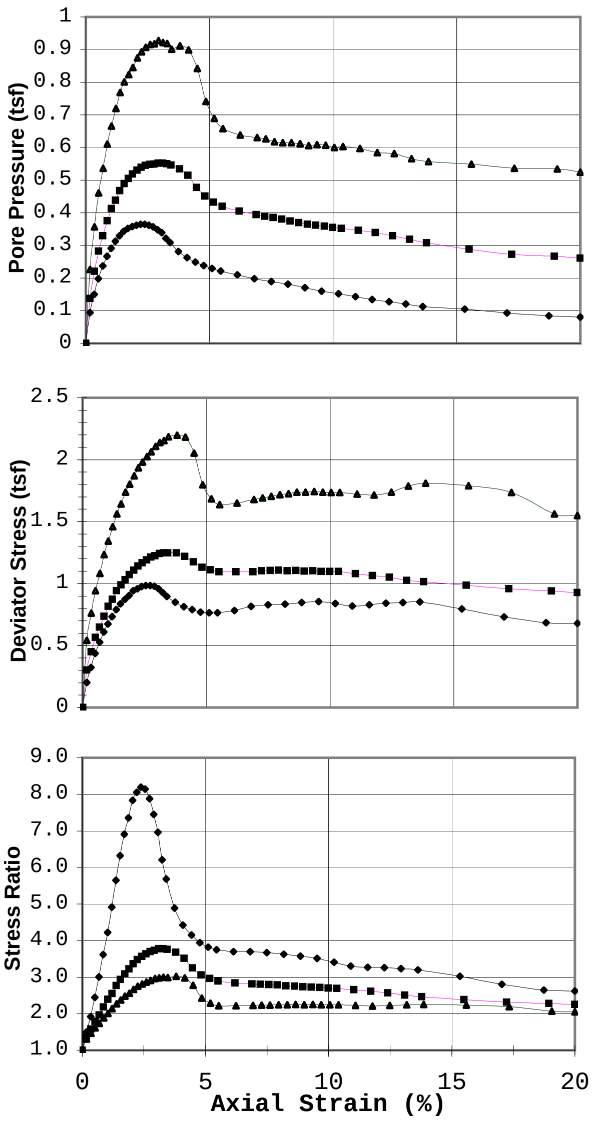
W.C. (%):
Yd (pcf):

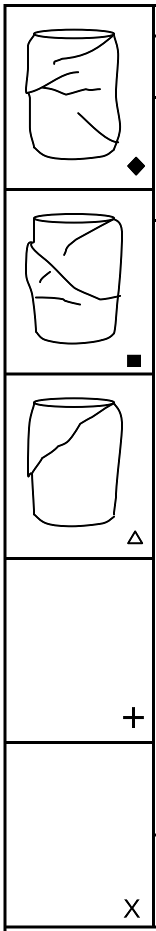
Sketch of Specimen After Failure

TRIAXIAL TEST ASTM: D 4767

Job No. **9288**
Date: **4/10/14**

Project: **Wild Rice River Structure**
Boring #: **14-207MU** Sample #: **1** Type: **5T** Depth (ft): **11-13**
Soil Type: **Fat Clay w/laminations of silt (CH)**





Failure Criterion: **Max. Stress Ratio**

Angle of internal friction, ' = **23.4 °**

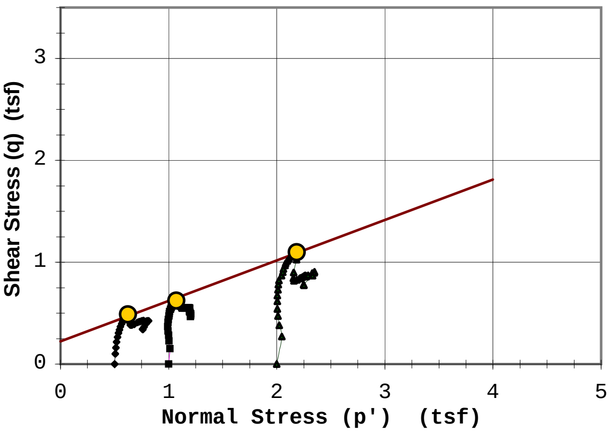
Apparent Cohesion, c' = **0.24 (tsf)**

Test Date:	3/21/14	Liquid Limit:	89.5
Test Type:	CU w/pp	Plastic Limit:	22.4
Strain Rate (in/min):	0.001467	Plasticity Index:	67.1
Strain Rate (%/min):	0.050	Spec. Gravity (Assumed):	2.72

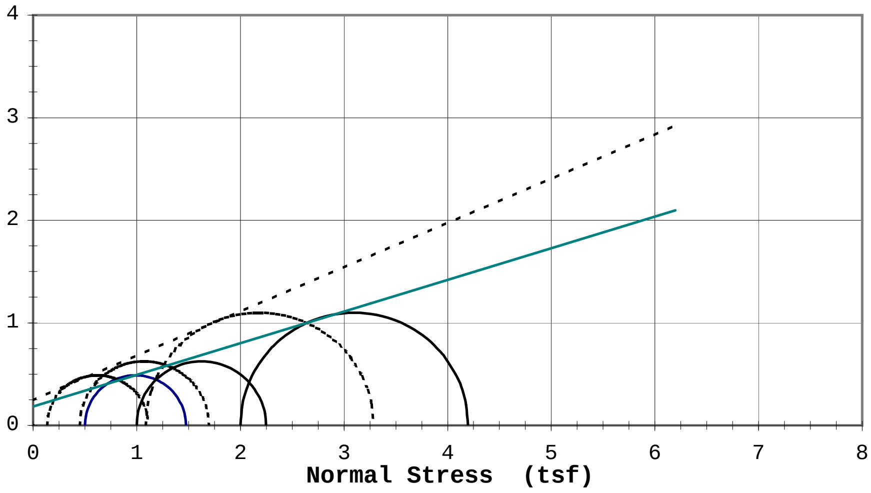
Before Consolidation	A	B	C	D	E
Diameter (in)	1.44	1.44	1.44		
Height (in)	2.94	2.94	2.95		
Water Content (%)	47.6	49.0	44.9		
Dry Density (pcf)	74.0	72.9	76.6		
Void Ratio	1.29	1.33	1.22		
After Consolidation					
Diameter (in)	1.43	1.43	1.42		
Height (in)	2.94	2.90	2.89		
Water Content (%)	46.3	46.0	40.6		
Dry Density (pcf)	75.2	75.4	80.7		
Void Ratio	1.26	1.25	1.10		
Back Pressure (tsf)	9.1	9.1	8.2		
Minor Principal Stress (tsf)	0.50	1.00	2.00		
Max. Deviator Stress (tsf)	0.98	1.25	2.20		
Ultimate Deviator Stress (tsf)	0.68	0.93	1.55		
Deviator Stress at Failure (tsf)	0.98	1.25	2.20		
Max. Pore Pressure Buildup (tsf)	0.36	0.55	0.93		
Pore Pressure Parameter "B"	0.95	0.95	0.95		
Pct. Axial Strain at Failure	2.4	3.3	3.8		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Radial drainage strips applied to trimmed specimen; Saturated, backpressured until "B" response was 0.95 to 1.00; Consolidated; All Drainage valves closed and immediately sheared.



Rupture Envelope at Failure
= 21.7 ° a = 0.2 (tsf)

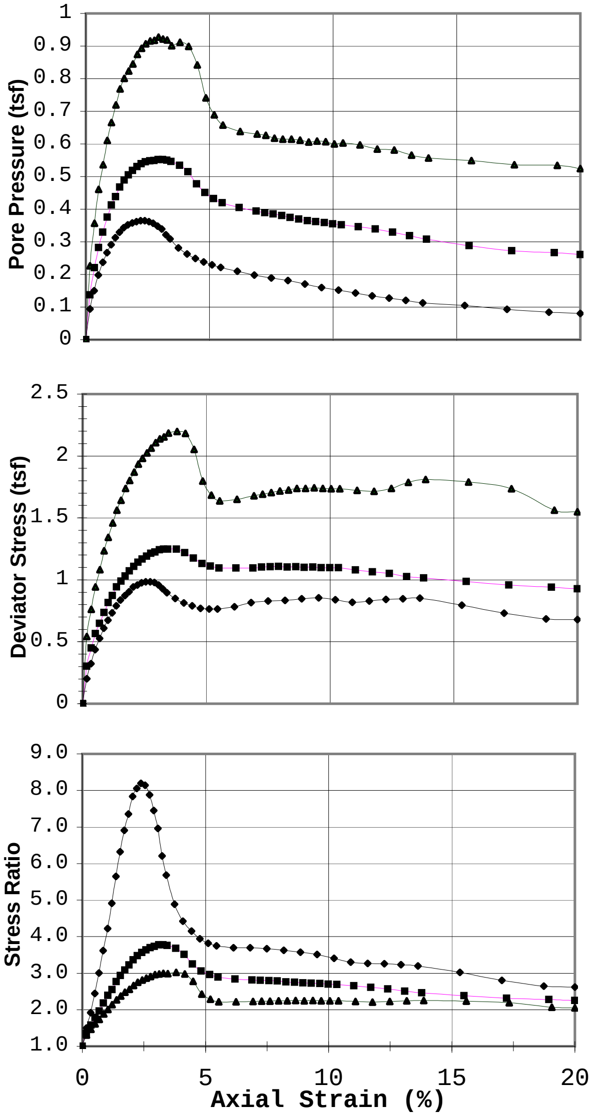


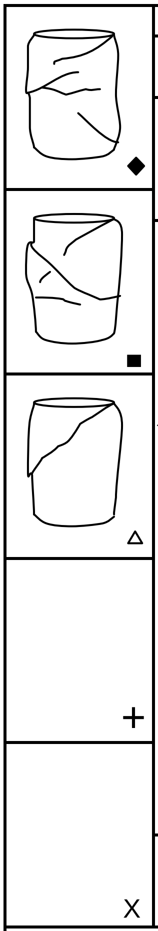
------------ Effective ': 23.4 ° c'= 0.24 (tsf)
________ Total : 17.2 ° c= 0.18 (tsf)

TRIAXIAL TEST ASTM: D 4767

Job No. **9288**
Date: **4/10/14**

Project: **Wild Rice River Structure**
Boring #: **14-207MU** Sample #: **1** Type: **5T** Depth (ft): **11-13**
Soil Type: **Fat Clay w/laminations of silt (CH)**





Failure Criterion: **Max. Deviator Stress**

Angle of internal friction, ' = **23.4 °**
Apparent Cohesion, c' = **0.24 (tsf)**

Test Date:	3/21/14	Liquid Limit:	89.5
Test Type:	CU w/pp	Plastic Limit:	22.4
Strain Rate (in/min):	0.001467	Plasticity Index:	67.1
Strain Rate (%/min):	0.050	Spec. Gravity (Assumed):	2.72

Before Consolidation	A	B	C	D	E
Diameter (in)	1.44	1.44	1.44		
Height (in)	2.94	2.94	2.95		
Water Content (%)	47.6	49.0	44.9		
Dry Density (pcf)	74.0	72.9	76.6		
Void Ratio	1.29	1.33	1.22		
After Consolidation					
Diameter (in)	1.43	1.43	1.42		
Height (in)	2.94	2.90	2.89		
Water Content (%)	46.3	46.0	40.6		
Dry Density (pcf)	75.2	75.4	80.7		
Void Ratio	1.26	1.25	1.10		
Back Pressure (tsf)	9.1	9.1	8.2		
Minor Principal Stress (tsf)	0.50	1.00	2.00		
Max. Deviator Stress (tsf)	0.98	1.25	2.20		
Ultimate Deviator Stress (tsf)	0.68	0.93	1.55		
Deviator Stress at Failure (tsf)	0.98	1.25	2.20		
Max. Pore Pressure Buildup (tsf)	0.36	0.55	0.93		
Pore Pressure Parameter "B"	0.95	0.95	0.95		
Pct. Axial Strain at Failure	2.6	3.4	3.8		

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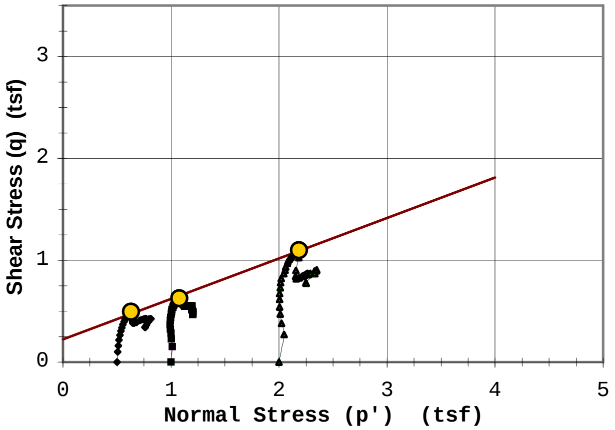
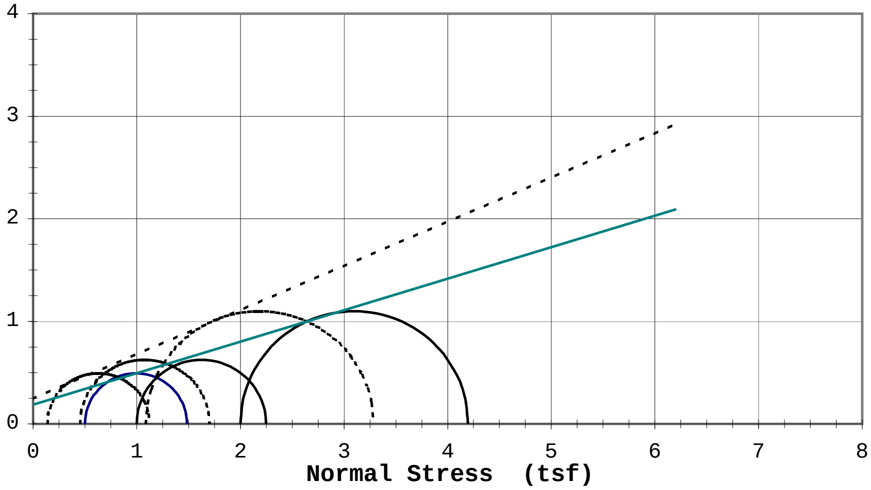
Remarks: Radial drainage strips applied to trimmed specimen; Saturated, backpressured until "B" response was 0.95 to 1.00; Consolidated; All Drainage valves closed and immediately sheared.



Rupture Envelope at Failure
= 21.6 ° a = 0.2 (tsf)



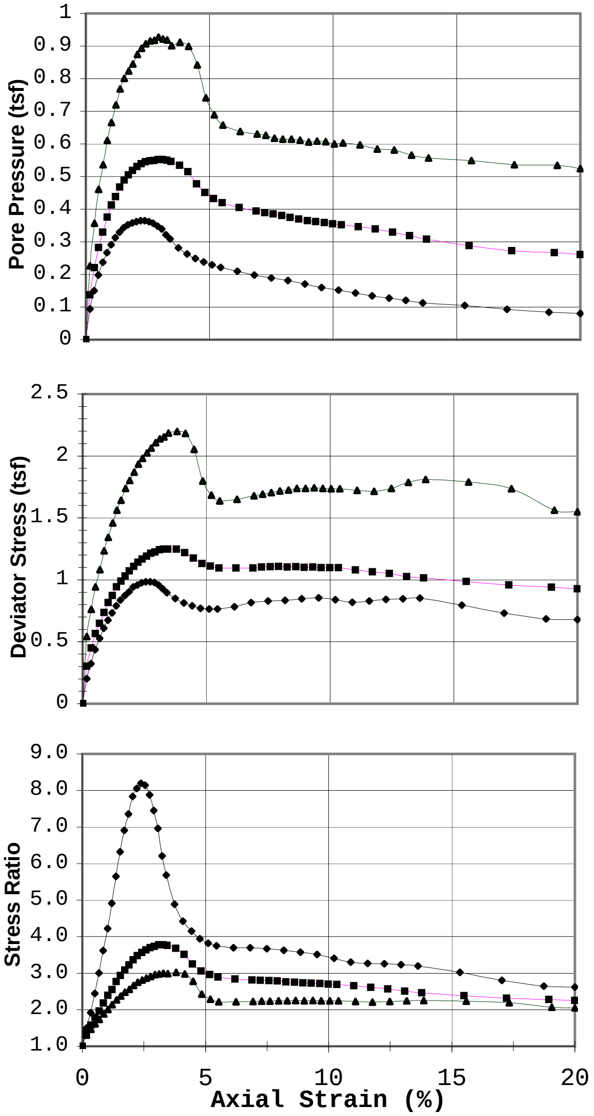
------------ Effective ': 23.4 ° c'= 0.24 (tsf)
________ Total : 17.1 ° c= 0.19 (tsf)

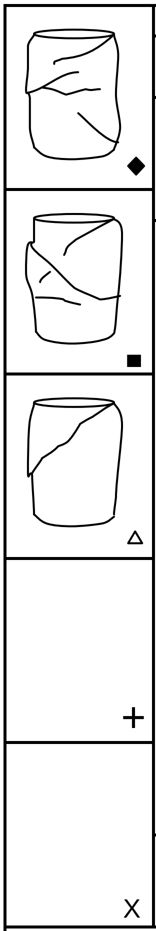
2401 W 66th Street
SOIL ENGINEERING TESTING, INC.
Richfield, Minnesota 55423-2031

TRIAXIAL TEST ASTM: D 4767

Job No. **9288**
Date: **4/10/14**

Project: **Wild Rice River Structure**
Boring #: **14-207MU** Sample #: **1** Type: **5T** Depth (ft): **11-13**
Soil Type: **Fat Clay w/laminations of silt (CH)**





Failure Criterion: **Given Strain of: 15%**

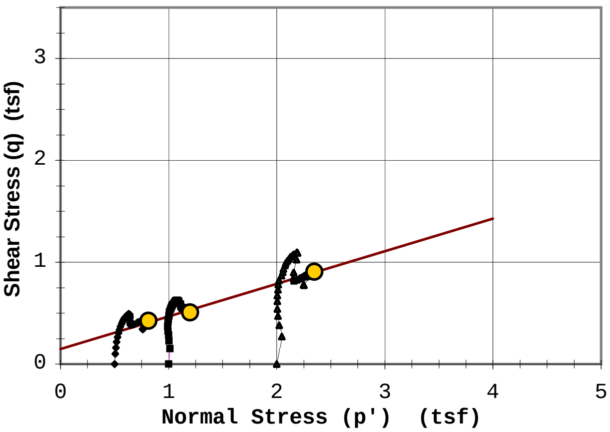
Angle of internal friction, ' = **18.6 °**
Apparent Cohesion, **c' =** **0.16 (tsf)**

Test Date:	3/21/14	Liquid Limit:	89.5
Test Type:	CU w/pp	Plastic Limit:	22.4
Strain Rate (in/min):	0.001467	Plasticity Index:	67.1
Strain Rate (%/min):	0.050	Spec. Gravity (Assumed):	2.72

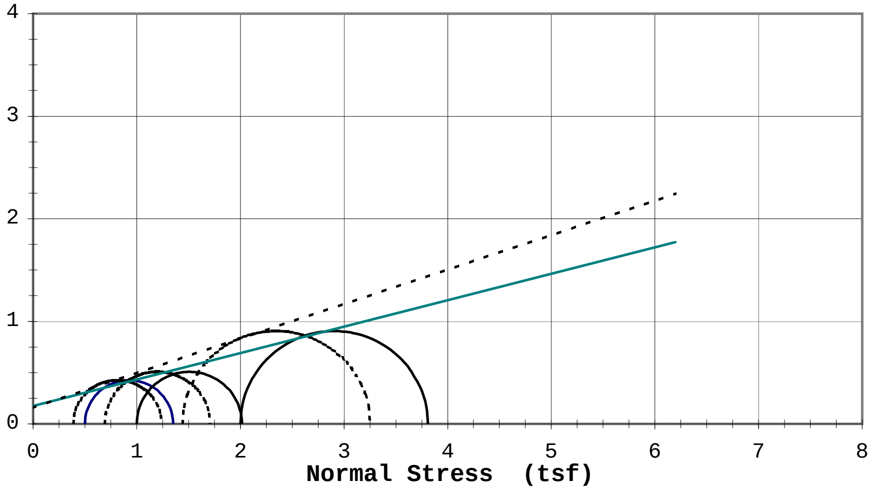
Before Consolidation	A	B	C	D	E
Diameter (in)	1.44	1.44	1.44		
Height (in)	2.94	2.94	2.95		
Water Content (%)	47.6	49.0	44.9		
Dry Density (pcf)	74.0	72.9	76.6		
Void Ratio	1.29	1.33	1.22		
After Consolidation					
Diameter (in)	1.43	1.43	1.42		
Height (in)	2.94	2.90	2.89		
Water Content (%)	46.3	46.0	40.6		
Dry Density (pcf)	75.2	75.4	80.7		
Void Ratio	1.26	1.25	1.10		
Back Pressure (tsf)	9.1	9.1	8.2		
Minor Principal Stress (tsf)	0.50	1.00	2.00		
Max. Deviator Stress (tsf)	0.98	1.25	2.20		
Ultimate Deviator Stress (tsf)	0.68	0.93	1.55		
Deviator Stress at Failure (tsf)	0.85	1.02	1.81		
Max. Pore Pressure Buildup (tsf)	0.36	0.55	0.93		
Pore Pressure Parameter "B"	0.95	0.95	0.95		
Pct. Axial Strain at Failure	15.0	15.0	15.0		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Radial drainage strips applied to trimmed specimen; Saturated, backpressured until "B" response was 0.95 to 1.00; Consolidated; All Drainage valves closed and immediately sheared.



Rupture Envelope at Failure
= 17.7 ° a = 0.1 (tsf)



------------	Effective ':	18.6 °	c'=	0.16 (tsf)
________	Total :	14.5 °	c=	0.17 (tsf)

Triaxial Data

Boring: 14-207MU Sample: 1 Depth: 11-13

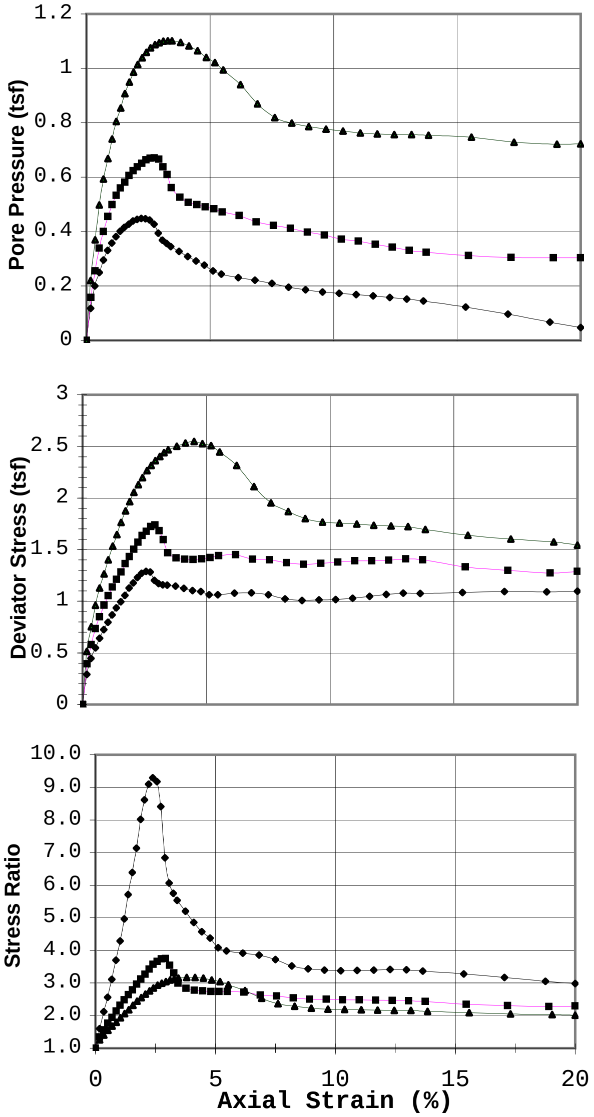
Job: **9288**
Date: 4/10/14

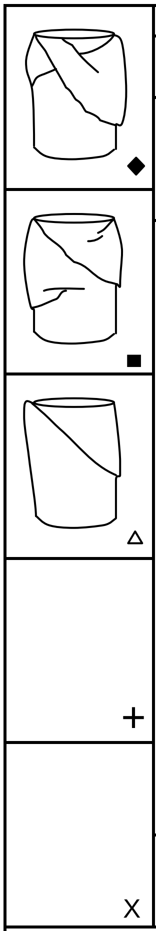
Sample 1			**Sample 2**			**Sample 3**			**Sample 4**			**Sample 5**		
Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00						
0.17	0.20	0.09	0.17	0.30	0.14	0.17	0.54	0.23						
0.34	0.32	0.15	0.34	0.45	0.22	0.35	0.76	0.36						
0.51	0.44	0.20	0.52	0.56	0.28	0.52	0.94	0.46						
0.68	0.53	0.24	0.69	0.65	0.33	0.69	1.08	0.54						
0.85	0.61	0.27	0.86	0.74	0.38	0.87	1.23	0.61						
1.02	0.67	0.29	1.04	0.82	0.41	1.04	1.34	0.67						
1.19	0.73	0.31	1.21	0.87	0.44	1.21	1.46	0.72						
1.36	0.79	0.33	1.38	0.94	0.47	1.39	1.56	0.77						
1.53	0.84	0.34	1.55	0.99	0.49	1.56	1.64	0.80						
1.70	0.87	0.35	1.72	1.03	0.50	1.73	1.74	0.82						
1.87	0.90	0.36	1.90	1.07	0.52	1.91	1.80	0.85						
2.04	0.94	0.36	2.07	1.11	0.53	2.08	1.87	0.87						
2.21	0.96	0.36	2.24	1.14	0.54	2.25	1.94	0.89						
2.38	0.98	0.36	2.41	1.17	0.55	2.43	1.98	0.91						
2.55	0.98	0.36	2.58	1.19	0.55	2.60	2.03	0.92						
2.73	0.98	0.36	2.76	1.21	0.55	2.77	2.06	0.92						
2.90	0.98	0.35	2.93	1.22	0.55	2.95	2.11	0.93						
3.07	0.96	0.34	3.10	1.24	0.55	3.12	2.14	0.92						
3.24	0.93	0.32	3.27	1.25	0.55	3.29	2.15	0.92						
3.41	0.90	0.31	3.45	1.25	0.55	3.47	2.18	0.90						
3.75	0.85	0.28	3.79	1.25	0.53	3.81	2.20	0.91						
4.09	0.81	0.26	4.14	1.22	0.51	4.16	2.18	0.90						
4.43	0.79	0.25	4.48	1.17	0.48	4.51	2.05	0.84						
4.77	0.77	0.24	4.82	1.13	0.45	4.85	1.80	0.74						
5.11	0.76	0.23	5.17	1.11	0.43	5.20	1.68	0.69						
5.45	0.76	0.22	5.51	1.10	0.42	5.55	1.64	0.66						
6.13	0.78	0.21	6.20	1.09	0.40	6.24	1.65	0.64						
6.81	0.82	0.20	6.89	1.10	0.39	6.93	1.68	0.63						
7.49	0.83	0.19	7.24	1.10	0.39	7.28	1.69	0.63						
8.18	0.84	0.18	7.58	1.10	0.39	7.63	1.71	0.62						
8.86	0.85	0.17	7.93	1.11	0.38	7.97	1.72	0.61						
9.54	0.85	0.16	8.27	1.10	0.37	8.32	1.72	0.61						
10.22	0.84	0.15	8.62	1.10	0.37	8.66	1.74	0.61						
10.90	0.82	0.14	8.96	1.10	0.36	9.01	1.74	0.61						
11.58	0.83	0.13	9.30	1.10	0.36	9.36	1.74	0.61						
12.26	0.84	0.13	9.65	1.10	0.36	9.70	1.74	0.61						
12.94	0.85	0.12	9.99	1.10	0.35	10.05	1.74	0.60						
13.63	0.85	0.11	10.34	1.10	0.35	10.40	1.74	0.60						
15.33	0.80	0.11	11.03	1.08	0.35	11.09	1.72	0.60						
17.03	0.73	0.09	11.72	1.06	0.34	11.78	1.72	0.58						
18.73	0.69	0.08	12.40	1.05	0.33	12.48	1.74	0.58						
20.00	0.68	0.08	13.09	1.03	0.32	13.17	1.79	0.57						
			13.78	1.02	0.31	13.86	1.81	0.56						
			15.51	0.99	0.29	15.60	1.79	0.55						
			17.23	0.96	0.27	17.33	1.74	0.54						
			18.95	0.94	0.27	19.06	1.56	0.53						
			20.00	0.93	0.26	20.00	1.55	0.52						

TRIAXIAL TEST ASTM: D 4767

Job No. **9288**
Date: **4/10/14**

Project: **Wild Rice River Structure**
Boring #: **14-207MU** Sample #: **2** Type: **5T** Depth (ft): **20.2-22.2**
Soil Type: **Fat Clay (CH)**





Failure Criterion: **Max. Stress Ratio**

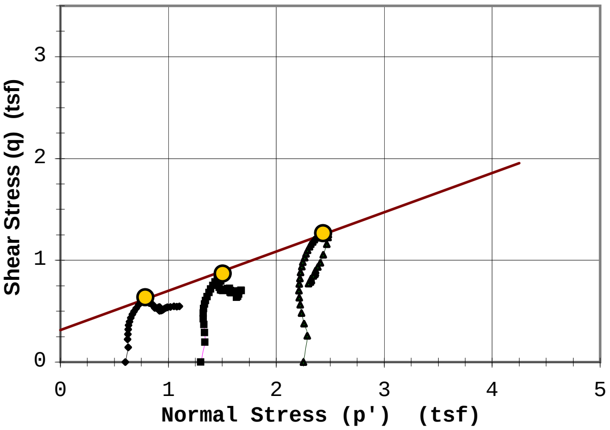
Angle of internal friction, ' = **22.7 °**
Apparent Cohesion, c' = **0.34 (tsf)**

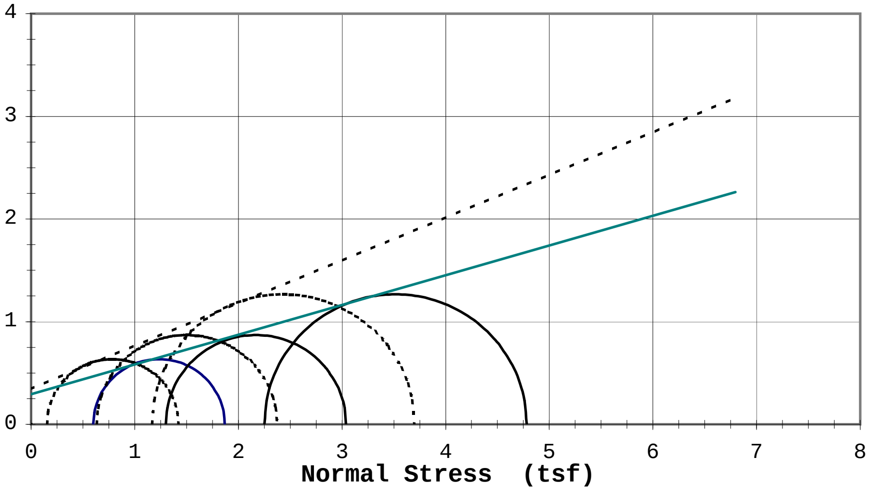
Test Date:	3/25/14	Liquid Limit:	71.9
Test Type:	CU w/pp	Plastic Limit:	21.8
Strain Rate (in/min):	0.001465	Plasticity Index:	50.1
Strain Rate (%/min):	0.050	Spec. Gravity (Assumed):	2.75

Before Consolidation	A	B	C	D	E
Diameter (in)	1.44	1.44	1.44		
Height (in)	2.94	2.94	2.94		
Water Content (%)	44.1	42.5	43.9		
Dry Density (pcf)	77.6	79.2	77.7		
Void Ratio	1.21	1.17	1.21		
After Consolidation					
Diameter (in)	1.43	1.43	1.42		
Height (in)	2.93	2.91	2.89		
Water Content (%)	42.2	40.3	40.6		
Dry Density (pcf)	79.5	81.4	81.1		
Void Ratio	1.16	1.11	1.12		
Back Pressure (tsf)	3.7	8.5	9.1		
Minor Principal Stress (tsf)	0.60	1.30	2.25		
Max. Deviator Stress (tsf)	1.29	1.74	2.55		
Ultimate Deviator Stress (tsf)	1.10	1.29	1.54		
Deviator Stress at Failure (tsf)	1.27	1.74	2.53		
Max. Pore Pressure Buildup (tsf)	0.45	0.67	1.10		
Pore Pressure Parameter "B"	0.95	0.95	0.95		
Pct. Axial Strain at Failure	2.4	2.9	4.2		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Radial drainage strips applied to trimmed specimen; Saturated, backpressured until "B" response was 0.95 to 1.00; Consolidated; All Drainage valves closed and immediately sheared.





Rupture Envelope at Failure		
= 21.1 °	a =	0.3 (tsf)

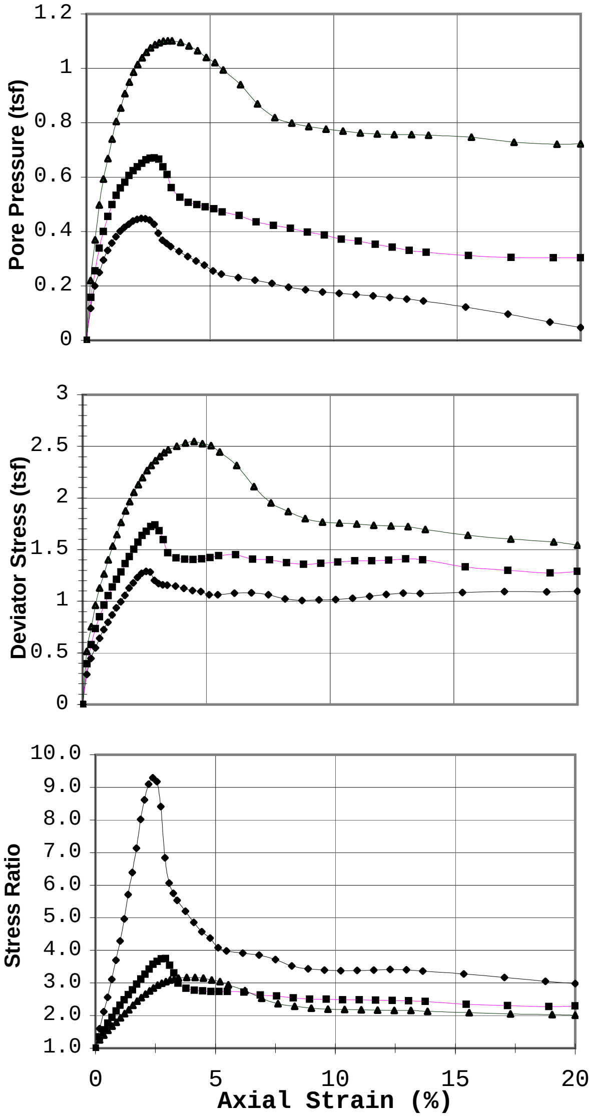
------------	Effective ':	22.7 °	c'=	0.34 (tsf)
________	Total :	16.1 °	c=	0.29 (tsf)

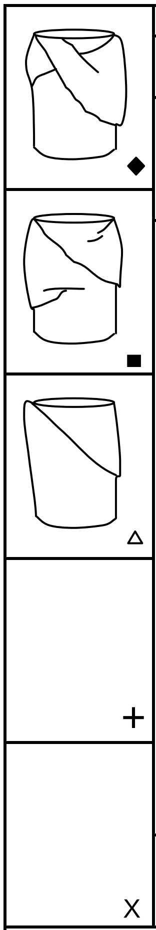
SOIL ENGINEERING TESTING, INC.
2401 W 66th Street
Richfield, Minnesota 55423-2031

TRIAXIAL TEST ASTM: D 4767

Job No. **9288**
Date: **4/10/14**

Project: **Wild Rice River Structure**
Boring #: **14-207MU** Sample #: **2** Type: **5T** Depth (ft): **20.2-22.2**
Soil Type: **Fat Clay (CH)**





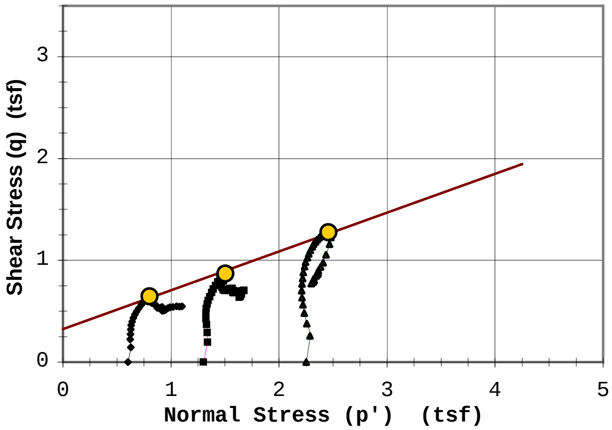
Failure Criterion: **Max. Deviator Stress**
Angle of internal friction, ' = **22.4 °**
Apparent Cohesion, c' = **0.35 (tsf)**

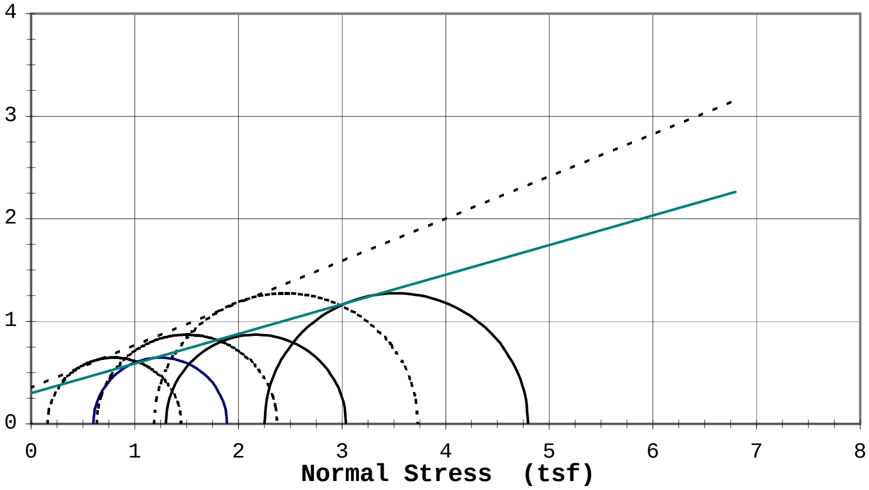
Test Date:	3/25/14	Liquid Limit:	71.9
Test Type:	CU w/pp	Plastic Limit:	21.8
Strain Rate (in/min):	0.001465	Plasticity Index:	50.1
Strain Rate (%/min):	0.050	Spec. Gravity (Assumed):	2.75

Before Consolidation	A	B	C	D	E
Diameter (in)	1.44	1.44	1.44		
Height (in)	2.94	2.94	2.94		
Water Content (%)	44.1	42.5	43.9		
Dry Density (pcf)	77.6	79.2	77.7		
Void Ratio	1.21	1.17	1.21		
After Consolidation					
Diameter (in)	1.43	1.43	1.42		
Height (in)	2.93	2.91	2.89		
Water Content (%)	42.2	40.3	40.6		
Dry Density (pcf)	79.5	81.4	81.1		
Void Ratio	1.16	1.11	1.12		
Back Pressure (tsf)	3.7	8.5	9.1		
Minor Principal Stress (tsf)	0.60	1.30	2.25		
Max. Deviator Stress (tsf)	1.29	1.74	2.55		
Ultimate Deviator Stress (tsf)	1.10	1.29	1.54		
Deviator Stress at Failure (tsf)	1.29	1.74	2.55		
Max. Pore Pressure Buildup (tsf)	0.45	0.67	1.10		
Pore Pressure Parameter "B"	0.95	0.95	0.95		
Pct. Axial Strain at Failure	2.6	2.9	4.5		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Radial drainage strips applied to trimmed specimen; Saturated, backpressured until "B" response was 0.95 to 1.00; Consolidated; All Drainage valves closed and immediately sheared.





Rupture Envelope at Failure
= 20.9 ° a = 0.3 (tsf)

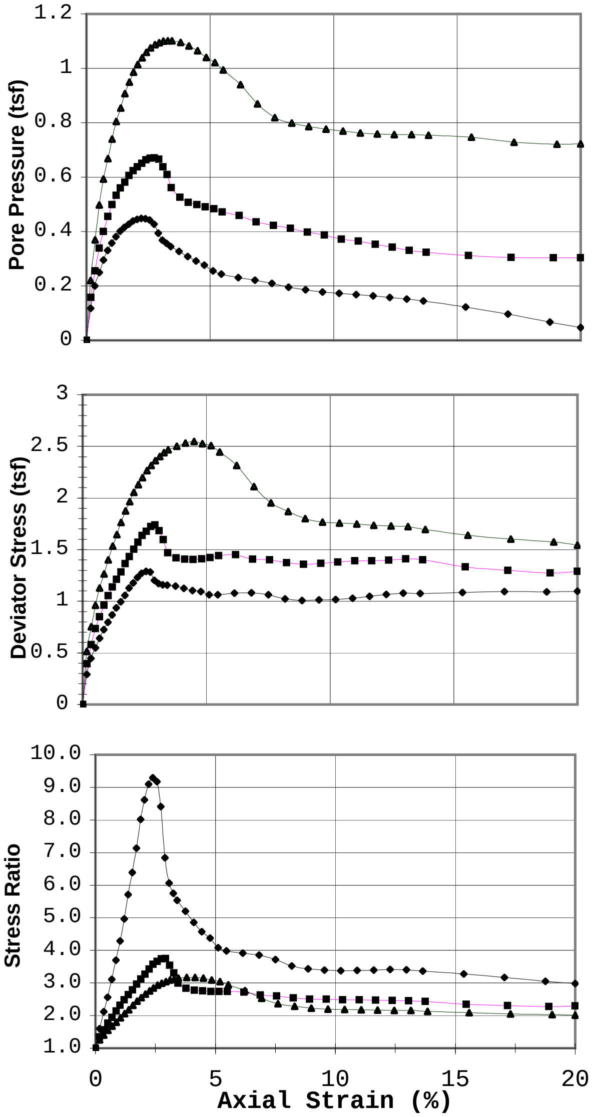
------------	Effective ':	22.4 °	c'=	0.35 (tsf)
________	Total :	16.1 °	c=	0.30 (tsf)

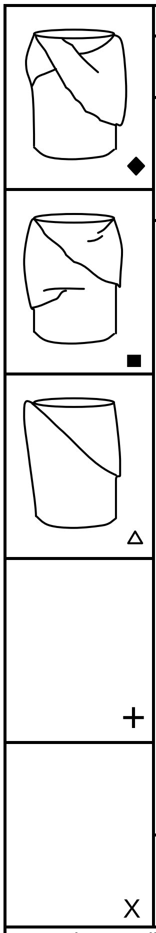
SOIL ENGINEERING TESTING, INC.
2401 W. 66th Street Richfield, Minnesota 55423-2031

TRIAXIAL TEST ASTM: D 4767

Job No. **9288**
Date: **4/10/14**

Project: **Wild Rice River Structure**
Boring #: **14-207MU** Sample #: **2** Type: **5T** Depth (ft): **20.2-22.2**
Soil Type: **Fat Clay (CH)**





Failure Criterion: **Given Strain of: 15%**

Angle of internal friction, ' = **13.2 °**
Apparent Cohesion, **c' =** **0.32 (tsf)**

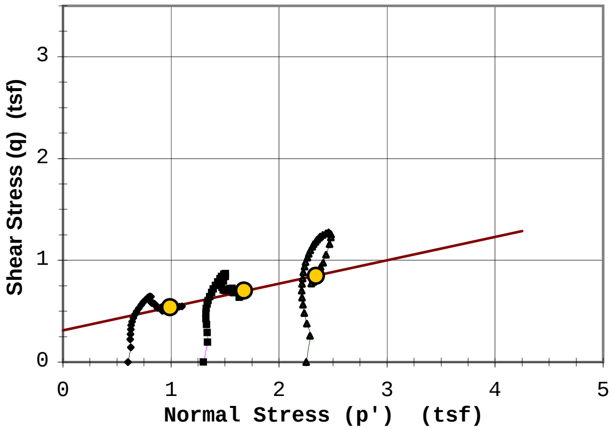
Test Date: 3/25/14
Test Type: CU w/pp
Strain Rate (in/min): 0.001465
Strain Rate (%/min): 0.050

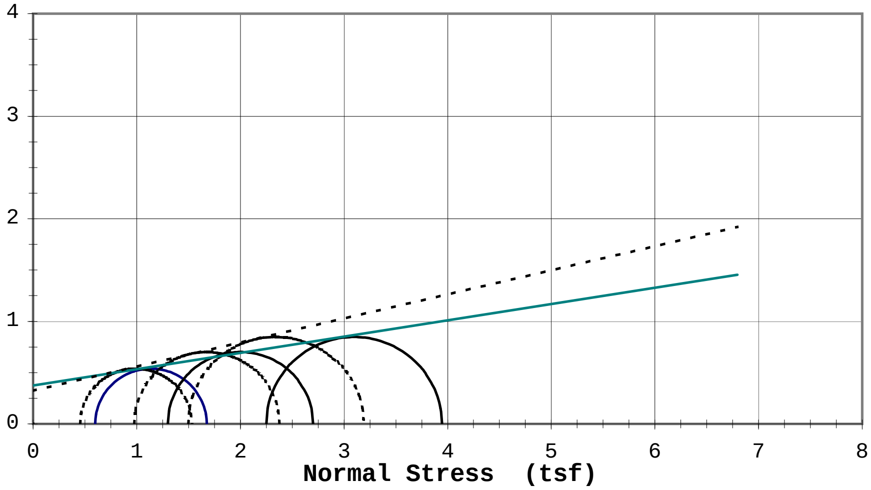
Liquid Limit: 71.9
Plastic Limit: 21.8
Plasticity Index: 50.1
Spec. Gravity (Assumed): 2.75

Before Consolidation	A	B	C	D	E
Diameter (in)	1.44	1.44	1.44		
Height (in)	2.94	2.94	2.94		
Water Content (%)	44.1	42.5	43.9		
Dry Density (pcf)	77.6	79.2	77.7		
Void Ratio	1.21	1.17	1.21		
After Consolidation					
Diameter (in)	1.43	1.43	1.42		
Height (in)	2.93	2.91	2.89		
Water Content (%)	42.2	40.3	40.6		
Dry Density (pcf)	79.5	81.4	81.1		
Void Ratio	1.16	1.11	1.12		
Back Pressure (tsf)	3.7	8.5	9.1		
Minor Principal Stress (tsf)	0.60	1.30	2.25		
Max. Deviator Stress (tsf)	1.29	1.74	2.55		
Ultimate Deviator Stress (tsf)	1.10	1.29	1.54		
Deviator Stress at Failure (tsf)	1.08	1.40	1.69		
Max. Pore Pressure Buildup (tsf)	0.45	0.67	1.10		
Pore Pressure Parameter "B"	0.95	0.95	0.95		
Pct. Axial Strain at Failure	15.0	15.0	15.0		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Radial drainage strips applied to trimmed specimen; Saturated, backpressured until "B" response was 0.95 to 1.00; Consolidated; All Drainage valves closed and immediately sheared.





Rupture Envelope at Failure
= 12.9 ° a = 0.3 (tsf)

------------ Effective ': 13.2 ° c'= 0.32 (tsf)
________ Total : 9.0 ° c= 0.37 (tsf)

SOIL ENGINEERING TESTING, INC.
2401 W. 66th Street
Richfield, Minnesota 55423-2031

Triaxial Data

Job: **9288**

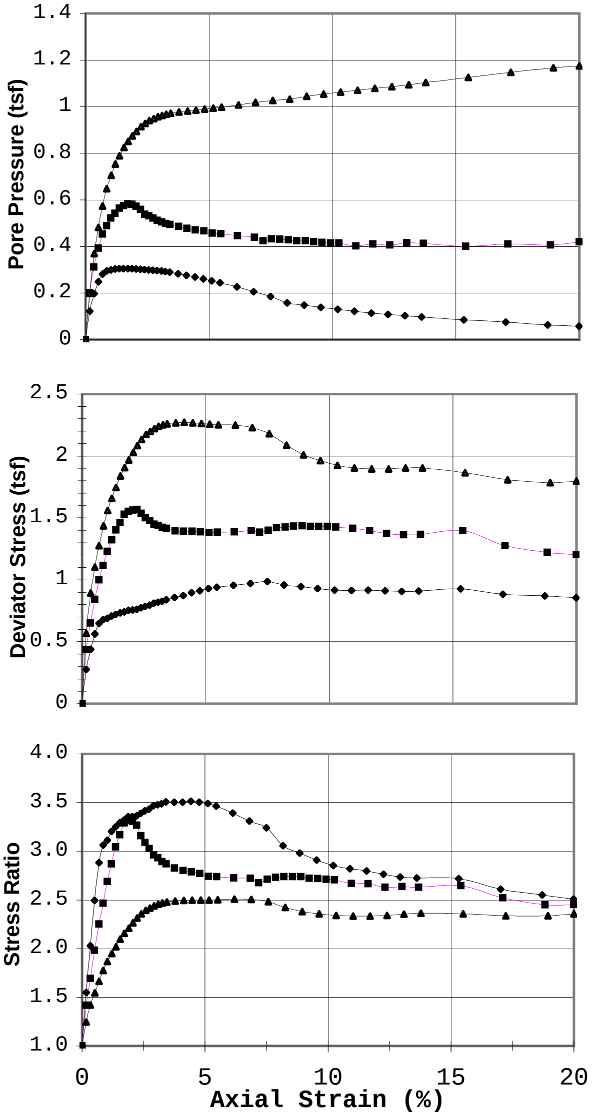
Boring: 14-207MU Sample: 2 Depth: 20.2-22.2 Date: 4/10/14

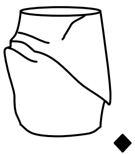
Sample 1			**Sample 2**			**Sample 3**			**Sample 4**			**Sample 5**		
Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00						
0.17	0.29	0.12	0.18	0.39	0.16	0.17	0.52	0.22						
0.34	0.45	0.20	0.35	0.58	0.25	0.35	0.75	0.37						
0.51	0.55	0.25	0.52	0.74	0.34	0.52	0.96	0.50						
0.68	0.64	0.29	0.69	0.85	0.40	0.69	1.13	0.59						
0.85	0.73	0.33	0.86	0.96	0.46	0.87	1.26	0.67						
1.02	0.80	0.36	1.03	1.05	0.50	1.04	1.40	0.74						
1.20	0.87	0.38	1.21	1.14	0.53	1.21	1.53	0.80						
1.37	0.94	0.40	1.38	1.21	0.56	1.38	1.65	0.85						
1.54	0.99	0.42	1.55	1.28	0.58	1.56	1.76	0.91						
1.71	1.06	0.43	1.72	1.36	0.61	1.73	1.87	0.95						
1.88	1.13	0.44	1.89	1.43	0.62	1.90	1.96	0.99						
2.05	1.18	0.45	2.07	1.50	0.64	2.08	2.05	1.01						
2.22	1.23	0.45	2.24	1.57	0.65	2.25	2.13	1.04						
2.39	1.27	0.45	2.41	1.64	0.66	2.42	2.20	1.06						
2.56	1.29	0.44	2.58	1.68	0.67	2.60	2.27	1.08						
2.73	1.28	0.43	2.75	1.72	0.67	2.77	2.32	1.09						
2.90	1.20	0.39	2.92	1.74	0.67	2.94	2.36	1.09						
3.07	1.17	0.37	3.10	1.68	0.64	3.12	2.40	1.10						
3.24	1.16	0.36	3.27	1.60	0.61	3.29	2.44	1.10						
3.41	1.16	0.35	3.44	1.47	0.56	3.46	2.47	1.10						
3.75	1.15	0.33	3.78	1.42	0.53	3.81	2.50	1.10						
4.09	1.13	0.31	4.13	1.41	0.51	4.15	2.53	1.08						
4.43	1.10	0.29	4.47	1.41	0.50	4.50	2.55	1.06						
4.78	1.09	0.28	4.81	1.41	0.49	4.85	2.53	1.04						
5.12	1.06	0.26	5.16	1.42	0.48	5.19	2.51	1.02						
5.46	1.06	0.24	5.50	1.44	0.47	5.54	2.45	0.99						
6.14	1.08	0.23	6.19	1.45	0.46	6.23	2.32	0.94						
6.82	1.08	0.22	6.88	1.41	0.43	6.92	2.11	0.87						
7.50	1.06	0.21	7.56	1.40	0.42	7.62	1.95	0.82						
8.19	1.02	0.19	8.25	1.37	0.41	8.31	1.87	0.80						
8.87	1.01	0.19	8.94	1.36	0.40	9.00	1.80	0.79						
9.55	1.01	0.18	9.63	1.37	0.39	9.69	1.77	0.78						
10.23	1.02	0.17	10.31	1.38	0.37	10.38	1.76	0.77						
10.91	1.03	0.17	11.00	1.39	0.37	11.08	1.75	0.76						
11.60	1.05	0.16	11.69	1.39	0.35	11.77	1.74	0.76						
12.28	1.07	0.16	12.37	1.40	0.34	12.46	1.73	0.76						
12.96	1.08	0.15	13.06	1.41	0.33	13.15	1.72	0.76						
13.64	1.08	0.15	13.75	1.40	0.32	13.85	1.69	0.75						
15.35	1.09	0.12	15.47	1.33	0.31	15.58	1.64	0.75						
17.05	1.09	0.10	17.18	1.30	0.31	17.31	1.60	0.73						
18.76	1.09	0.07	18.90	1.27	0.30	19.04	1.58	0.72						
20.00	1.10	0.05	20.00	1.29	0.30	20.00	1.54	0.72						

TRIAXIAL TEST ASTM: D 4767

Job No. **9288**
Date: **4/15/14**

Project: **Wild Rice River Structure**
Boring #: **14-207MU** Sample #: **3** Type: **5T** Depth (ft): **31-33**
Soil Type: **Fat Clay (CH)**

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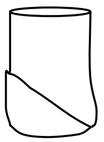
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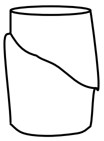
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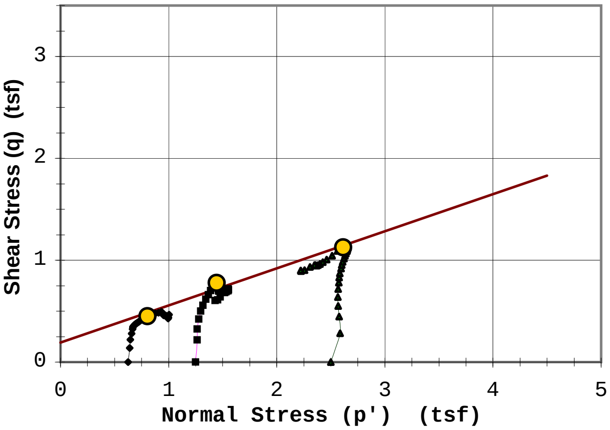
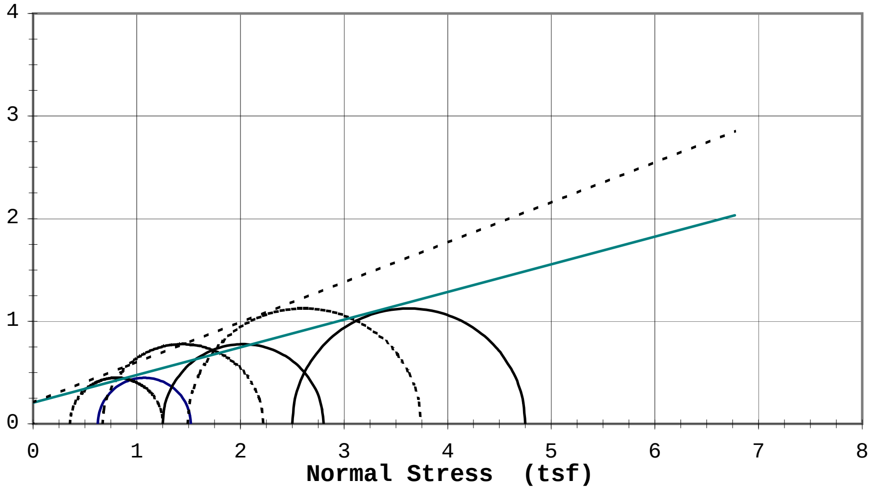
Failure Criterion:	**Max. Stress Ratio**				
Angle of internal friction, ' =	**21.3 °**				
Apparent Cohesion, c' =	**0.21 (tsf)**				
Test Date:	3/25/14	Liquid Limit:	71.4		
Test Type:	CU w/pp	Plastic Limit:	25.0		
Strain Rate (in/min):	0.00147	Plasticity Index:	46.4		
Strain Rate (%/min):	0.050	Spec. Gravity (Assumed):	2.77		

Before Consolidation	A	B	C	D	E
Diameter (in)	1.44	1.44	1.44		
Height (in)	2.94	2.94	2.95		
Water Content (%)	44.9	45.2	40.8		
Dry Density (pcf)	76.6	76.3	80.4		
Void Ratio	1.26	1.27	1.15		
After Consolidation					
Diameter (in)	1.44	1.43	1.42		
Height (in)	2.94	2.92	2.90		
Water Content (%)	44.8	44.2	38.2		
Dry Density (pcf)	77.1	77.8	84.0		
Void Ratio	1.24	1.22	1.06		
Back Pressure (tsf)	8.5	6.7	6.9		
Minor Principal Stress (tsf)	0.63	1.25	2.50		
Max. Deviator Stress (tsf)	0.99	1.57	2.27		
Ultimate Deviator Stress (tsf)	0.86	1.20	1.80		
Deviator Stress at Failure (tsf)	0.90	1.55	2.25		
Max. Pore Pressure Buildup (tsf)	0.31	0.58	1.18		
Pore Pressure Parameter "B"	0.95	0.95	0.95		
Pct. Axial Strain at Failure	4.4	1.9	6.2		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Radial drainage strips applied to trimmed specimen; Saturated, backpressured until "B" response was 0.95 to 1.00; Consolidated; All Drainage valves closed and immediately sheared.





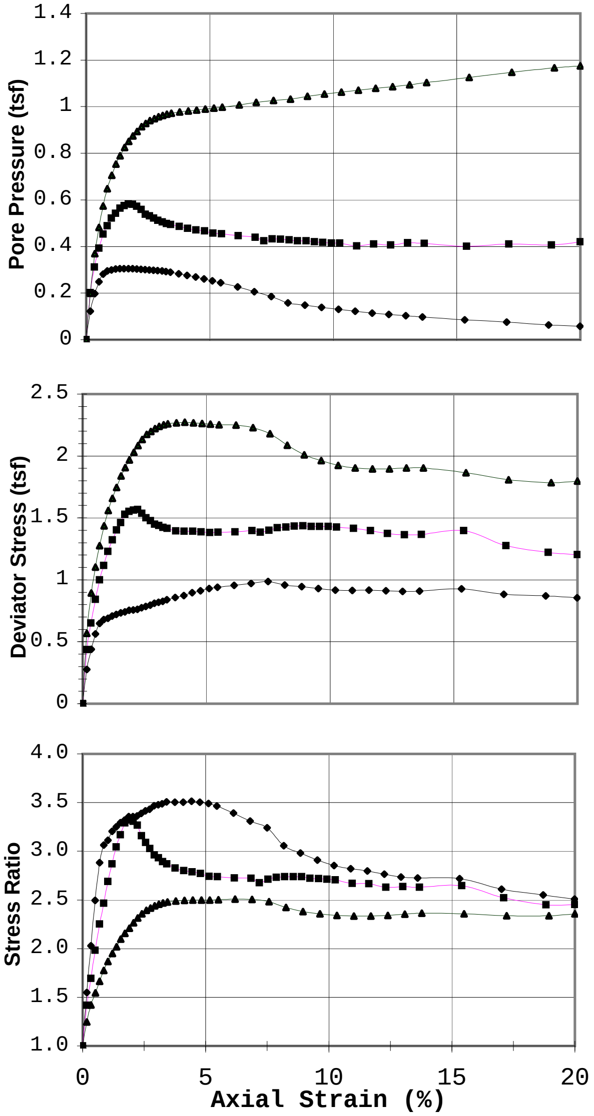
Rupture Envelope at Failure
 = 20.0 ° a = 0.2 (tsf)

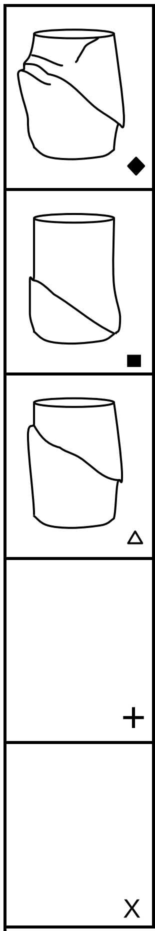
------------	Effective ':	21.3 °	c'=	0.21 (tsf)
________	Total :	15.1 °	c=	0.21 (tsf)

TRIAXIAL TEST ASTM: D 4767

Job No. **9288**
Date: **4/15/14**

Project: **Wild Rice River Structure**
Boring #: **14-207MU** Sample #: **3** Type: **5T** Depth (ft): **31-33**
Soil Type: **Fat Clay (CH)**





Failure Criterion: **Max. Deviator Stress**
Angle of internal friction, ' = **21.2 °**
Apparent Cohesion, c' = **0.21 (tsf)**

Test Date:	3/25/14	Liquid Limit:	71.4
Test Type:	CU w/pp	Plastic Limit:	25.0
Strain Rate (in/min):	0.00147	Plasticity Index:	46.4
Strain Rate (%/min):	0.050	Spec. Gravity (Assumed):	2.77

Before Consolidation	A	B	C	D	E
Diameter (in)	1.44	1.44	1.44		
Height (in)	2.94	2.94	2.95		
Water Content (%)	44.9	45.2	40.8		
Dry Density (pcf)	76.6	76.3	80.4		
Void Ratio	1.26	1.27	1.15		
After Consolidation					
Diameter (in)	1.44	1.43	1.42		
Height (in)	2.94	2.92	2.90		
Water Content (%)	44.8	44.2	38.2		
Dry Density (pcf)	77.1	77.8	84.0		
Void Ratio	1.24	1.22	1.06		
Back Pressure (tsf)	8.5	6.7	6.9		
Minor Principal Stress (tsf)	0.63	1.25	2.50		
Max. Deviator Stress (tsf)	0.99	1.57	2.27		
Ultimate Deviator Stress (tsf)	0.86	1.20	1.80		
Deviator Stress at Failure (tsf)	0.99	1.57	2.27		
Max. Pore Pressure Buildup (tsf)	0.31	0.58	1.18		
Pore Pressure Parameter "B"	0.95	0.95	0.95		
Pct. Axial Strain at Failure	7.5	2.2	4.1		

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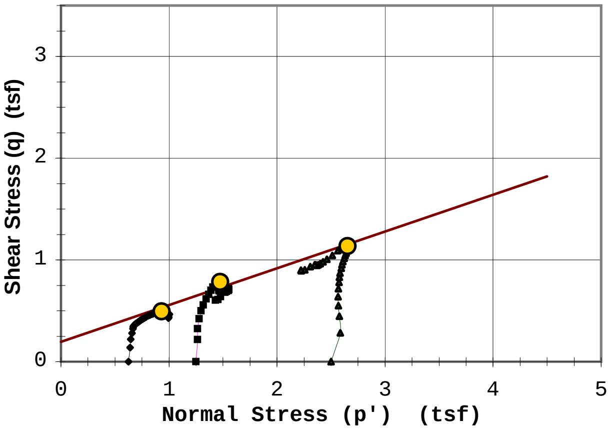
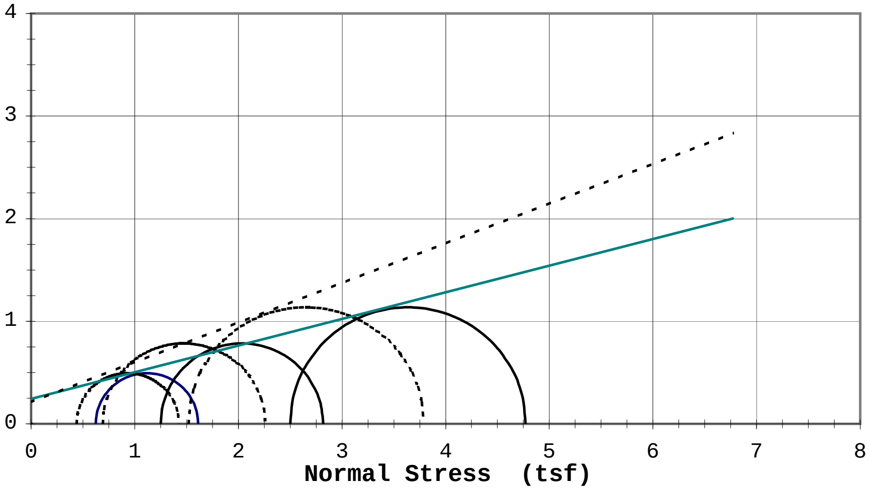
Remarks: Radial drainage strips applied to trimmed specimen; Saturated, backpressured until "B" response was 0.95 to 1.00; Consolidated; All Drainage valves closed and immediately sheared.





Rupture Envelope at Failure
= 19.9 ° a = 0.2 (tsf)

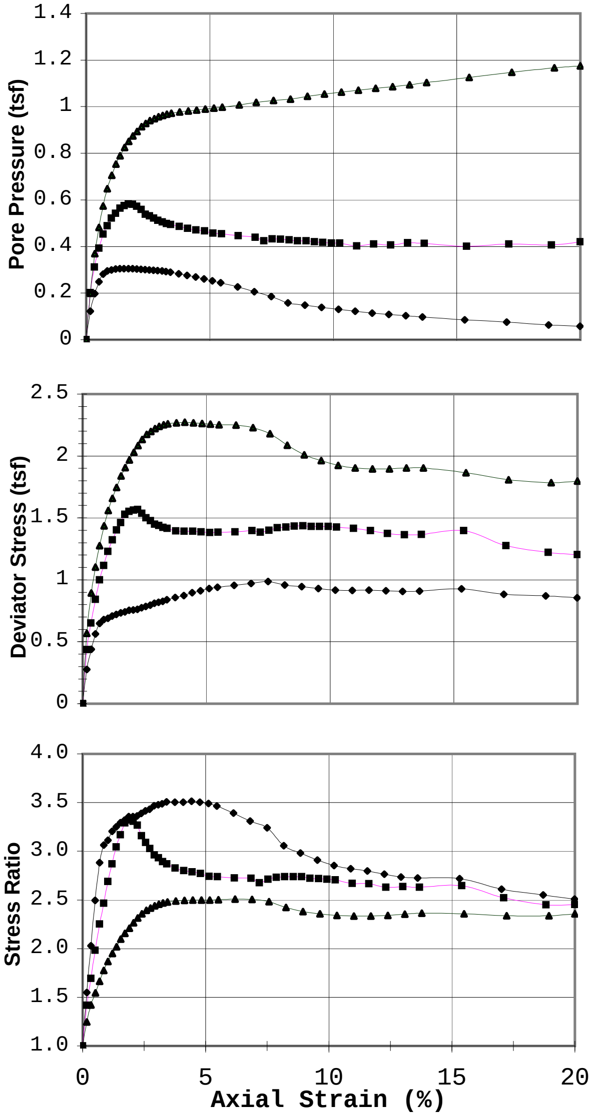
------------ Effective ': 21.2 ° c'= 0.21 (tsf)
________ Total : 14.6 ° c= 0.24 (tsf)

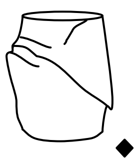
SOIL ENGINEERING TESTING, INC.
2401 W 66th Street
Richfield, Minnesota 55423-2031

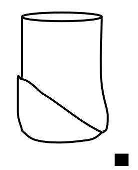
TRIAXIAL TEST ASTM: D 4767

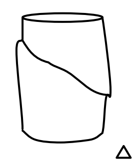
Job No. **9288**
Date: **4/15/14**

Project: **Wild Rice River Structure**
Boring #: **14-207MU** Sample #: **3** Type: **5T** Depth (ft): **31-33**
Soil Type: **Fat Clay (CH)**









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Failure Criterion: **Given Strain of: 15%**

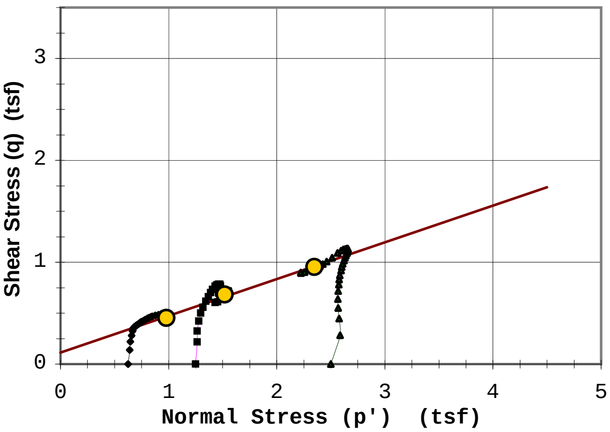
Angle of internal friction, ' = **21.1 °**
Apparent Cohesion, c' = **0.12 (tsf)**

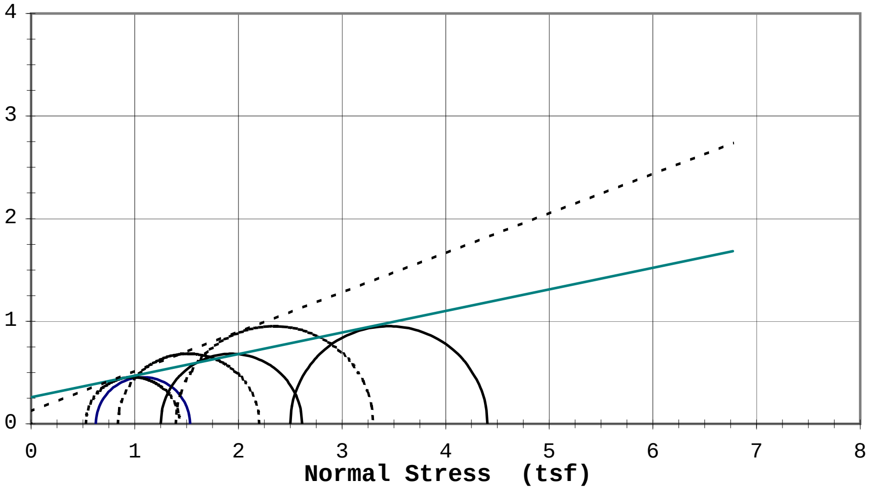
Test Date:	3/25/14	Liquid Limit:	71.4
Test Type:	CU w/pp	Plastic Limit:	25.0
Strain Rate (in/min):	0.00147	Plasticity Index:	46.4
Strain Rate (%/min):	0.050	Spec. Gravity (Assumed):	2.77

Before Consolidation	A	B	C	D	E
Diameter (in)	1.44	1.44	1.44		
Height (in)	2.94	2.94	2.95		
Water Content (%)	44.9	45.2	40.8		
Dry Density (pcf)	76.6	76.3	80.4		
Void Ratio	1.26	1.27	1.15		
After Consolidation					
Diameter (in)	1.44	1.43	1.42		
Height (in)	2.94	2.92	2.90		
Water Content (%)	44.8	44.2	38.2		
Dry Density (pcf)	77.1	77.8	84.0		
Void Ratio	1.24	1.22	1.06		
Back Pressure (tsf)	8.5	6.7	6.9		
Minor Principal Stress (tsf)	0.63	1.25	2.50		
Max. Deviator Stress (tsf)	0.99	1.57	2.27		
Ultimate Deviator Stress (tsf)	0.86	1.20	1.80		
Deviator Stress at Failure (tsf)	0.91	1.37	1.90		
Max. Pore Pressure Buildup (tsf)	0.31	0.58	1.18		
Pore Pressure Parameter "B"	0.95	0.95	0.95		
Pct. Axial Strain at Failure	15.0	15.0	15.0		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Radial drainage strips applied to trimmed specimen; Saturated, backpressured until "B" response was 0.95 to 1.00; Consolidated; All Drainage valves closed and immediately sheared.





Rupture Envelope at Failure
= 19.8 ° a = 0.1 (tsf)

------------	Effective ':	21.1 °	c'=	0.12 (tsf)
________	Total :	11.9 °	c=	0.26 (tsf)

SOIL ENGINEERING TESTING, INC.
2401 W 66th Street Richfield, Minnesota 55423-2031

Triaxial Data

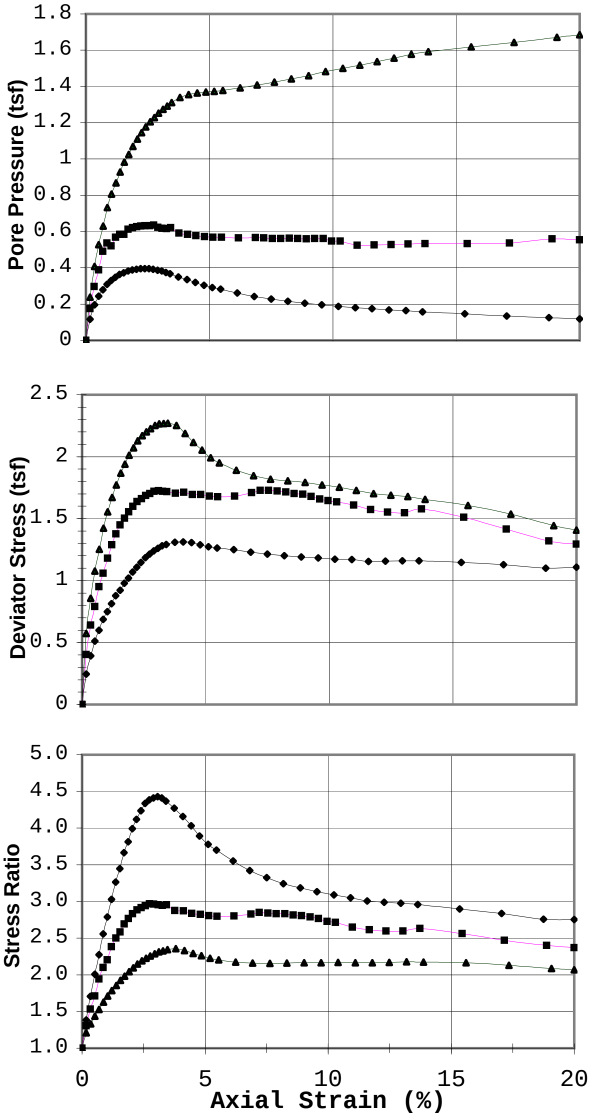
Job: **9288**
Boring: 14-207MU
Sample: 3
Depth: 31-33
Date: 4/15/14

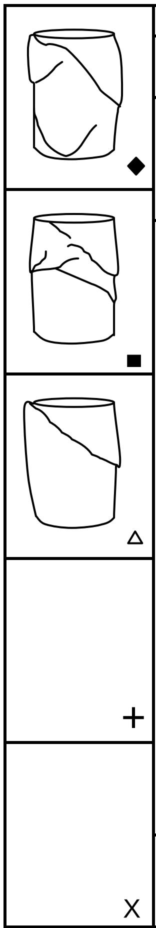
Sample 1			**Sample 2**			**Sample 3**			**Sample 4**			**Sample 5**		
Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00						
0.17	0.28	0.12	0.17	0.44	0.20	0.17	0.57	0.20						
0.34	0.44	0.20	0.34	0.65	0.31	0.34	0.89	0.37						
0.51	0.56	0.25	0.51	0.84	0.39	0.52	1.10	0.48						
0.68	0.65	0.28	0.68	1.00	0.45	0.69	1.28	0.57						
0.85	0.68	0.30	0.86	1.11	0.49	0.86	1.44	0.65						
1.02	0.69	0.30	1.03	1.23	0.52	1.03	1.56	0.71						
1.19	0.71	0.30	1.20	1.32	0.54	1.21	1.66	0.75						
1.36	0.72	0.30	1.37	1.40	0.56	1.38	1.74	0.79						
1.53	0.73	0.31	1.54	1.46	0.58	1.55	1.84	0.82						
1.71	0.74	0.31	1.71	1.53	0.58	1.72	1.91	0.85						
1.88	0.75	0.30	1.88	1.55	0.58	1.89	1.97	0.87						
2.05	0.76	0.30	2.05	1.56	0.57	2.07	2.03	0.89						
2.21	0.76	0.30	2.23	1.57	0.56	2.24	2.09	0.91						
2.39	0.77	0.30	2.40	1.54	0.54	2.41	2.13	0.93						
2.56	0.79	0.30	2.57	1.50	0.53	2.58	2.17	0.94						
2.73	0.79	0.30	2.74	1.48	0.52	2.76	2.20	0.95						
2.90	0.81	0.30	2.91	1.45	0.51	2.93	2.22	0.96						
3.07	0.82	0.29	3.08	1.44	0.50	3.10	2.24	0.96						
3.23	0.83	0.29	3.25	1.42	0.50	3.27	2.25	0.97						
3.41	0.84	0.29	3.42	1.41	0.49	3.44	2.26	0.97						
3.75	0.86	0.28	3.77	1.40	0.49	3.79	2.27	0.98						
4.09	0.87	0.28	4.11	1.39	0.48	4.13	2.27	0.98						
4.43	0.90	0.27	4.45	1.39	0.47	4.48	2.27	0.99						
4.77	0.91	0.26	4.79	1.39	0.47	4.82	2.26	0.99						
5.11	0.93	0.25	5.13	1.38	0.46	5.17	2.26	0.99						
5.45	0.94	0.24	5.48	1.38	0.45	5.51	2.25	1.00						
6.13	0.95	0.23	6.16	1.39	0.45	6.20	2.25	1.01						
6.81	0.97	0.21	6.85	1.40	0.44	6.89	2.23	1.02						
7.49	0.99	0.19	7.19	1.38	0.42	7.58	2.18	1.03						
8.17	0.96	0.16	7.53	1.40	0.43	8.27	2.09	1.03						
8.85	0.95	0.15	7.87	1.42	0.43	8.96	2.01	1.04						
9.53	0.93	0.14	8.22	1.43	0.43	9.64	1.96	1.05						
10.21	0.92	0.13	8.56	1.43	0.42	10.33	1.93	1.06						
10.89	0.92	0.12	8.90	1.44	0.42	11.02	1.90	1.07						
11.57	0.92	0.11	9.24	1.43	0.42	11.71	1.90	1.08						
12.25	0.91	0.11	9.58	1.43	0.42	12.40	1.90	1.09						
12.94	0.91	0.10	9.93	1.43	0.41	13.09	1.90	1.09						
13.61	0.91	0.10	10.27	1.43	0.41	13.78	1.90	1.10						
15.32	0.93	0.08	10.95	1.42	0.40	15.50	1.87	1.13						
17.02	0.88	0.08	11.64	1.40	0.41	17.22	1.81	1.15						
18.72	0.87	0.06	12.32	1.38	0.41	18.94	1.78	1.17						
19.98	0.86	0.06	13.01	1.36	0.42	20.00	1.80	1.18						
			13.69	1.37	0.41									
			15.40	1.40	0.40									
			17.12	1.28	0.41									
			18.83	1.22	0.41									
			20.00	1.20	0.42									

TRIAXIAL TEST ASTM: D 4767

Job No. **9288**
Date: **4/10/14**

Project: **Wild Rice River Structure**
Boring #: **14-207MU** Sample #: **5** Type: **5T** Depth (ft): **46-48**
Soil Type: **Fat Clay (CH)**





Failure Criterion: **Max. Stress Ratio**

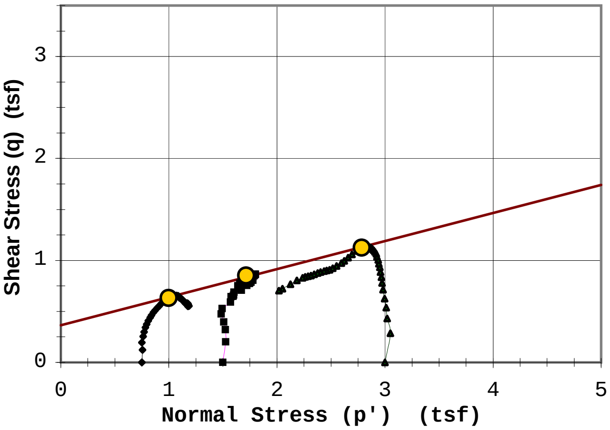
Angle of internal friction, ' = **16.0 °**
Apparent Cohesion, c' = **0.38 (tsf)**

Test Date:	3/25/14	Liquid Limit:	84.0
Test Type:	CU w/pp	Plastic Limit:	24.8
Strain Rate (in/min):	0.014665	Plasticity Index:	59.2
Strain Rate (%/min):	0.500	Spec. Gravity (Assumed):	2.73

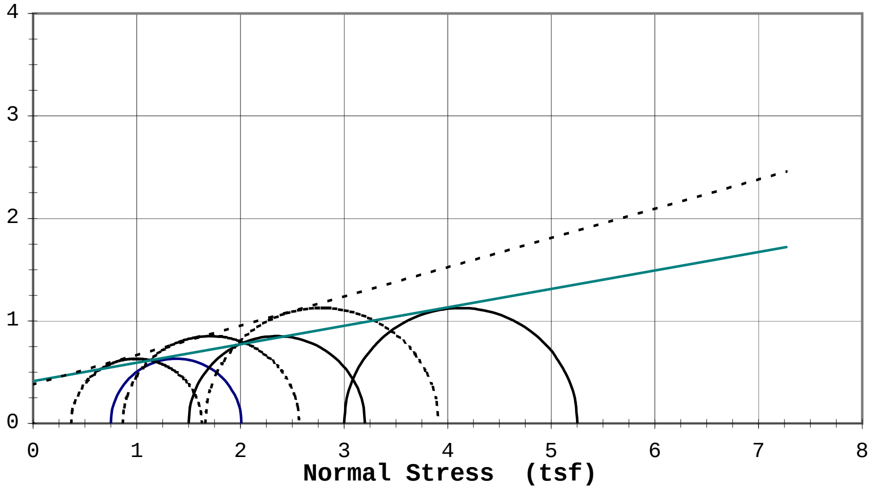
Before Consolidation	A	B	C	D	E
Diameter (in)	1.44	1.44	1.44		
Height (in)	2.94	2.94	2.94		
Water Content (%)	48.5	44.7	48.4		
Dry Density (pcf)	73.0	76.4	73.2		
Void Ratio	1.34	1.23	1.33		
After Consolidation					
Diameter (in)	1.44	1.43	1.41		
Height (in)	2.93	2.91	2.88		
Water Content (%)	48.0	42.8	43.5		
Dry Density (pcf)	73.8	78.6	77.9		
Void Ratio	1.31	1.17	1.19		
Back Pressure (tsf)	8.9	8.9	9.1		
Minor Principal Stress (tsf)	0.75	1.50	3.00		
Max. Deviator Stress (tsf)	1.31	1.73	2.27		
Ultimate Deviator Stress (tsf)	1.11	1.29	1.41		
Deviator Stress at Failure (tsf)	1.26	1.70	2.25		
Max. Pore Pressure Buildup (tsf)	0.40	0.64	1.68		
Pore Pressure Parameter "B"	0.95	0.95	0.95		
Pct. Axial Strain at Failure	3.1	2.7	3.8		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Radial drainage strips applied to trimmed specimen; Saturated, backpressured until "B" response was 0.95 to 1.00; Consolidated; All Drainage valves closed and immediately sheared.



Rupture Envelope at Failure
= 15.4 ° a = 0.4 (tsf)



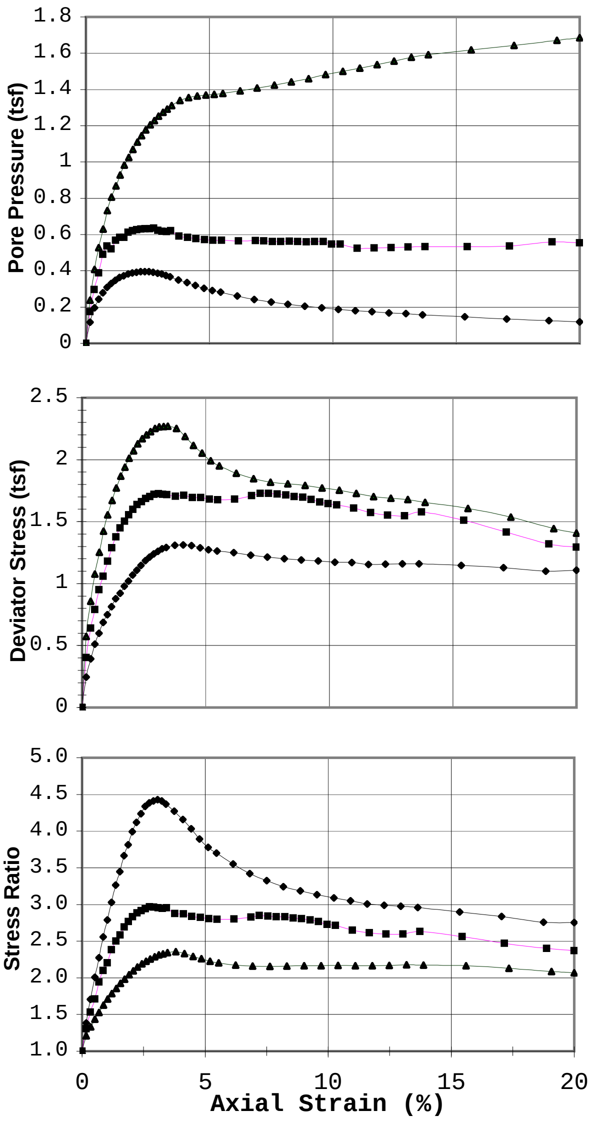
------------	Effective ':	16.0 °	c'=	0.38 (tsf)
________	Total :	10.2 °	c=	0.41 (tsf)

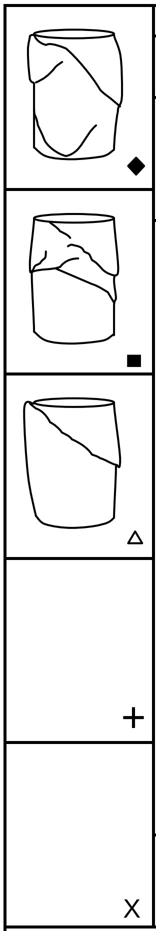
SOIL ENGINEERING TESTING, INC.
2401 W 66th Street Richfield, Minnesota 55423-2031

TRIAXIAL TEST ASTM: D 4767

Job No. **9288**
Date: **4/10/14**

Project: **Wild Rice River Structure**
Boring #: **14-207MU** Sample #: **5** Type: **5T** Depth (ft): **46-48**
Soil Type: **Fat Clay (CH)**





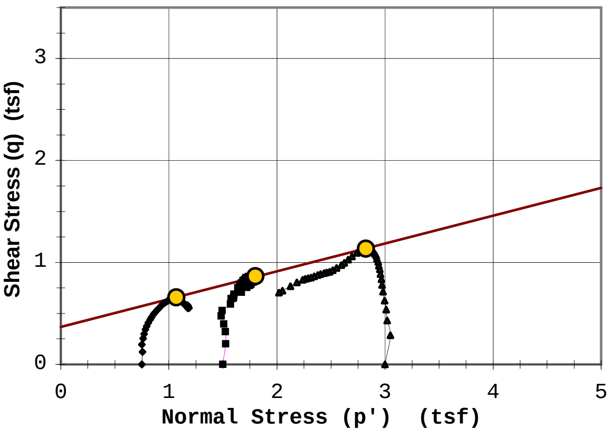
Failure Criterion:	**Max. Deviator Stress**
Angle of internal friction, ' =	**15.8 °**
Apparent Cohesion, **c' =**	**0.38 (tsf)**

Test Date:	3/25/14	Liquid Limit:	84.0
Test Type:	CU w/pp	Plastic Limit:	24.8
Strain Rate (in/min):	0.014665	Plasticity Index:	59.2
Strain Rate (%/min):	0.500	Spec. Gravity (Assumed):	2.73

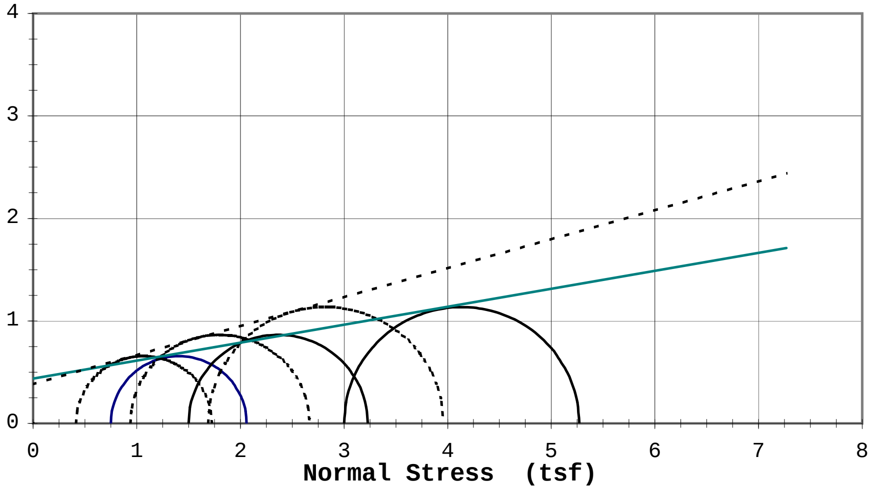
Before Consolidation	**A**	**B**	**C**	**D**	**E**
Diameter (in)	1.44	1.44	1.44		
Height (in)	2.94	2.94	2.94		
Water Content (%)	48.5	44.7	48.4		
Dry Density (pcf)	73.0	76.4	73.2		
Void Ratio	1.34	1.23	1.33		
After Consolidation					
Diameter (in)	1.44	1.43	1.41		
Height (in)	2.93	2.91	2.88		
Water Content (%)	48.0	42.8	43.5		
Dry Density (pcf)	73.8	78.6	77.9		
Void Ratio	1.31	1.17	1.19		
Back Pressure (tsf)	8.9	8.9	9.1		
Minor Principal Stress (tsf)	0.75	1.50	3.00		
Max. Deviator Stress (tsf)	1.31	1.73	2.27		
Ultimate Deviator Stress (tsf)	1.11	1.29	1.41		
Deviator Stress at Failure (tsf)	1.31	1.73	2.27		
Max. Pore Pressure Buildup (tsf)	0.40	0.64	1.68		
Pore Pressure Parameter "B"	0.95	0.95	0.95		
Pct. Axial Strain at Failure	4.1	7.6	3.5		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Radial drainage strips applied to trimmed specimen; Saturated, backpressured until "B" response was 0.95 to 1.00; Consolidated; All Drainage valves closed and immediately sheared.



Rupture Envelope at Failure
= 15.3 ° a = 0.4 (tsf)

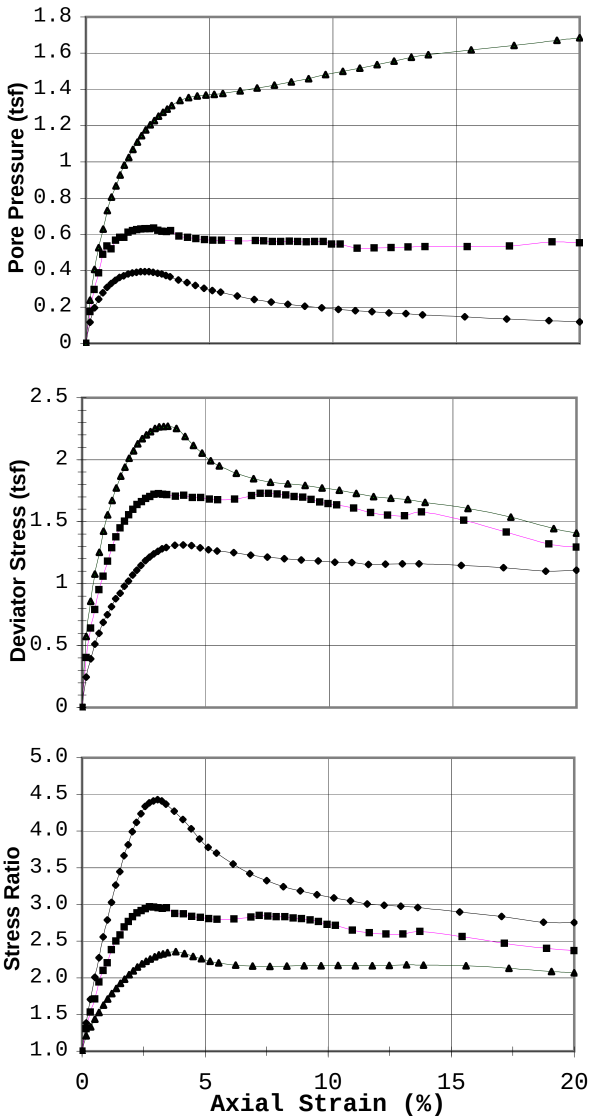


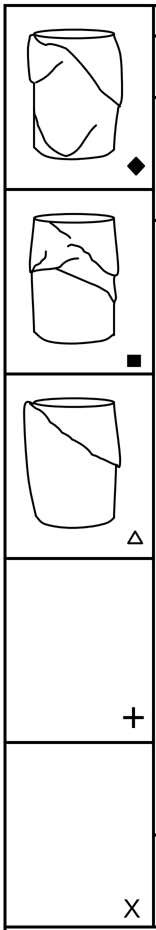
------------	Effective ':	15.8 °	c'=	0.38 (tsf)
________	Total :	10.0 °	c=	0.44 (tsf)

TRIAXIAL TEST ASTM: D 4767

Job No. **9288**
Date: **4/10/14**

Project: **Wild Rice River Structure**
Boring #: **14-207MU** Sample #: **5** Type: **5T** Depth (ft): **46-48**
Soil Type: **Fat Clay (CH)**





Failure Criterion: **Given Strain of: 15%**

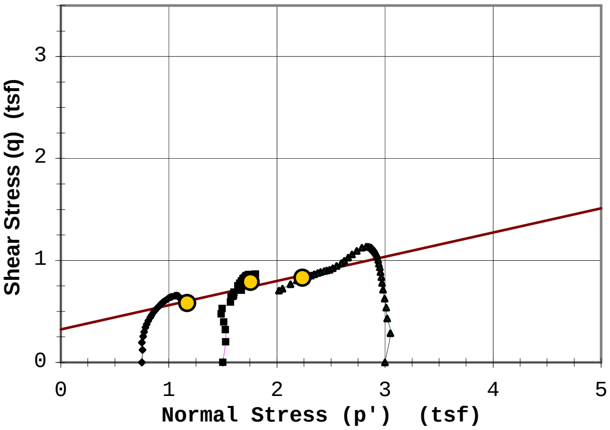
Angle of internal friction, ' = **13.7 °**
Apparent Cohesion, c' = **0.33 (tsf)**

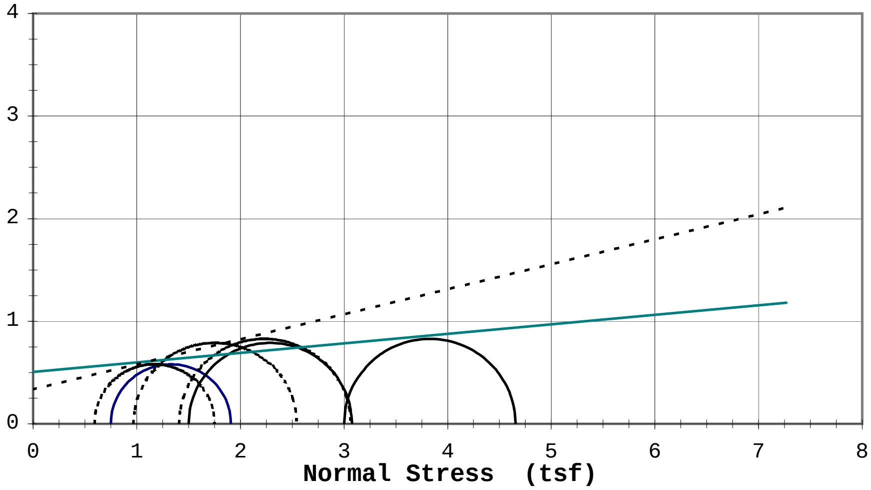
Test Date:	3/25/14	Liquid Limit:	84.0
Test Type:	CU w/pp	Plastic Limit:	24.8
Strain Rate (in/min):	0.014665	Plasticity Index:	59.2
Strain Rate (%/min):	0.500	Spec. Gravity (Assumed):	2.73

Before Consolidation	A	B	C	D	E
Diameter (in)	1.44	1.44	1.44		
Height (in)	2.94	2.94	2.94		
Water Content (%)	48.5	44.7	48.4		
Dry Density (pcf)	73.0	76.4	73.2		
Void Ratio	1.34	1.23	1.33		
After Consolidation					
Diameter (in)	1.44	1.43	1.41		
Height (in)	2.93	2.91	2.88		
Water Content (%)	48.0	42.8	43.5		
Dry Density (pcf)	73.8	78.6	77.9		
Void Ratio	1.31	1.17	1.19		
Back Pressure (tsf)	8.9	8.9	9.1		
Minor Principal Stress (tsf)	0.75	1.50	3.00		
Max. Deviator Stress (tsf)	1.31	1.73	2.27		
Ultimate Deviator Stress (tsf)	1.11	1.29	1.41		
Deviator Stress at Failure (tsf)	1.16	1.58	1.66		
Max. Pore Pressure Buildup (tsf)	0.40	0.64	1.68		
Pore Pressure Parameter "B"	0.95	0.95	0.95		
Pct. Axial Strain at Failure	15.0	15.0	15.0		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Radial drainage strips applied to trimmed specimen; Saturated, backpressured until "B" response was 0.95 to 1.00; Consolidated; All Drainage valves closed and immediately sheared.





Rupture Envelope at Failure	
= 13.4 °	a = 0.3 (tsf)

------------	Effective ':	13.7 °	c'=	0.33 (tsf)
________	Total :	5.3 °	c=	0.50 (tsf)

Triaxial Data

Job: **9288**

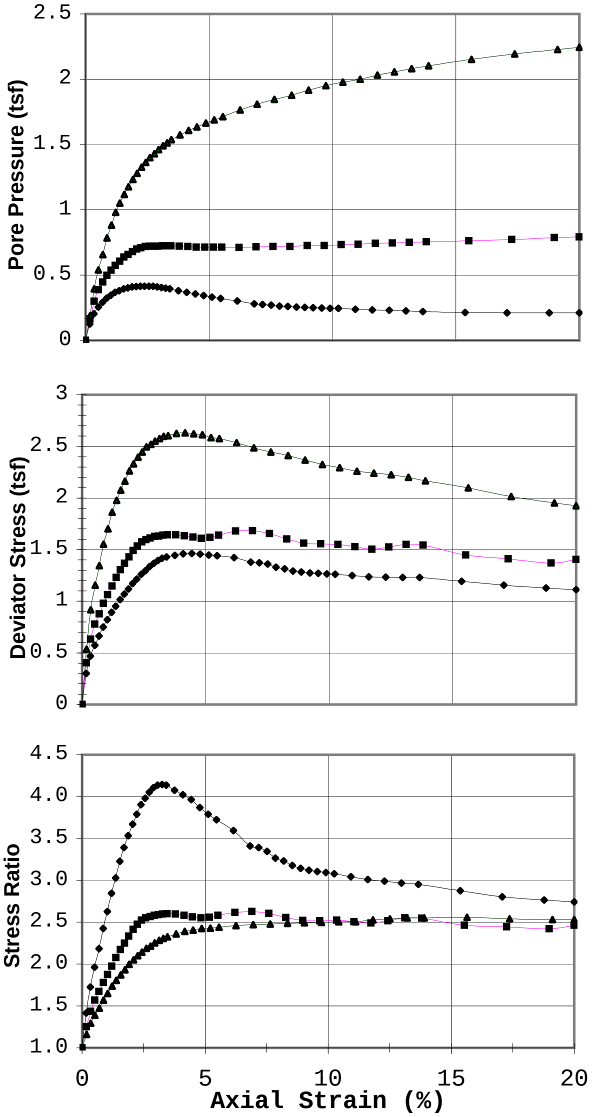
Boring: 14-207MU Sample: 5 Depth: 46-48 Date: 4/10/14

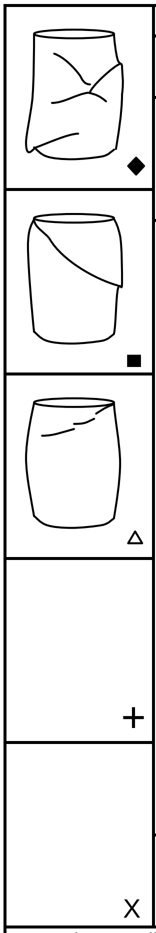
Sample 1			**Sample 2**			**Sample 3**			**Sample 4**			**Sample 5**		
Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00						
0.17	0.24	0.12	0.17	0.40	0.17	0.17	0.57	0.24						
0.34	0.39	0.20	0.34	0.64	0.30	0.35	0.86	0.41						
0.51	0.51	0.24	0.51	0.79	0.39	0.52	1.08	0.53						
0.69	0.60	0.28	0.69	0.95	0.49	0.69	1.25	0.63						
0.86	0.69	0.31	0.86	1.06	0.54	0.87	1.42	0.73						
1.03	0.75	0.33	1.03	1.18	0.52	1.04	1.56	0.81						
1.19	0.81	0.35	1.20	1.29	0.57	1.21	1.67	0.87						
1.36	0.88	0.36	1.37	1.38	0.58	1.39	1.77	0.93						
1.53	0.92	0.37	1.54	1.45	0.58	1.56	1.87	0.98						
1.70	0.98	0.38	1.72	1.50	0.61	1.73	1.94	1.03						
1.88	1.02	0.39	1.89	1.55	0.62	1.91	2.01	1.07						
2.05	1.07	0.39	2.06	1.60	0.63	2.08	2.07	1.11						
2.22	1.11	0.39	2.23	1.64	0.63	2.25	2.13	1.15						
2.39	1.15	0.40	2.40	1.66	0.63	2.43	2.17	1.18						
2.56	1.19	0.39	2.57	1.69	0.63	2.60	2.20	1.20						
2.73	1.21	0.39	2.75	1.70	0.64	2.77	2.23	1.23						
2.90	1.24	0.39	2.92	1.72	0.62	2.95	2.25	1.25						
3.07	1.26	0.38	3.09	1.72	0.62	3.12	2.26	1.27						
3.24	1.28	0.37	3.26	1.72	0.62	3.30	2.27	1.29						
3.41	1.29	0.37	3.43	1.72	0.62	3.47	2.27	1.31						
3.75	1.31	0.35	3.78	1.71	0.59	3.82	2.25	1.34						
4.09	1.31	0.33	4.12	1.71	0.58	4.16	2.19	1.36						
4.43	1.31	0.32	4.46	1.69	0.58	4.51	2.12	1.36						
4.78	1.29	0.30	4.81	1.69	0.57	4.86	2.05	1.37						
5.12	1.27	0.29	5.15	1.68	0.57	5.20	1.99	1.37						
5.46	1.26	0.28	5.49	1.67	0.57	5.55	1.95	1.38						
6.14	1.25	0.26	6.18	1.68	0.57	6.24	1.89	1.39						
6.82	1.23	0.24	6.87	1.71	0.57	6.94	1.85	1.41						
7.50	1.21	0.23	7.21	1.73	0.56	7.63	1.82	1.42						
8.18	1.20	0.22	7.55	1.73	0.56	8.32	1.81	1.44						
8.86	1.19	0.21	7.90	1.72	0.56	9.02	1.79	1.46						
9.55	1.18	0.20	8.24	1.72	0.56	9.71	1.77	1.48						
10.23	1.17	0.19	8.58	1.70	0.56	10.41	1.75	1.50						
10.91	1.17	0.18	8.93	1.70	0.56	11.10	1.73	1.52						
11.59	1.16	0.17	9.27	1.68	0.56	11.79	1.70	1.54						
12.27	1.16	0.17	9.61	1.66	0.56	12.49	1.69	1.56						
12.96	1.16	0.16	9.96	1.65	0.55	13.18	1.68	1.58						
13.64	1.16	0.16	10.30	1.63	0.55	13.87	1.66	1.59						
15.34	1.15	0.15	10.99	1.61	0.52	15.61	1.61	1.62						
17.05	1.13	0.13	11.67	1.57	0.53	17.34	1.54	1.64						
18.75	1.10	0.12	12.36	1.55	0.53	19.08	1.44	1.67						
20.00	1.11	0.12	13.05	1.55	0.53	20.00	1.41	1.68						
			13.73	1.58	0.53									
			15.45	1.51	0.53									
			17.17	1.42	0.54									
			18.88	1.32	0.56									
			20.00	1.29	0.55									

TRIAXIAL TEST ASTM: D 4767

Job No. **9288**
Date: **4/9/14**

Project: **Wild Rice River Structure**
Boring #: **14-207MU** Sample #: **7** Type: **5T** Depth (ft): **63.2-65.2**
Soil Type: **Fat Clay (CH)**





Failure Criterion: **Max. Stress Ratio**

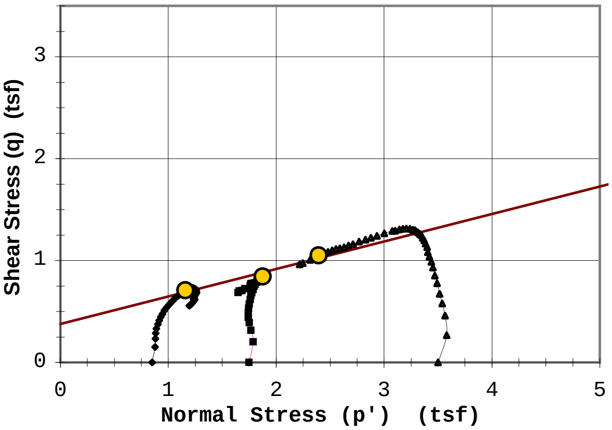
Angle of internal friction, ' = **15.7 °**
Apparent Cohesion, c' = **0.39 (tsf)**

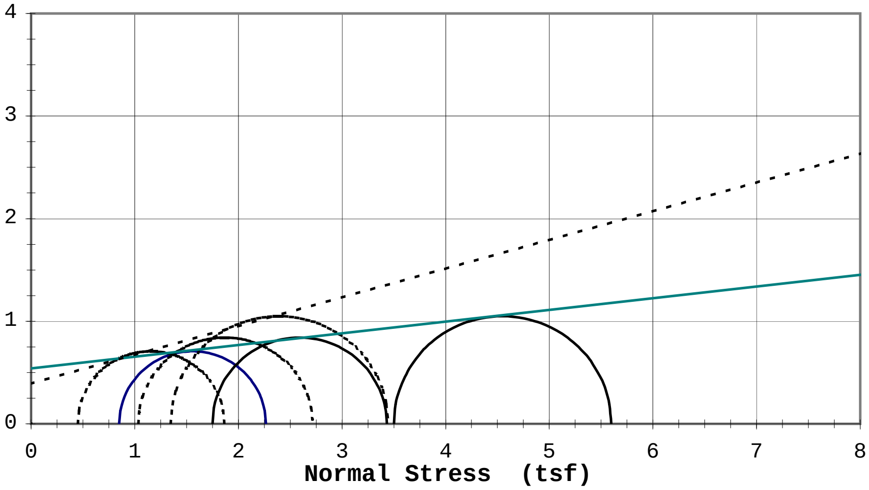
Test Date:	3/25/14	Liquid Limit:	76.4
Test Type:	CU w/pp	Plastic Limit:	23.7
Strain Rate (in/min):	0.001465	Plasticity Index:	52.7
Strain Rate (%/min):	0.050	Spec. Gravity (Assumed):	2.71

Before Consolidation	A	B	C	D	E
Diameter (in)	1.44	1.44	1.44		
Height (in)	2.95	2.94	2.94		
Water Content (%)	44.9	45.0	43.5		
Dry Density (pcf)	75.8	76.0	77.3		
Void Ratio	1.23	1.23	1.19		
After Consolidation					
Diameter (in)	1.43	1.42	1.41		
Height (in)	2.93	2.90	2.88		
Water Content (%)	44.0	41.6	38.7		
Dry Density (pcf)	77.1	79.6	82.5		
Void Ratio	1.19	1.13	1.05		
Back Pressure (tsf)	5.8	5.8	5.8		
Minor Principal Stress (tsf)	0.85	1.75	3.50		
Max. Deviator Stress (tsf)	1.46	1.68	2.63		
Ultimate Deviator Stress (tsf)	1.11	1.40	1.93		
Deviator Stress at Failure (tsf)	1.42	1.68	2.10		
Max. Pore Pressure Buildup (tsf)	0.42	0.79	2.24		
Pore Pressure Parameter "B"	0.95	0.95	0.95		
Pct. Axial Strain at Failure	3.2	6.9	15.6		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Radial drainage strips applied to trimmed specimen; Saturated, backpressured until "B" response was 0.95 to 1.00; Consolidated; All Drainage valves closed and immediately sheared.





Rupture Envelope at Failure
= 15.1 ° a = 0.4 (tsf)

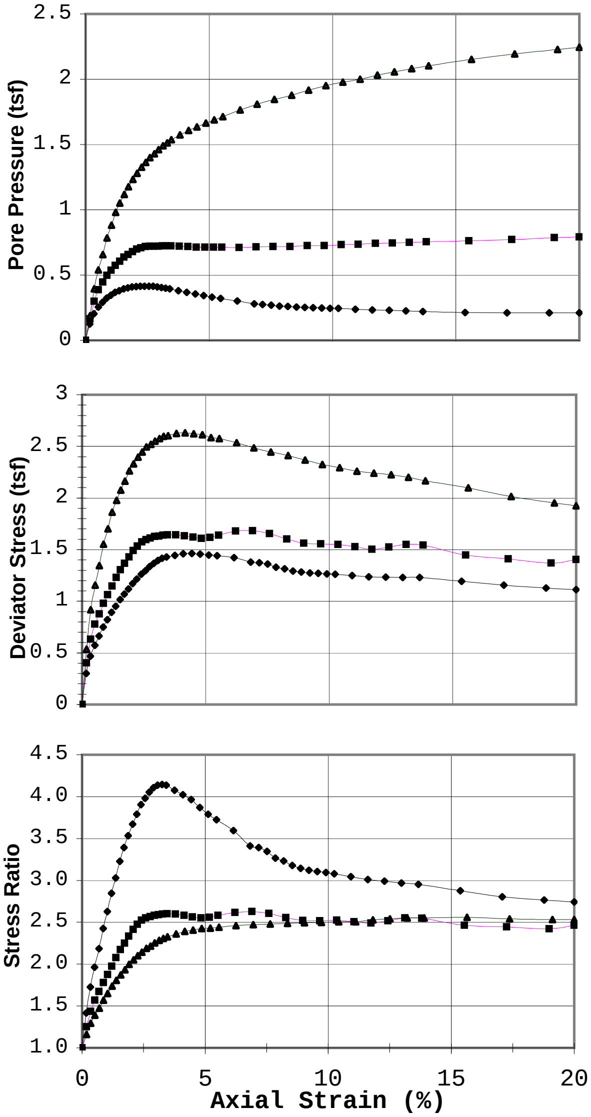
------------	Effective ':	15.7 °	c'=	0.39	(tsf)
________	Total :	6.5 °	c=	0.54	(tsf)

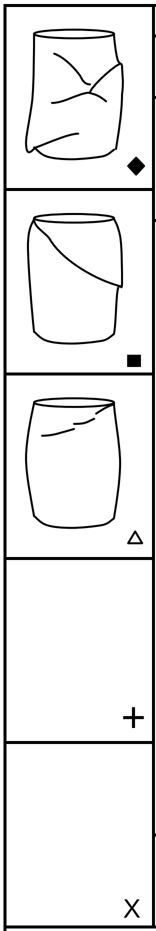
SOIL ENGINEERING TESTING, INC.
2401 W 66th Street Richfield, Minnesota 55423-2031

TRIAXIAL TEST ASTM: D 4767

Job No. **9288**
Date: **4/9/14**

Project: **Wild Rice River Structure**
Boring #: **14-207MU** Sample #: **7** Type: **5T** Depth (ft): **63.2-65.2**
Soil Type: **Fat Clay (CH)**





Failure Criterion: **Max. Deviator Stress**

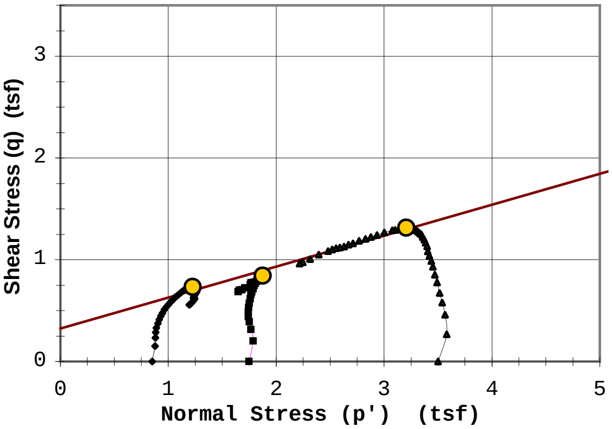
Angle of internal friction, ' = **17.7 °**
Apparent Cohesion, c' = **0.34 (tsf)**

Test Date:	3/25/14	Liquid Limit:	76.4
Test Type:	CU w/pp	Plastic Limit:	23.7
Strain Rate (in/min):	0.001465	Plasticity Index:	52.7
Strain Rate (%/min):	0.050	Spec. Gravity (Assumed):	2.71

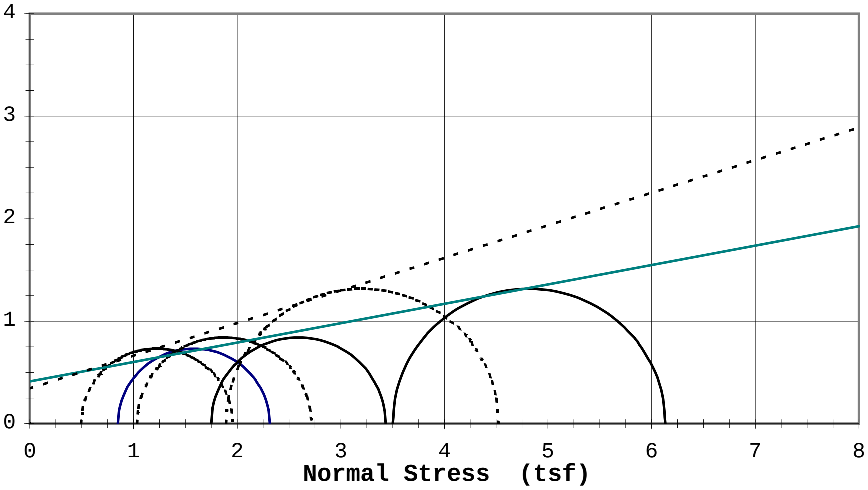
Before Consolidation	A	B	C	D	E
Diameter (in)	1.44	1.44	1.44		
Height (in)	2.95	2.94	2.94		
Water Content (%)	44.9	45.0	43.5		
Dry Density (pcf)	75.8	76.0	77.3		
Void Ratio	1.23	1.23	1.19		
After Consolidation					
Diameter (in)	1.43	1.42	1.41		
Height (in)	2.93	2.90	2.88		
Water Content (%)	44.0	41.6	38.7		
Dry Density (pcf)	77.1	79.6	82.5		
Void Ratio	1.19	1.13	1.05		
Back Pressure (tsf)	5.8	5.8	5.8		
Minor Principal Stress (tsf)	0.85	1.75	3.50		
Max. Deviator Stress (tsf)	1.46	1.68	2.63		
Ultimate Deviator Stress (tsf)	1.11	1.40	1.93		
Deviator Stress at Failure (tsf)	1.46	1.68	2.63		
Max. Pore Pressure Buildup (tsf)	0.42	0.79	2.24		
Pore Pressure Parameter "B"	0.95	0.95	0.95		
Pct. Axial Strain at Failure	4.4	6.9	4.2		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Radial drainage strips applied to trimmed specimen; Saturated, backpressured until "B" response was 0.95 to 1.00; Consolidated; All Drainage valves closed and immediately sheared.



Rupture Envelope at Failure
= 16.9 ° a = 0.3 (tsf)

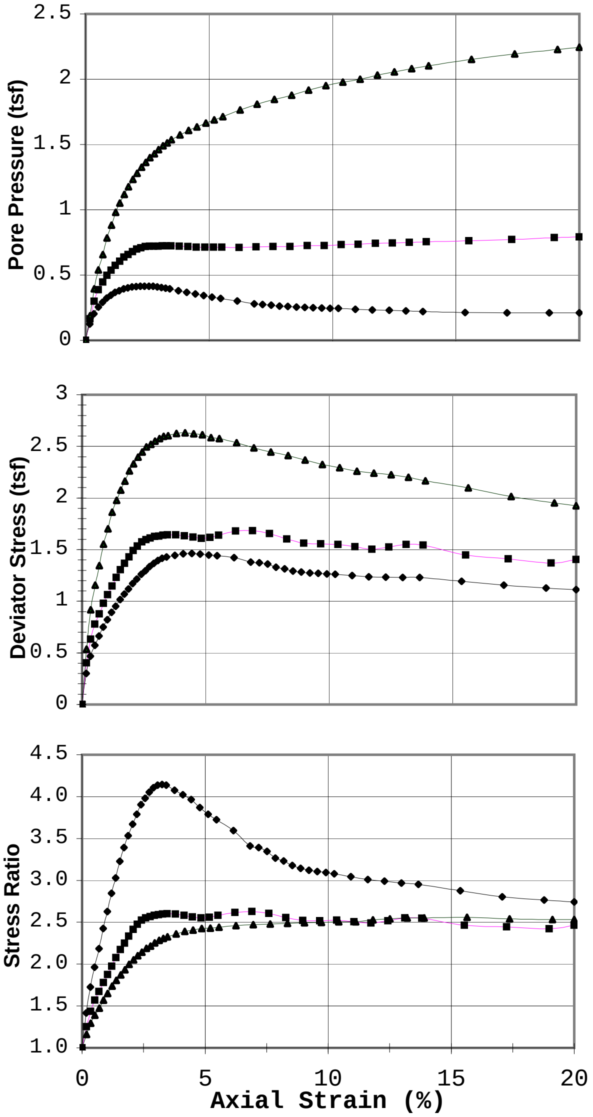


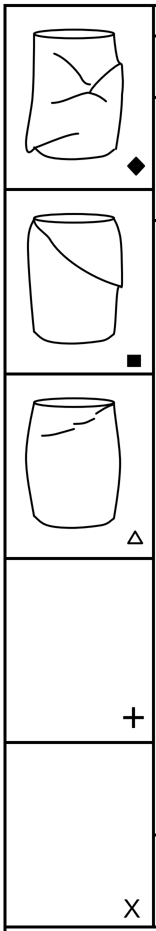
------------	Effective ':	17.7 °	c'=	0.34 (tsf)
________	Total :	10.7 °	c=	0.41 (tsf)

TRIAXIAL TEST ASTM: D 4767

Job No. **9288**
Date: **4/9/14**

Project: **Wild Rice River Structure**
Boring #: **14-207MU** Sample #: **7** Type: **5T** Depth (ft): **63.2-65.2**
Soil Type: **Fat Clay (CH)**





Failure Criterion: **Given Strain of: 15%**

Angle of internal friction, ' = **22.5 °**

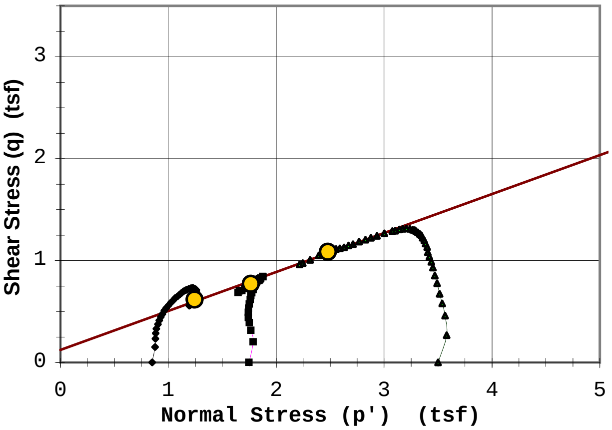
Apparent Cohesion, c' = **0.13 (tsf)**

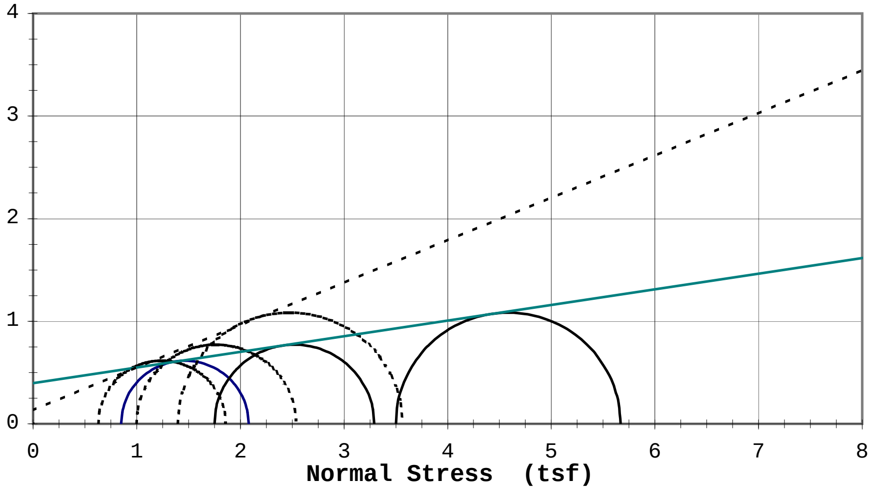
Test Date:	3/25/14	Liquid Limit:	76.4
Test Type:	CU w/pp	Plastic Limit:	23.7
Strain Rate (in/min):	0.001465	Plasticity Index:	52.7
Strain Rate (%/min):	0.050	Spec. Gravity (Assumed):	2.71

Before Consolidation	A	B	C	D	E
Diameter (in)	1.44	1.44	1.44		
Height (in)	2.95	2.94	2.94		
Water Content (%)	44.9	45.0	43.5		
Dry Density (pcf)	75.8	76.0	77.3		
Void Ratio	1.23	1.23	1.19		
After Consolidation					
Diameter (in)	1.43	1.42	1.41		
Height (in)	2.93	2.90	2.88		
Water Content (%)	44.0	41.6	38.7		
Dry Density (pcf)	77.1	79.6	82.5		
Void Ratio	1.19	1.13	1.05		
Back Pressure (tsf)	5.8	5.8	5.8		
Minor Principal Stress (tsf)	0.85	1.75	3.50		
Max. Deviator Stress (tsf)	1.46	1.68	2.63		
Ultimate Deviator Stress (tsf)	1.11	1.40	1.93		
Deviator Stress at Failure (tsf)	1.23	1.54	2.17		
Max. Pore Pressure Buildup (tsf)	0.42	0.79	2.24		
Pore Pressure Parameter "B"	0.95	0.95	0.95		
Pct. Axial Strain at Failure	15.0	15.0	15.0		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Radial drainage strips applied to trimmed specimen; Saturated, backpressured until "B" response was 0.95 to 1.00; Consolidated; All Drainage valves closed and immediately sheared.





Rupture Envelope at Failure
= 20.9 ° a = 0.1 (tsf)

------------	Effective ':	22.5 °	c'=	0.13 (tsf)
________	Total :	8.7 °	c=	0.40 (tsf)

SOIL ENGINEERING TESTING, INC.

2401 W 66th Street Richfield, Minnesota 55423-2031

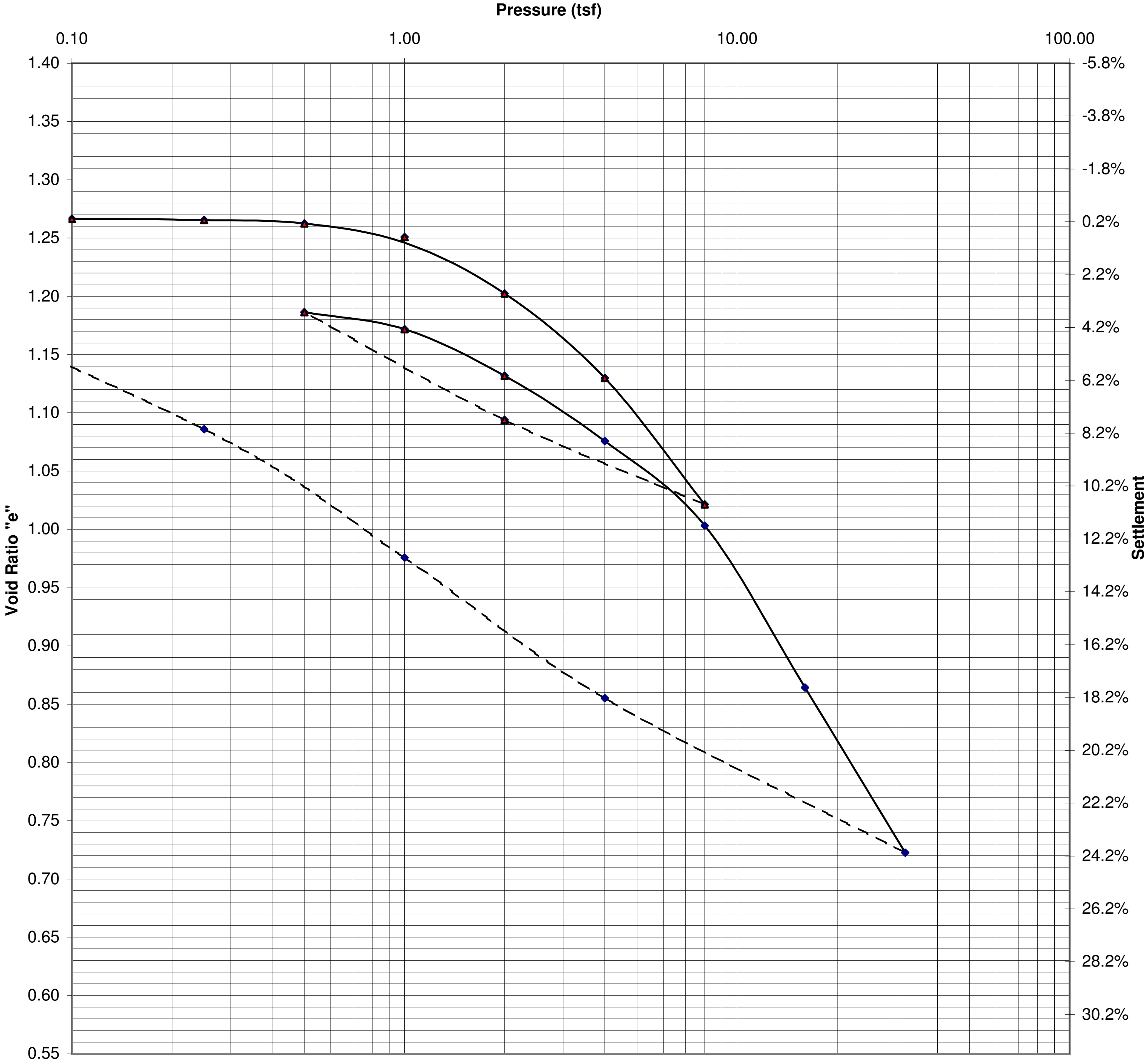
Triaxial Data

Job: **9288**

Boring: 14-207MU
Sample: 7
Depth: 63.2-65.2
Date: 4/9/14

Sample 1			Sample 2			Sample 3			Sample 4			Sample 5		
Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)	Strain (%)	Deviator Stress (tsf)	Pore Pressure (tsf)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00						
0.17	0.30	0.12	0.18	0.40	0.16	0.18	0.54	0.19						
0.34	0.47	0.20	0.35	0.63	0.30	0.35	0.92	0.39						
0.51	0.57	0.25	0.52	0.78	0.39	0.52	1.16	0.54						
0.68	0.66	0.29	0.69	0.88	0.44	0.70	1.35	0.66						
0.85	0.75	0.32	0.87	0.98	0.50	0.87	1.55	0.78						
1.02	0.82	0.35	1.04	1.06	0.54	1.05	1.70	0.88						
1.20	0.89	0.37	1.21	1.15	0.57	1.22	1.86	0.98						
1.37	0.95	0.38	1.38	1.23	0.61	1.39	1.98	1.05						
1.54	1.02	0.39	1.56	1.31	0.64	1.57	2.08	1.12						
1.71	1.07	0.40	1.73	1.37	0.66	1.74	2.16	1.18						
1.88	1.12	0.41	1.90	1.43	0.68	1.91	2.26	1.23						
2.05	1.17	0.41	2.07	1.49	0.70	2.09	2.33	1.28						
2.22	1.22	0.41	2.25	1.54	0.71	2.26	2.39	1.33						
2.39	1.26	0.42	2.42	1.57	0.72	2.44	2.44	1.36						
2.56	1.30	0.41	2.59	1.60	0.72	2.61	2.49	1.40						
2.73	1.33	0.41	2.76	1.61	0.72	2.78	2.52	1.43						
2.90	1.37	0.41	2.94	1.63	0.72	2.96	2.55	1.46						
3.07	1.39	0.41	3.11	1.63	0.72	3.13	2.58	1.49						
3.24	1.42	0.40	3.28	1.64	0.72	3.30	2.60	1.51						
3.42	1.43	0.39	3.45	1.64	0.72	3.48	2.60	1.54						
3.76	1.44	0.38	3.80	1.64	0.72	3.83	2.62	1.57						
4.10	1.46	0.37	4.15	1.63	0.72	4.17	2.63	1.61						
4.44	1.46	0.36	4.49	1.62	0.71	4.52	2.62	1.64						
4.78	1.46	0.34	4.84	1.61	0.71	4.87	2.61	1.67						
5.12	1.45	0.33	5.18	1.62	0.71	5.22	2.59	1.69						
5.46	1.44	0.32	5.53	1.64	0.71	5.56	2.58	1.71						
6.15	1.42	0.30	6.22	1.68	0.71	6.26	2.53	1.76						
6.83	1.38	0.28	6.91	1.68	0.72	6.95	2.49	1.81						
7.17	1.37	0.28	7.60	1.65	0.72	7.65	2.45	1.85						
7.51	1.36	0.27	8.29	1.60	0.72	8.34	2.41	1.88						
7.86	1.33	0.26	8.98	1.56	0.72	9.03	2.37	1.92						
8.20	1.31	0.26	9.67	1.56	0.72	9.73	2.32	1.95						
8.54	1.29	0.26	10.36	1.55	0.73	10.43	2.29	1.98						
8.88	1.28	0.25	11.05	1.53	0.74	11.12	2.26	2.00						
9.22	1.27	0.25	11.74	1.50	0.74	11.82	2.24	2.03						
9.56	1.27	0.25	12.43	1.52	0.75	12.51	2.22	2.06						
9.90	1.26	0.25	13.12	1.55	0.75	13.21	2.20	2.08						
10.25	1.26	0.24	13.81	1.54	0.75	13.90	2.17	2.10						
10.93	1.25	0.24	15.53	1.45	0.76	15.64	2.10	2.15						
11.61	1.24	0.23	17.26	1.41	0.77	17.38	2.01	2.19						
12.29	1.23	0.23	18.99	1.37	0.79	19.11	1.95	2.23						
12.98	1.23	0.22	20.00	1.40	0.79	20.00	1.93	2.24						
13.66	1.23	0.22												
15.37	1.19	0.21												
17.08	1.16	0.21												
18.78	1.13	0.21												
20.00	1.11	0.21												

Void Ratio and % Settlement vs. Log of Pressure



Project: Wild Rice River Structure				Date: 4/8/14
Sample #: 1	Boring #: 14-207MU	Depth ft: 11-13		Job #: 9288
Soil Type: Fat Clay (CH)				
Initial W/C (%): 44.9	Dry Density (pcf): 74.9	LL: 89.5 PL: 22.4 PI: 67.1		Gs: 2.72 (Assumed)
Organic Content (%):	Initial Height (in.): 0.748	Diameter (in.): 2.503	e_o= 1.268	
Preconsolidation Pressure (Pc): 3.7 tsf		Compression Index (Cc): 0.36	Recompression Index (Cr): 0.11	

Remarks: Testing performed in general accordance with ASTM:D2435

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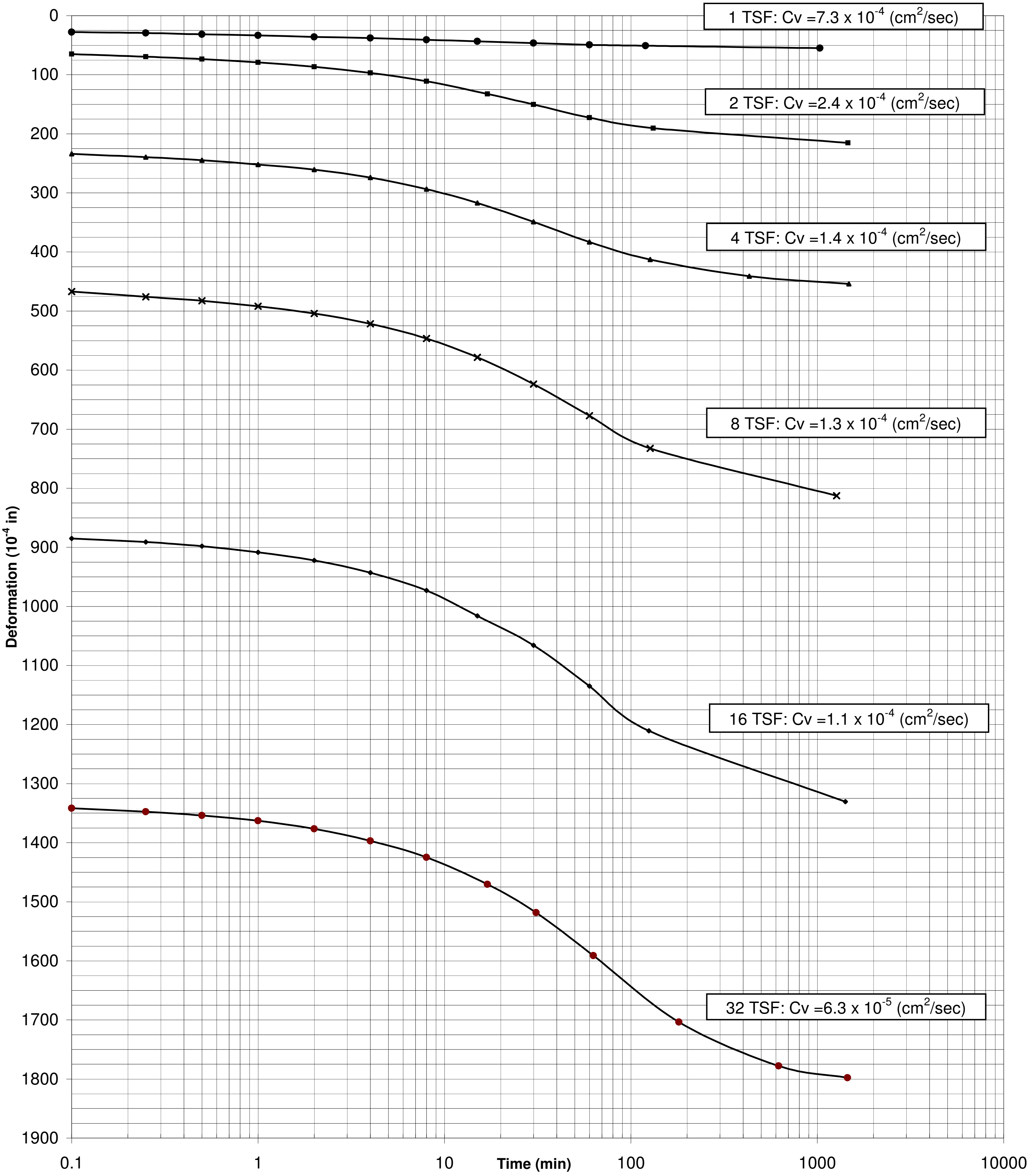
Consolidation Log of Time Curves



Project: Wild Rice River Structure	Date: 4/8/14

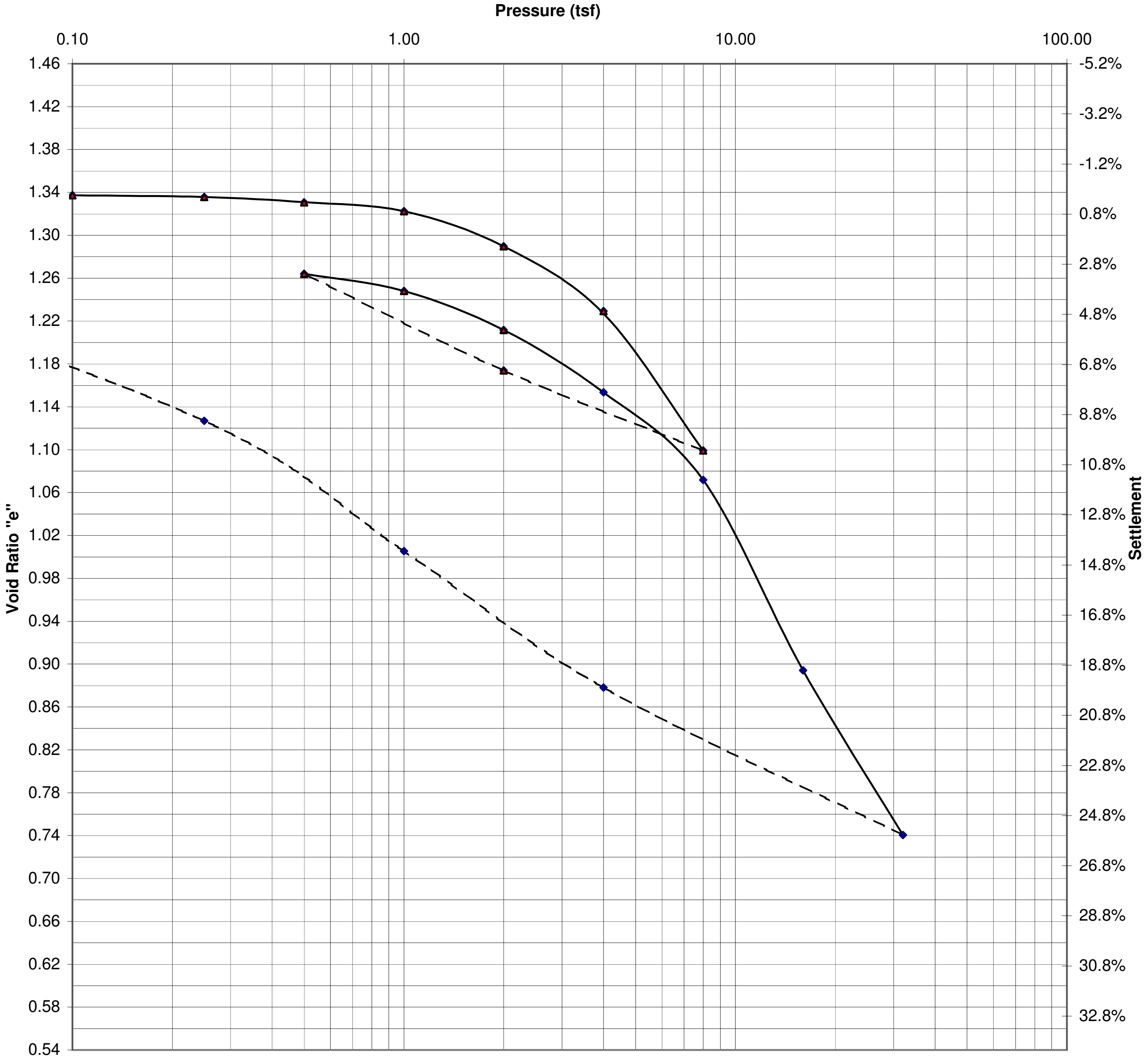
Sample #: 1	Boring #: 14-207MU	Depth ft: 11-13	Job #: 9288

SOIL ENGINEERING TESTING, INC.

2401 W 66th Street

Richfield, Minnesota 55423-2031

Void Ratio and % Settlement vs. Log of Pressure



Project: Wild Rice River Structure

Date: 4/8/14

Sample #: 2

Boring #: 14-207MU

Depth ft: 20.2-22.2

Job #: 9288

Soil Type: Fat Clay (CH)

Initial W/C (%): 47.3

Dry Density (pcf): 73.4

LL: 71.9 PL: 21.8 PI: 50.1

Gs: 2.75 (Assumed)

Organic Content (%):

Initial Height (in.): 0.746

Diameter (in.): 2.506

e_o= 1.338

Preconsolidation Pressure (Pc): 4.5 tsf

Compression Index (Cc): 0.57

Recompression Index (Cr): 0.15

Remarks: Testing performed in general accordance with ASTM:D2435

SOIL ENGINEERING TESTING, INC.

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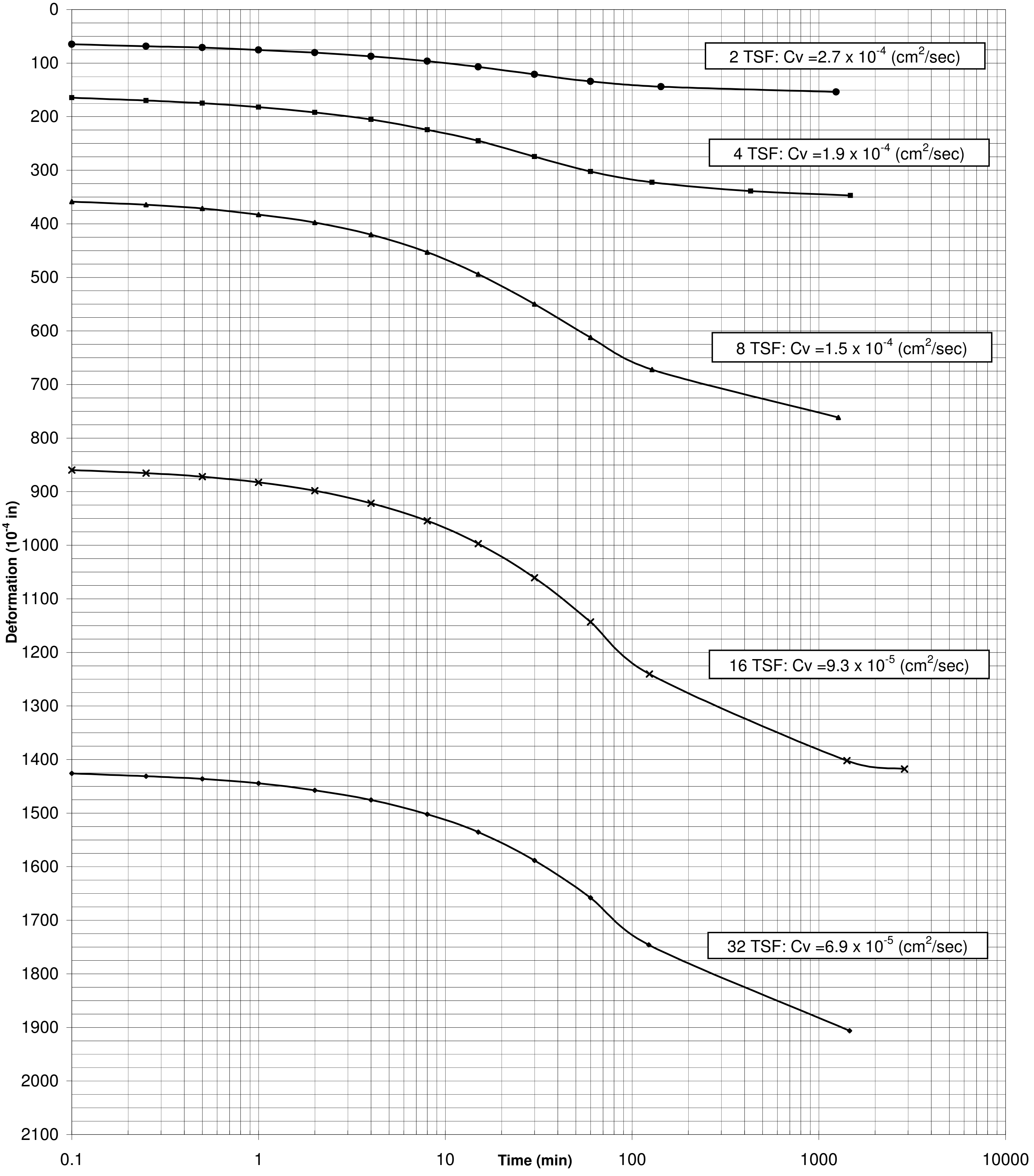
Richfield, Minnesota 55423-2031

Consolidation Log of Time Curves



Project: Wild Rice River Structure			Date: 4/8/14
Sample #: 2	Boring #: 14-207MU	Depth ft: 20.2-22.2	Job #: 9288

SOIL ENGINEERING TESTING, INC.

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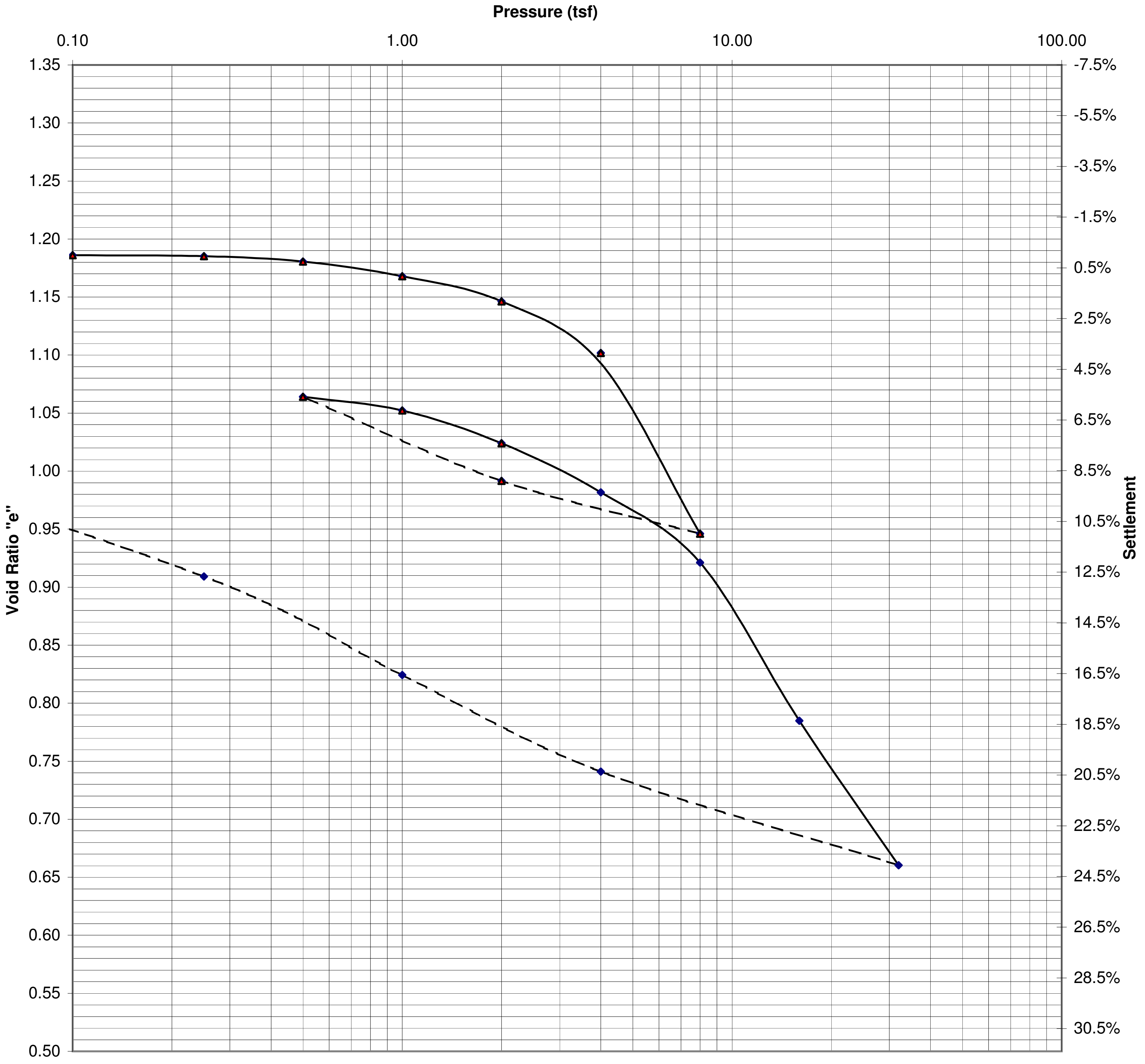
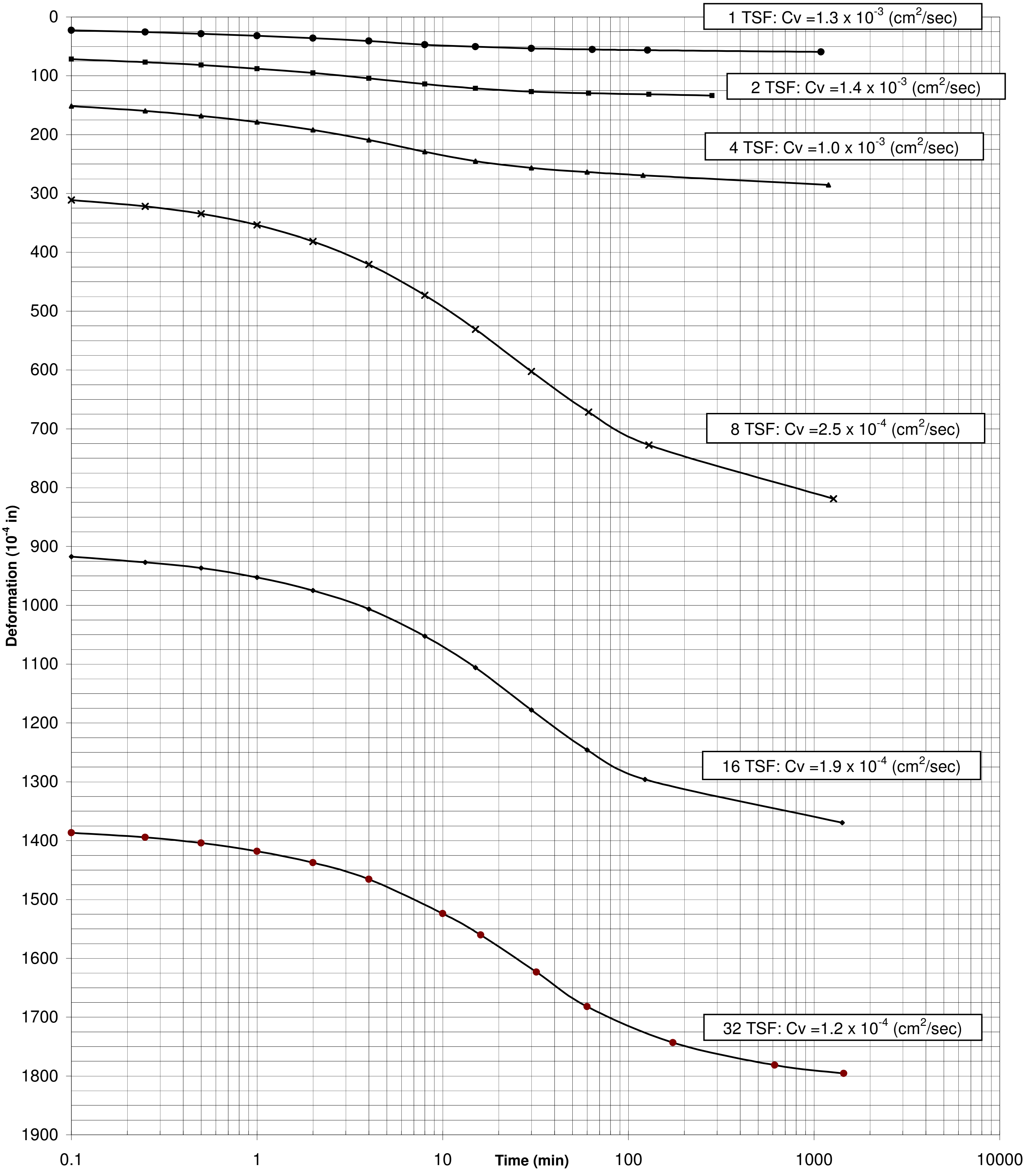
Void Ratio and % Settlement vs. Log of Pressure



Project: Wild Rice River Structure				Date: 4/9/14
Sample #: 3	Boring #: 14-207MU	Depth ft: 31-33		Job #: 9288
Soil Type: Fat Clay (CH)				
Initial W/C (%): 39.6	Dry Density (pcf): 79.1	LL: 71.4 PL: 25.0 PI: 46.4		Gs: 2.77 (Assumed)
Organic Content (%):	Initial Height (in.): 0.747	Diameter (in.): 2.506	e_o= 1.185	
Preconsolidation Pressure (Pc): 3.7 tsf	Compression Index (Cc): 0.49	Recompression Index (Cr): 0.12		

Remarks: Testing performed in general accordance with ASTM:D2435

Consolidation Log of Time Curves



Project: Wild Rice River Structure			Date: 4/9/14
Sample #: 3	Boring #: 14-207MU	Depth ft: 31-33	Job #: 9288

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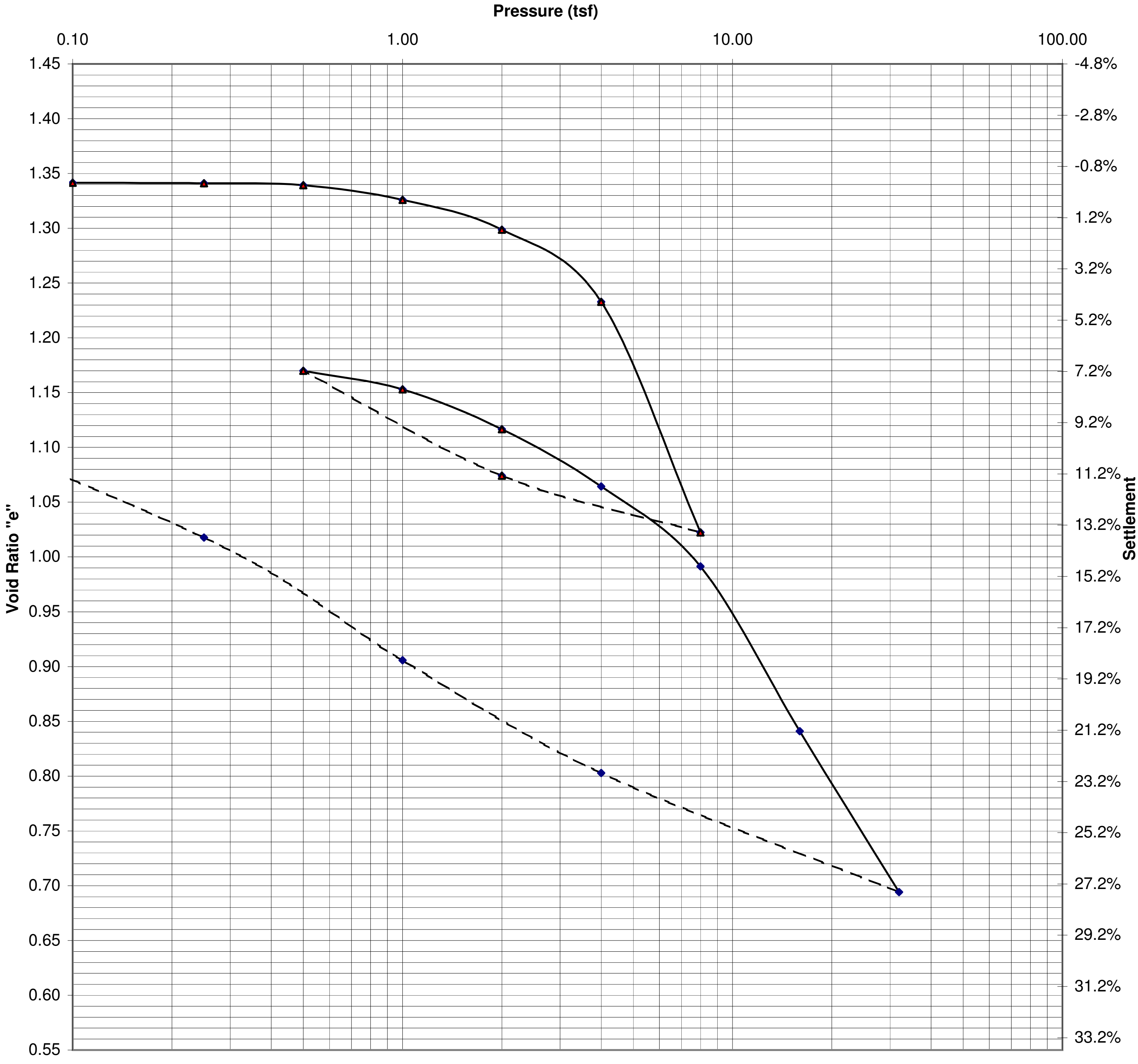
Void Ratio and % Settlement vs. Log of Pressure



Project: Wild Rice River Structure						Date:	4/9/14
Sample #: 5	Boring #: 14-207MU		Depth ft: 46-48			Job #:	9288
Soil Type: Fat Clay (CH)							
Initial W/C (%): 47.9	Dry Density (pcf): 72.9	LL: 84.0	PL: 24.8	PI: 59.2		Gs: 2.73	(Assumed)
Organic Content (%):	Initial Height (in.): 0.748		Diameter (in.): 2.507		e_o= 1.339		
Preconsolidation Pressure (Pc): 3.6 tsf		Compression Index (Cc): 0.54		Recompression Index (Cr): 0.14			

Remarks: Testing performed in general accordance with ASTM:D2435

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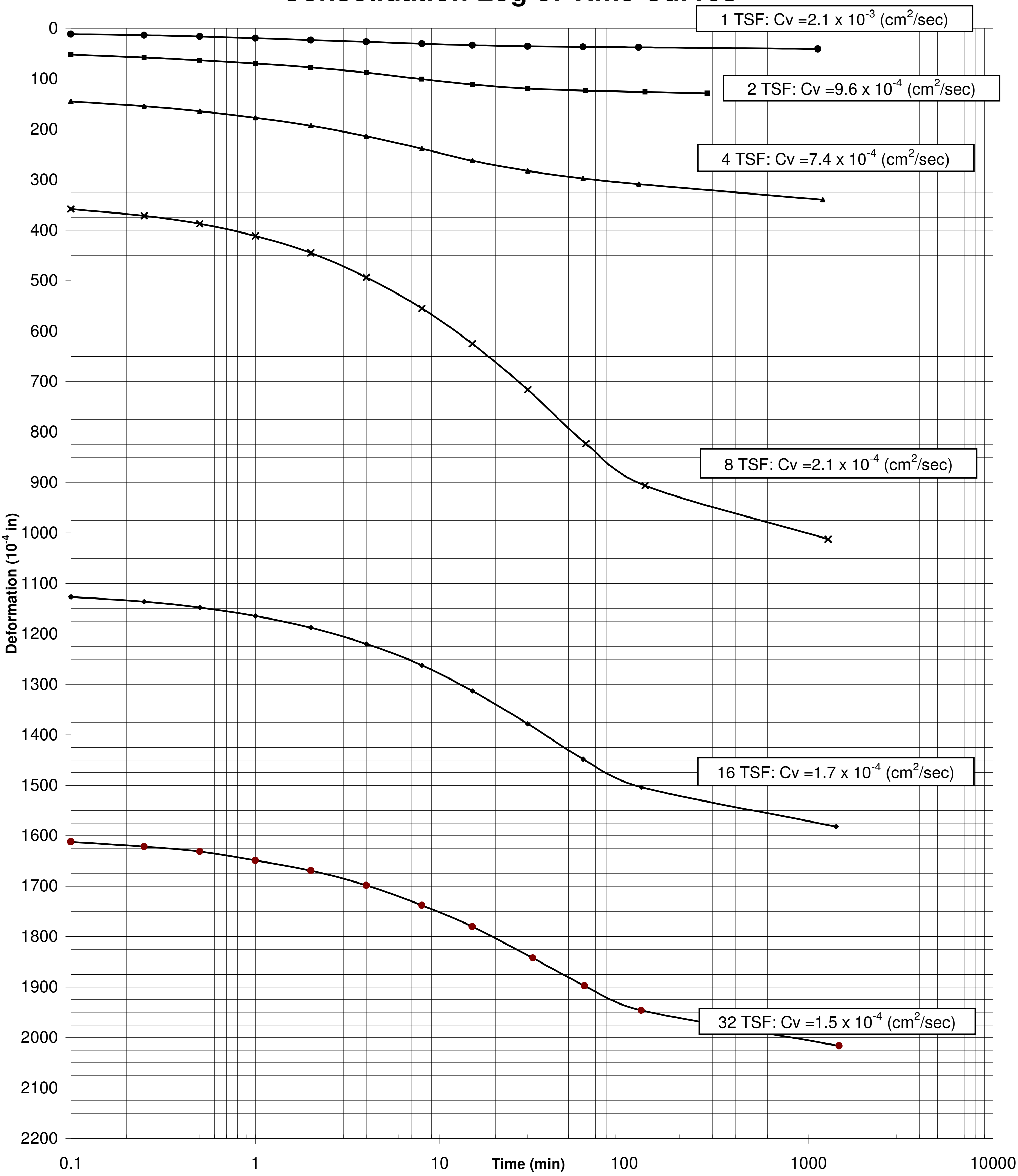
Richfield, Minnesota 55423-2031

Consolidation Log of Time Curves



Project: Wild Rice River Structure			Date: 4/9/14
Sample #: 5	Boring #: 14-207MU	Depth ft: 46-48	Job #: 9288

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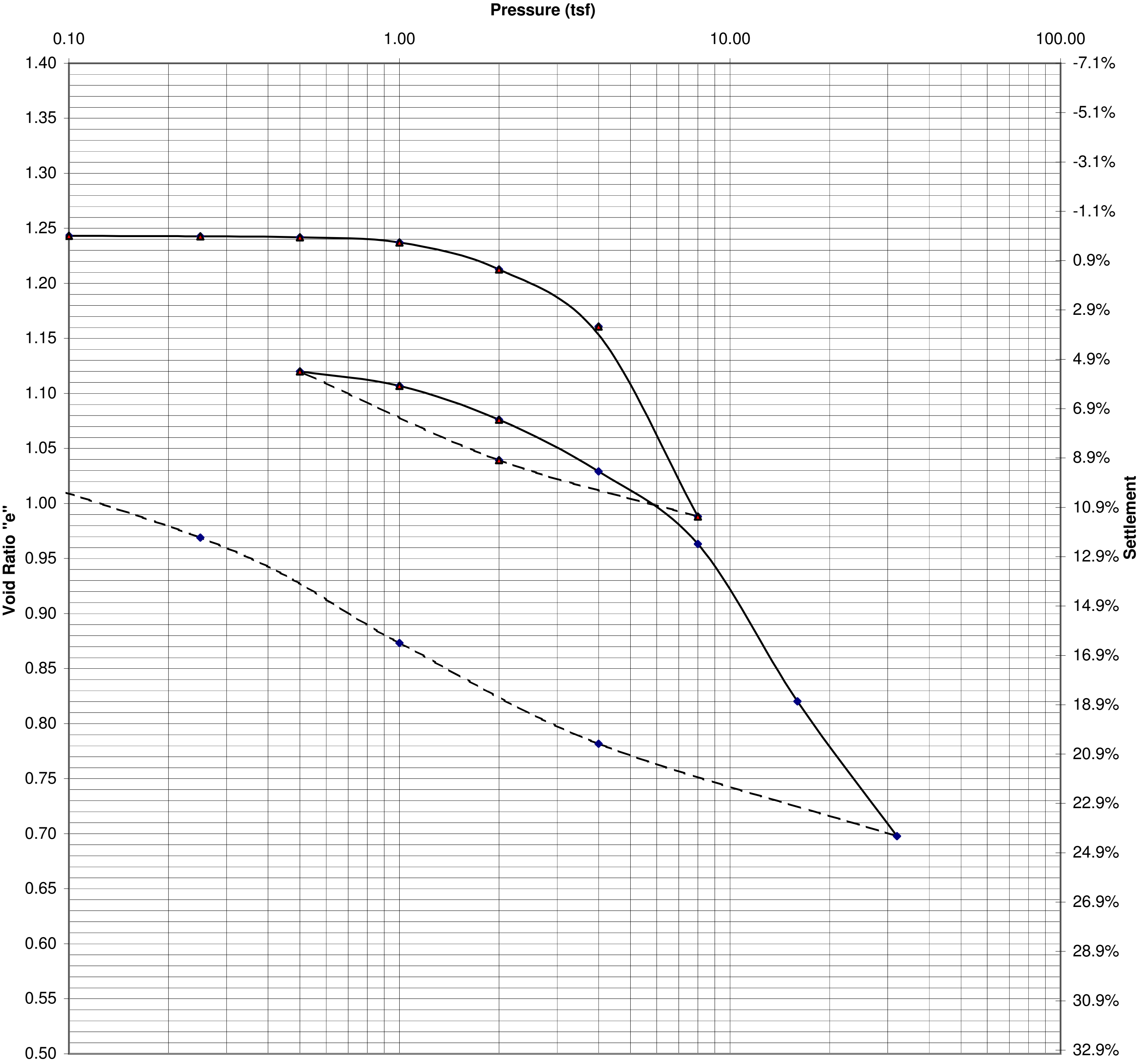
Richfield, Minnesota 55423-2031

Void Ratio and % Settlement vs. Log of Pressure



Project: Wild Rice River Structure			Date: 4/9/14
Sample #: 7	Boring #: 14-207MU	Depth ft: 63.2-65.2	Job #: 9288
Soil Type: Fat Clay (CH)			
Initial W/C (%): 45.2	Dry Density (pcf): 75.5	LL: 76.4 PL: 23.7 PI: 52.7	Gs: 2.71 (Assumed)
Organic Content (%):	Initial Height (in.): 0.759	Diameter (in.): 2.505	e_o= 1.242
Preconsolidation Pressure (Pc): 3.6 tsf	Compression Index (Cc): 0.51	Recompression Index (Cr): 0.13	

Remarks: Testing performed in general accordance with ASTM:D2435

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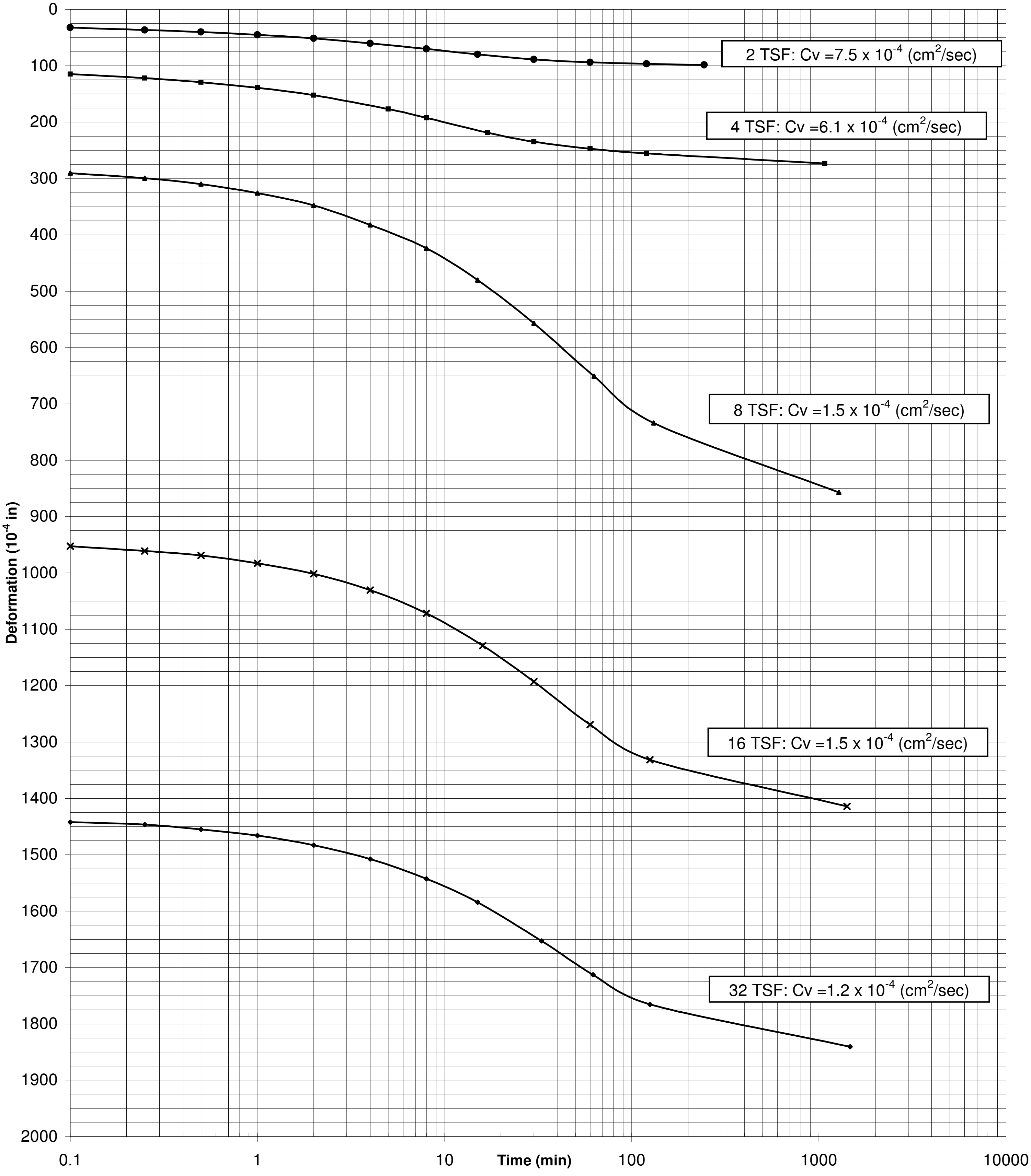
Richfield, Minnesota 55423-2031

Consolidation Log of Time Curves



Project: Wild Rice River Structure			Date: 4/9/14
Sample #: 7	Boring #: 14-207MU	Depth ft: 63.2-65.2	Job #: 9288

SOIL ENGINEERING TESTING, INC.

2401 W 66th Street

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Attachment B

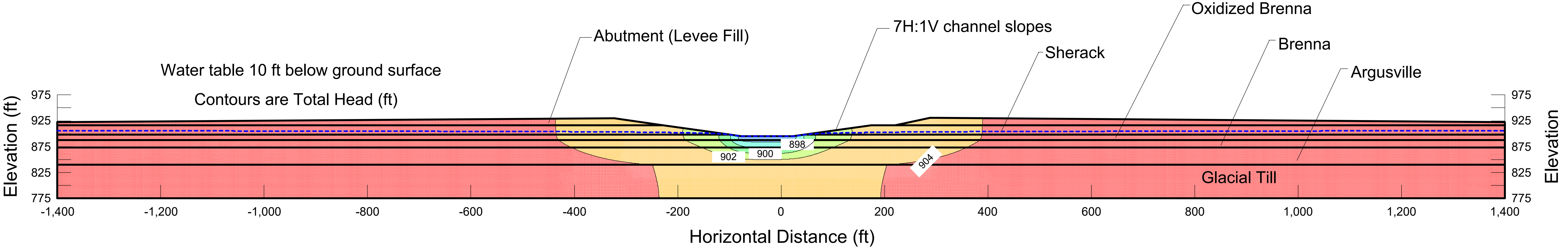
Stability Model Outputs

Analysis 1

Fargo-Moorhead Metropolitan Feasibility Study
Interstate 29 Southbound - Geotech Analysis 1
Case: Steady State Seepage Analysis
Last Saved Date: 6/30/2014

Name: Sherack Formation (Drained)   K-Function: Sherack_Formation (k=1.13E-2 ft/day)   Ky'/Kx' Ratio: 0.25   Vol. WC. Function: Sherack_Formation
Name: Oxidized Brenna (Drained)   K-Function: OX_Brenna_Formation (k=1.4E-3 ft/day)   Ky'/Kx' Ratio: 1   Vol. WC. Function: OX_Brenna_Formation
Name: Brenna (Drained)   K-Function: Brenna_Formation (k=2.8E-4 ft/day)   Ky'/Kx' Ratio: 1   Vol. WC. Function: Brenna_Formation
Name: Argusville Formation (Drained)   K-Function: Argusville_Formation (k=2.8E-4 ft/day)   Ky'/Kx' Ratio: 1   Vol. WC. Function: Argusville_Formation
Name: Glacial Till (Drained)   K-Function: Glacial_Till (k=5.7E-2 ft/day)   Ky'/Kx' Ratio: 0.25   Vol. WC. Function: Glacial_Till
Name: Levee Fill (Ultimate Drained)   K-Function: Sherack_Formation (k=1.13E-2 ft/day)   Ky'/Kx' Ratio: 1   Vol. WC. Function: Sherack_Formation

Main Diversion Channel
Interstate 29 Southbound



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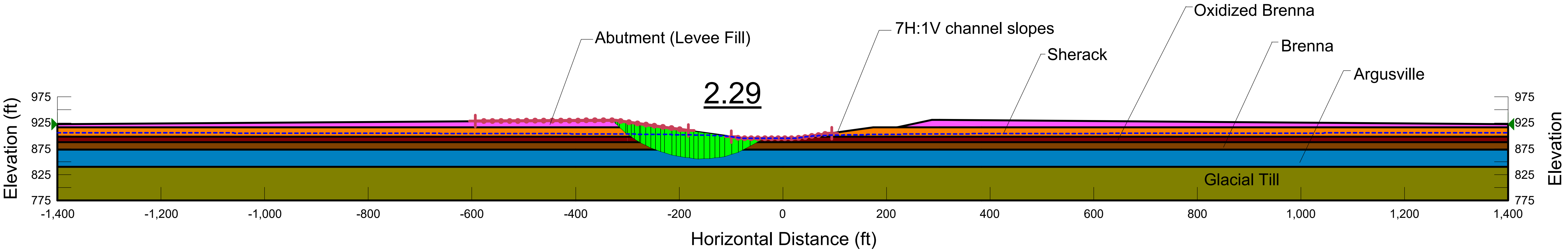
Fargo-Moorhead Metropolitan Feasibility Study
Interstate 29 Southbound - Geotech Analysis 1
Case: Slope Stability ESSA L-R (2)
Last Saved Date: 6/30/2014

Factor of Safety: 2.29

Name: Sherack Formation (Drained)   Unit Weight: 115 pcf   Cohesion': 0 psf   Phi 1: 28 °   Phi 2: 11 °   Bilinear Normal: 2,000 psf   Unit Wt. Above Water Table: 113.1 pcf
Name: Oxidized Brenna (Drained)   Unit Weight: 108 pcf   Unit Wt. Above Water Table: 107.2 pcf   Strength Function: CIU Triaxial (WRR, 15% Axial Strain Failure Criterion)
Name: Brenna (Drained)   Unit Weight: 106 pcf   Unit Wt. Above Water Table: 105.5 pcf   Strength Function: CIU Triaxial (WRR, 15% Axial Strain Failure Criterion)
Name: Argusville Formation (Drained)   Unit Weight: 110 pcf   Unit Wt. Above Water Table: 109.2 pcf   Strength Function: CIU Triaxial (WRR, 15% Axial Strain Failure Criterion)
Name: Glacial Till (Drained)   Unit Weight: 123 pcf   Cohesion': 225 psf   Unit Wt. Above Water Table: 121.9 pcf   Phi': 22 °
Name: Levee Fill (Ultimate Drained)   Unit Weight: 115 pcf   Cohesion': 150 psf   Phi 1: 24 °   Phi 2: 11 °   Bilinear Normal: 1,500 psf

Main Diversion Channel
Interstate 29 Southbound



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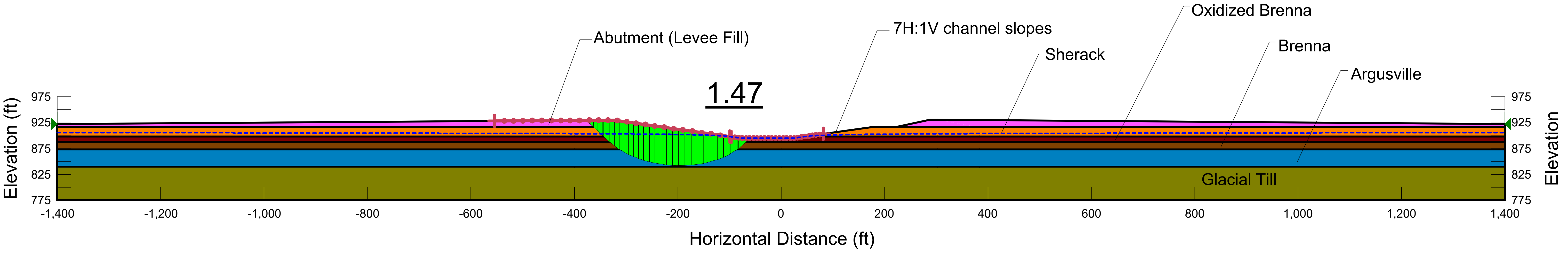
Fargo-Moorhead Metropolitan Feasibility Study
Interstate 29 Southbound - Geotech Analysis 1
Case: Slope Stability USSA L-R
Last Saved Date: 6/30/2014

Factor of Safety: 1.47

Name: Sherack Formation (Undrained) Unit Weight: 115 pcf Cohesion': 900 psf Phi': 0 ° Unit Wt. Above Water Table: 113.1 pcf
Name: Oxidized Brenna (Undrained) Unit Weight: 108 pcf Cohesion': 900 psf Phi': 0 ° Unit Wt. Above Water Table: 107.2 pcf
Name: Brenna (Undrained) Unit Weight: 106 pcf Cohesion': 575 psf Phi': 0 ° Unit Wt. Above Water Table: 105.5 pcf
Name: Argusville Formation (Undrained) Unit Weight: 110 pcf Unit Wt. Above Water Table: 109.2 pcf C-Top of Layer: 575 psf C-Rate of Change: 10 psf/ft
Name: Glacial Till (Undrained) Unit Weight: 123 pcf Cohesion': 1,900 psf Unit Wt. Above Water Table: 121.9 pcf
Name: Levee Fill (Ultimate Undrained) Unit Weight: 115 pcf Cohesion': 900 psf Phi': 0 °

Main Diversion Channel
Interstate 29 Southbound



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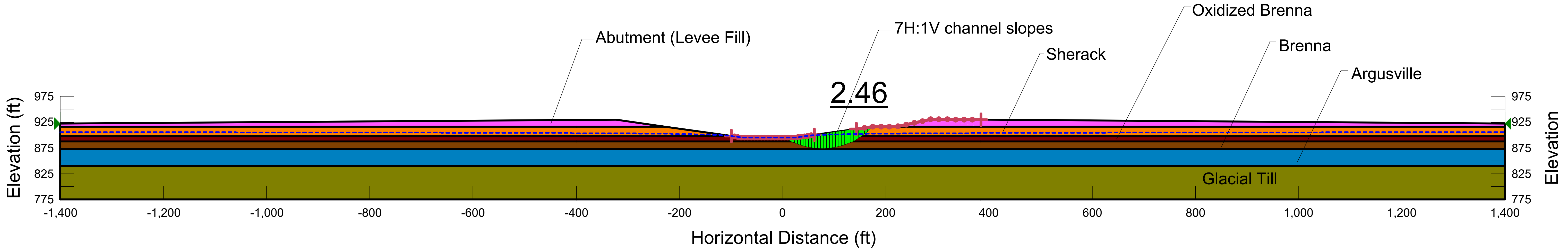
Fargo-Moorhead Metropolitan Feasibility Study
Interstate 29 Southbound - Geotech Analysis 1
Case: Slope Stability ESSA R-L
Last Saved Date: 6/30/2014

Factor of Safety: 2.46

Name: Sherack Formation (Drained)   Unit Weight: 115 pcf   Cohesion': 0 psf   Phi 1: 28 °   Phi 2: 11 °   Bilinear Normal: 2,000 psf   Unit Wt. Above Water Table: 113.1 pcf
Name: Oxidized Brenna (Drained)   Unit Weight: 108 pcf   Unit Wt. Above Water Table: 107.2 pcf   Strength Function: CIU Triaxial (WRR, 15% Axial Strain Failure Criterion)
Name: Brenna (Drained)   Unit Weight: 106 pcf   Unit Wt. Above Water Table: 105.5 pcf   Strength Function: CIU Triaxial (WRR, 15% Axial Strain Failure Criterion)
Name: Argusville Formation (Drained)   Unit Weight: 110 pcf   Unit Wt. Above Water Table: 109.2 pcf   Strength Function: CIU Triaxial (WRR, 15% Axial Strain Failure Criterion)
Name: Glacial Till (Drained)   Unit Weight: 123 pcf   Cohesion': 225 psf   Unit Wt. Above Water Table: 121.9 pcf   Phi': 22 °
Name: Levee Fill (Ultimate Drained)   Unit Weight: 115 pcf   Cohesion': 150 psf   Phi 1: 24 °   Phi 2: 11 °   Bilinear Normal: 1,500 psf

Main Diversion Channel
Interstate 29 Southbound



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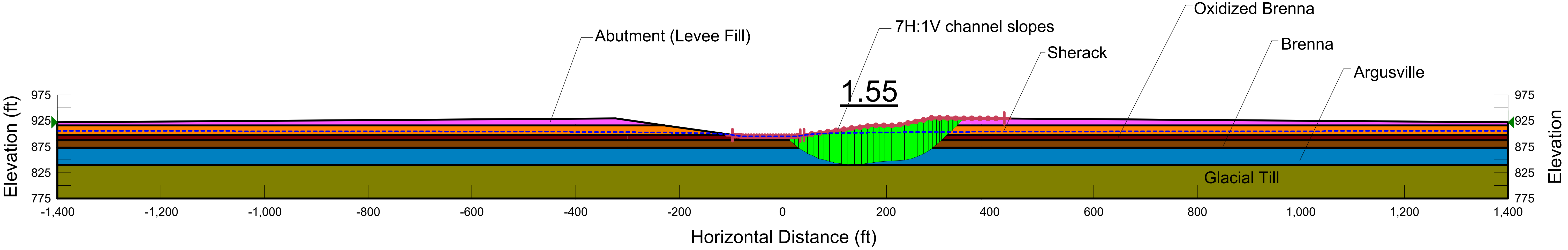
Fargo-Moorhead Metropolitan Feasibility Study
Interstate 29 Southbound - Geotech Analysis 1
Case: Slope Stability USSA R-L
Last Saved Date: 6/30/2014

Factor of Safety: 1.55

Name: Sherack Formation (Undrained) Unit Weight: 115 pcf Cohesion': 900 psf Phi': 0 ° Unit Wt. Above Water Table: 113.1 pcf
Name: Oxidized Brenna (Undrained) Unit Weight: 108 pcf Cohesion': 900 psf Phi': 0 ° Unit Wt. Above Water Table: 107.2 pcf
Name: Brenna (Undrained) Unit Weight: 106 pcf Cohesion': 575 psf Phi': 0 ° Unit Wt. Above Water Table: 105.5 pcf
Name: Argusville Formation (Undrained) Unit Weight: 110 pcf Unit Wt. Above Water Table: 109.2 pcf C-Top of Layer: 575 psf C-Rate of Change: 10 psf/ft
Name: Glacial Till (Undrained) Unit Weight: 123 pcf Cohesion': 1,900 psf Unit Wt. Above Water Table: 121.9 pcf
Name: Levee Fill (Ultimate Undrained) Unit Weight: 115 pcf Cohesion': 900 psf Phi': 0 °

Main Diversion Channel
Interstate 29 Southbound



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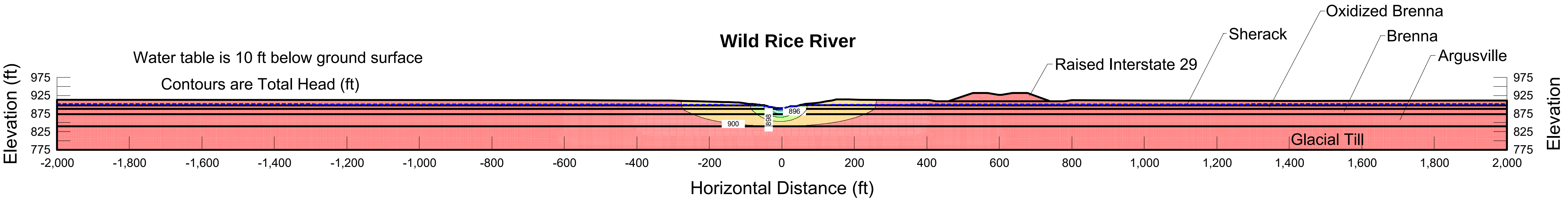
Analysis 2

Fargo-Moorhead Metropolitan Feasibility Study
Wild Rice River - Geotech Analysis 2
Case: Steady State Seepage Analysis
Last Saved Date: 7/15/2014

Name: Sherack Formation (Drained)_Peak K-Function: Sherack_Formation (k=1.13E-2 ft/day) Ky'/Kx' Ratio: 0.25 Vol. WC. Function: Sherack_Formation
Name: Oxidized Brenna (Drained) K-Function: OX_Brenna_Formation (k=1.4E-3 ft/day) Ky'/Kx' Ratio: 1 Vol. WC. Function: OX_Brenna_Formation
Name: Brenna (Drained) K-Function: Brenna_Formation (k=2.8E-4 ft/day) Ky'/Kx' Ratio: 1 Vol. WC. Function: Brenna_Formation
Name: Argusville Formation (Drained) K-Function: Argusville_Formation (k=2.8E-4 ft/day) Ky'/Kx' Ratio: 1 Vol. WC. Function: Argusville_Formation
Name: Glacial Till (Drained) K-Function: Glacial_Till (k=5.7E-2 ft/day) Ky'/Kx' Ratio: 0.25 Vol. WC. Function: Glacial_Till
Name: Levee Fill (Peak Drained) K-Function: Sherack_Formation (k=1.13E-2 ft/day) Ky'/Kx' Ratio: 1 Vol. WC. Function: Sherack_Formation



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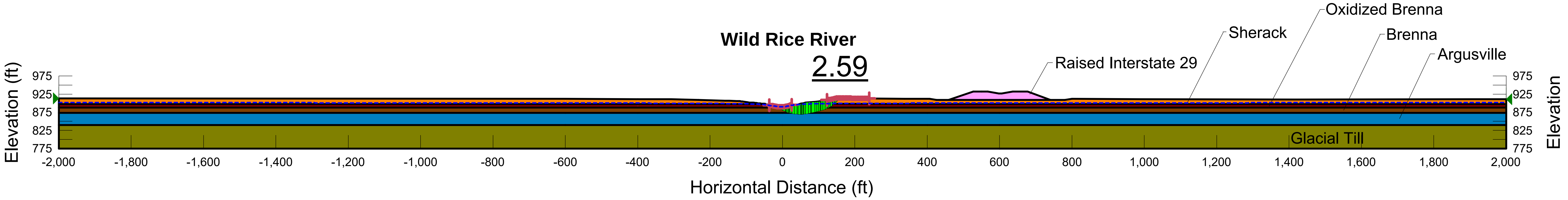
Fargo-Moorhead Metropolitan Feasibility Study
Wild Rice River - Geotech Analysis 2
Case: WRR Stability ESSA R-L (2)
Last Saved Date: 7/15/2014

Factor of Safety: 2.59

Name: Sherack Formation (Drained)_Peak   Unit Weight: 115 pcf   Cohesion': 0 psf   Phi 1: 30 °   Phi 2: 25 °   Bilinear Normal: 2,000 psf   Unit Wt. Above Water Table: 113.1 pcf
Name: Oxidized Brenna (Drained)   Unit Weight: 108 pcf   Unit Wt. Above Water Table: 107.2 pcf   Strength Function: CIU Triaxial (WRR, Peak Axial Strain Failure Criterion)
Name: Brenna (Drained)   Unit Weight: 106 pcf   Unit Wt. Above Water Table: 105.5 pcf   Strength Function: CIU Triaxial (WRR, Peak Axial Strain Failure Criterion)
Name: Argusville Formation (Drained)   Unit Weight: 110 pcf   Unit Wt. Above Water Table: 109.2 pcf   Strength Function: CIU Triaxial (WRR, Peak Axial Strain Failure Criterion)
Name: Glacial Till (Drained)   Unit Weight: 123 pcf   Cohesion': 225 psf   Unit Wt. Above Water Table: 121.9 pcf   Phi': 25 °
Name: Levee Fill (Peak Drained)   Unit Weight: 115 pcf   Cohesion': 150 psf   Phi 1: 28 °   Phi 2: 21 °   Bilinear Normal: 1,500 psf



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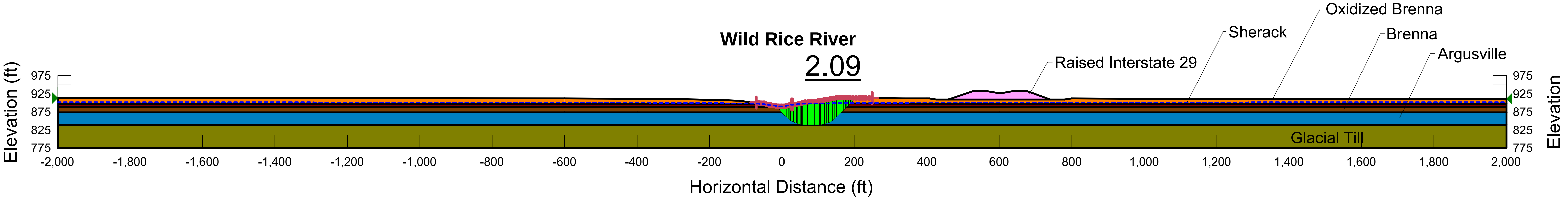
Fargo-Moorhead Metropolitan Feasibility Study
Wild Rice River - Geotech Analysis 2
Case: WRR Stability USSA R-L
Last Saved Date: 7/15/2014

Factor of Safety: 2.09

Name: Sherack Formation (Undrained)   Unit Weight: 115 pcf   Cohesion': 1,400 psf   Phi': 0 °   Unit Wt. Above Water Table: 113.1 pcf
Name: Oxidized Brenna (Undrained)   Unit Weight: 108 pcf   Cohesion': 1,000 psf   Phi': 0 °   Unit Wt. Above Water Table: 107.2 pcf
Name: Brenna (Undrained)   Unit Weight: 106 pcf   Cohesion': 650 psf   Phi': 0 °   Unit Wt. Above Water Table: 105.5 pcf
Name: Argusville Formation (Undrained)   Unit Weight: 110 pcf   Cohesion': 825 psf   Phi': 0 °   Unit Wt. Above Water Table: 109.2 pcf
Name: Glacial Till (Undrained)   Unit Weight: 123 pcf   Cohesion': 2,200 psf   Unit Wt. Above Water Table: 121.9 pcf
Name: Levee Fill (Peak Undrained)   Unit Weight: 115 pcf   Cohesion': 1,400 psf   Phi': 0 °



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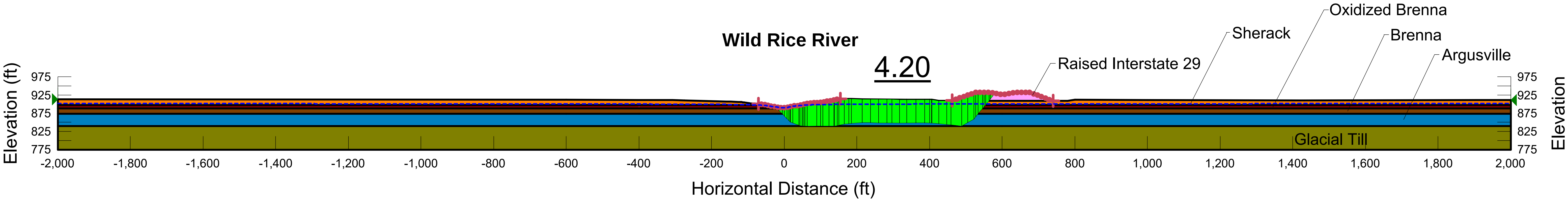
Fargo-Moorhead Metropolitan Feasibility Study
Wild Rice River - Geotech Analysis 2
Case: Slope Stability ESSA R-L
Last Saved Date: 7/15/2014

Factor of Safety: 4.20

Name: Sherack Formation (Drained)_Peak Unit Weight: 115 pcf Cohesion': 0 psf Phi 1: 30 ° Phi 2: 25 ° Bilinear Normal: 2,000 psf Unit Wt. Above Water Table: 113.1 pcf
Name: Oxidized Brenna (Drained) Unit Weight: 108 pcf Unit Wt. Above Water Table: 107.2 pcf Strength Function: CIU Triaxial (WRR, Peak Axial Strain Failure Criterion)
Name: Brenna (Drained) Unit Weight: 106 pcf Unit Wt. Above Water Table: 105.5 pcf Strength Function: CIU Triaxial (WRR, Peak Axial Strain Failure Criterion)
Name: Argusville Formation (Drained) Unit Weight: 110 pcf Unit Wt. Above Water Table: 109.2 pcf Strength Function: CIU Triaxial (WRR, Peak Axial Strain Failure Criterion)
Name: Glacial Till (Drained) Unit Weight: 123 pcf Cohesion': 225 psf Unit Wt. Above Water Table: 121.9 pcf Phi': 25 °
Name: Levee Fill (Peak Drained) Unit Weight: 115 pcf Cohesion': 150 psf Phi 1: 28 ° Phi 2: 21 ° Bilinear Normal: 1,500 psf



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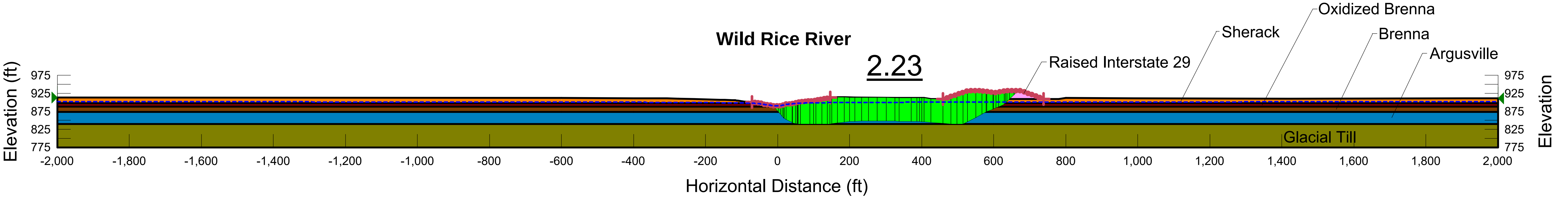
Fargo-Moorhead Metropolitan Feasibility Study
Wild Rice River - Geotech Analysis 2
Case: Slope Stability USSA R-L
Last Saved Date: 7/15/2014

Factor of Safety: 2.23

Name: Sherack Formation (Undrained)   Unit Weight: 115 pcf   Cohesion': 1,400 psf   Phi': 0 °   Unit Wt. Above Water Table: 113.1 pcf
Name: Oxidized Brenna (Undrained)   Unit Weight: 108 pcf   Cohesion': 1,000 psf   Phi': 0 °   Unit Wt. Above Water Table: 107.2 pcf
Name: Brenna (Undrained)   Unit Weight: 106 pcf   Cohesion': 650 psf   Phi': 0 °   Unit Wt. Above Water Table: 105.5 pcf
Name: Argusville Formation (Undrained)   Unit Weight: 110 pcf   Cohesion': 825 psf   Phi': 0 °   Unit Wt. Above Water Table: 109.2 pcf
Name: Glacial Till (Undrained)   Unit Weight: 123 pcf   Cohesion': 2,200 psf   Unit Wt. Above Water Table: 121.9 pcf
Name: Levee Fill (Peak Undrained)   Unit Weight: 115 pcf   Cohesion': 1,400 psf   Phi': 0 °



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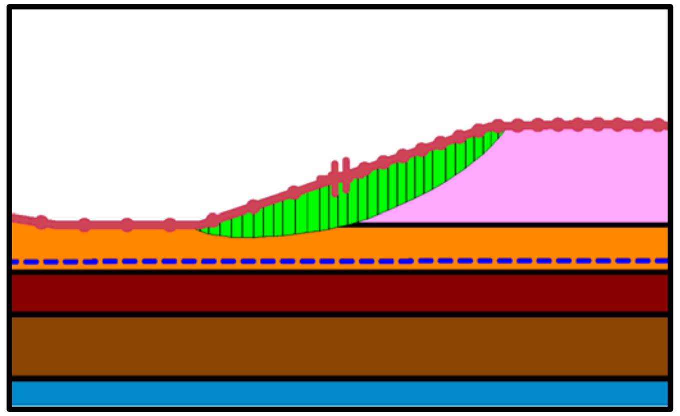
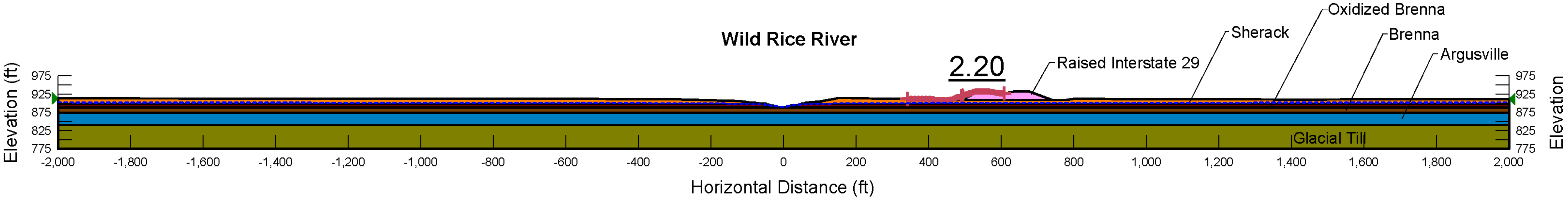
Fargo-Moorhead Metropolitan Feasibility Study
Wild Rice River - Geotech Analysis 2
Case: I29 Slope Stability ESSA
Last Saved Date: 7/15/2014



Factor of Safety: 2.20

Name: Sherack Formation (Drained)_Peak Unit Weight: 115 pcf Cohesion': 0 psf Phi 1: 30 ° Phi 2: 25 ° Bilinear Normal: 2,000 psf Unit Wt. Above Water Table: 113.1 pcf
Name: Oxidized Brenna (Drained) Unit Weight: 108 pcf Unit Wt. Above Water Table: 107.2 pcf Strength Function: CIU Triaxial (WRR, Peak Axial Strain Failure Criterion)
Name: Brenna (Drained) Unit Weight: 106 pcf Unit Wt. Above Water Table: 105.5 pcf Strength Function: CIU Triaxial (WRR, Peak Axial Strain Failure Criterion)
Name: Argusville Formation (Drained) Unit Weight: 110 pcf Unit Wt. Above Water Table: 109.2 pcf Strength Function: CIU Triaxial (WRR, Peak Axial Strain Failure Criterion)
Name: Glacial Till (Drained) Unit Weight: 123 pcf Cohesion': 225 psf Unit Wt. Above Water Table: 121.9 pcf Phi': 25 °
Name: Levee Fill (Peak Drained) Unit Weight: 115 pcf Cohesion': 150 psf Phi 1: 28 ° Phi 2: 21 ° Bilinear Normal: 1,500 psf



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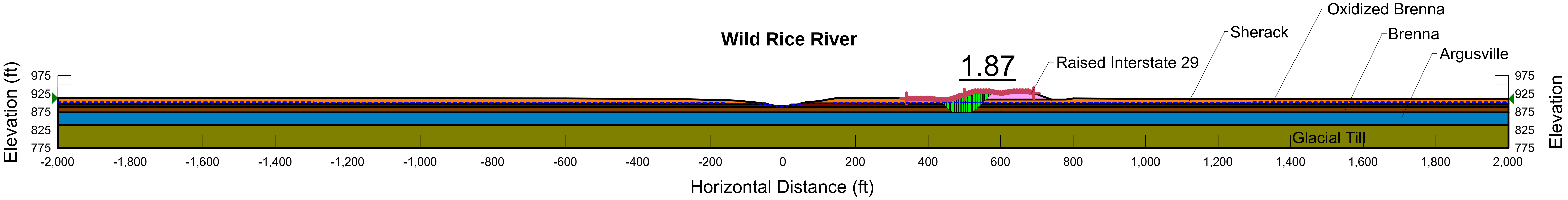
Fargo-Moorhead Metropolitan Feasibility Study
Wild Rice River - Geotech Analysis 2
Case: I29 Slope Stability USSA
Last Saved Date: 7/15/2014

Factor of Safety: 1.87

Name: Sherack Formation (Undrained) Unit Weight: 115 pcf Cohesion': 1,400 psf Phi': 0 ° Unit Wt. Above Water Table: 113.1 pcf
Name: Oxidized Brenna (Undrained) Unit Weight: 108 pcf Cohesion': 1,000 psf Phi': 0 ° Unit Wt. Above Water Table: 107.2 pcf
Name: Brenna (Undrained) Unit Weight: 106 pcf Cohesion': 650 psf Phi': 0 ° Unit Wt. Above Water Table: 105.5 pcf
Name: Argusville Formation (Undrained) Unit Weight: 110 pcf Cohesion': 825 psf Phi': 0 ° Unit Wt. Above Water Table: 109.2 pcf
Name: Glacial Till (Undrained) Unit Weight: 123 pcf Cohesion': 2,200 psf Unit Wt. Above Water Table: 121.9 pcf
Name: Levee Fill (Peak Undrained) Unit Weight: 115 pcf Cohesion': 1,400 psf Phi': 0 °



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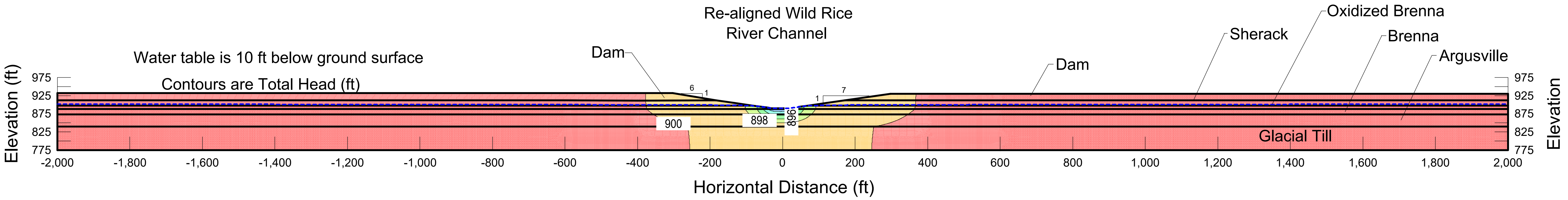
Analysis 3

Fargo-Moorhead Metropolitan Area Flood Risk Management Project
Geotechnical Analysis 3 - 0' Offset_20' EMB
Case: Steady State Seepage Analysis
Last Saved Date: 6/30/2014

Name: Sherack Formation (Drained) K-Function: Sherack_Formation (k=1.13E-2 ft/day) Ky'/Kx' Ratio: 0.25 Vol. WC. Function: Sherack_Formation
Name: Oxidized Brenna (Drained) K-Function: OX_Brenna_Formation (k=1.4E-3 ft/day) Ky'/Kx' Ratio: 1 Vol. WC. Function: OX_Brenna_Formation
Name: Brenna (Drained) K-Function: Brenna_Formation (k=2.8E-4 ft/day) Ky'/Kx' Ratio: 1 Vol. WC. Function: Brenna_Formation
Name: Argusville Formation (Drained) K-Function: Argusville_Formation (k=2.8E-4 ft/day) Ky'/Kx' Ratio: 1 Vol. WC. Function: Argusville_Formation
Name: Glacial Till (Drained) K-Function: Glacial_Till (k=5.7E-2 ft/day) Ky'/Kx' Ratio: 0.25 Vol. WC. Function: Glacial_Till
Name: Levee Fill (Ultimate Drained) K-Function: Sherack_Formation (k=1.13E-2 ft/day) Ky'/Kx' Ratio: 1 Vol. WC. Function: Sherack_Formation



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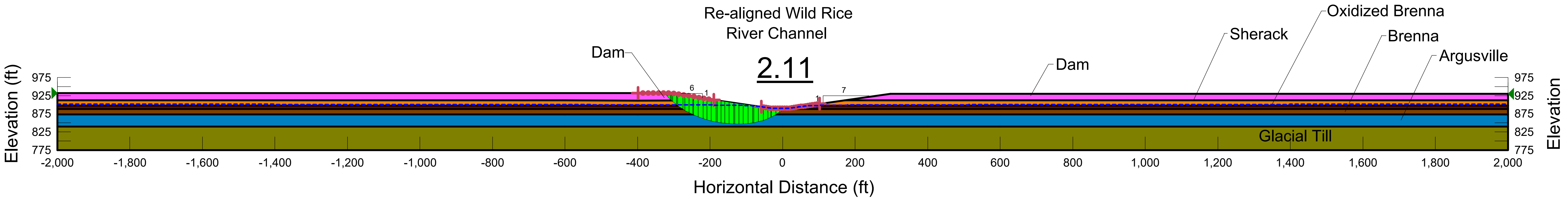
Fargo-Moorhead Metropolitan Area Flood Risk Management Project
Geotechnical Analysis 3 - 0' Offset_20' EMB
Case: Slope Stability ESSA L-R (2)
Last Saved Date: 6/30/2014

Factor of Safety: 2.11

Name: Sherack Formation (Drained)   Unit Weight: 115 pcf   Cohesion': 0 psf   Phi 1: 28 °   Phi 2: 11 °   Bilinear Normal: 2,000 psf   Unit Wt. Above Water Table: 113.1 pcf
Name: Oxidized Brenna (Drained)   Unit Weight: 108 pcf   Unit Wt. Above Water Table: 107.2 pcf   Strength Function: CIU Triaxial (WRR, 15% Axial Strain Failure Criterion)
Name: Brenna (Drained)   Unit Weight: 106 pcf   Unit Wt. Above Water Table: 105.5 pcf   Strength Function: CIU Triaxial (WRR, 15% Axial Strain Failure Criterion)
Name: Argusville Formation (Drained)   Unit Weight: 110 pcf   Unit Wt. Above Water Table: 109.2 pcf   Strength Function: CIU Triaxial (WRR, 15% Axial Strain Failure Criterion)
Name: Glacial Till (Drained)   Unit Weight: 123 pcf   Cohesion': 225 psf   Unit Wt. Above Water Table: 121.9 pcf   Phi': 22 °
Name: Levee Fill (Ultimate Drained)   Unit Weight: 115 pcf   Cohesion': 150 psf   Phi 1: 24 °   Phi 2: 11 °   Bilinear Normal: 1,500 psf



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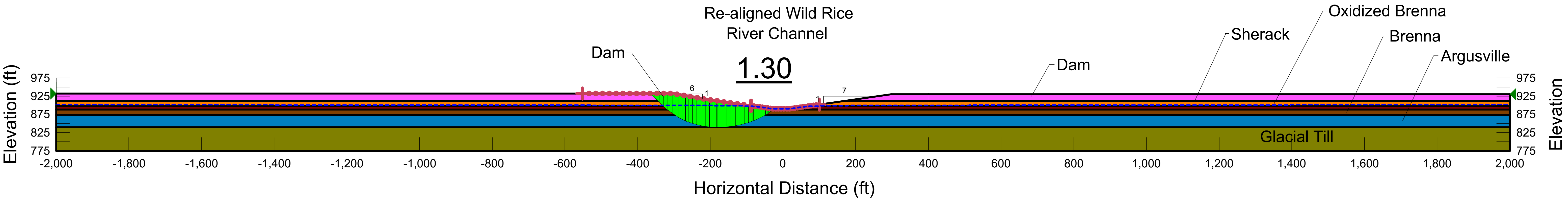
Fargo-Moorhead Metropolitan Area Flood Risk Management Project
Geotechnical Analysis 3 - 0' Offset_20' EMB
Case: Slope Stability USSA L-R
Last Saved Date: 6/30/2014

Factor of Safety: 1.30

Name: Sherack Formation (Undrained) Unit Weight: 115 pcf Cohesion': 900 psf Phi': 0 ° Unit Wt. Above Water Table: 113.1 pcf
Name: Oxidized Brenna (Undrained) Unit Weight: 108 pcf Cohesion': 900 psf Phi': 0 ° Unit Wt. Above Water Table: 107.2 pcf
Name: Brenna (Undrained) Unit Weight: 106 pcf Cohesion': 575 psf Phi': 0 ° Unit Wt. Above Water Table: 105.5 pcf
Name: Argusville Formation (Undrained) Unit Weight: 110 pcf Unit Wt. Above Water Table: 109.2 pcf C-Top of Layer: 575 psf C-Rate of Change: 10 psf/ft
Name: Glacial Till (Undrained) Unit Weight: 123 pcf Cohesion': 1,900 psf Unit Wt. Above Water Table: 121.9 pcf
Name: Levee Fill (Ultimate Undrained) Unit Weight: 115 pcf Cohesion': 900 psf Phi': 0 °



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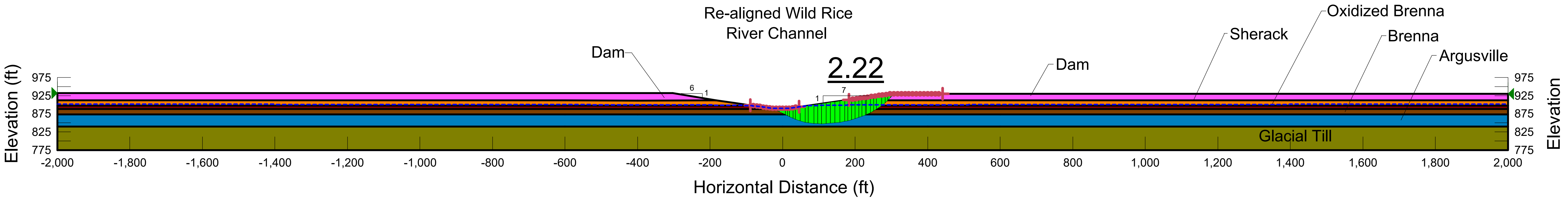
Fargo-Moorhead Metropolitan Area Flood Risk Management Project
Geotechnical Analysis 3 - 0' Offset_20' EMB
Case: Slope Stability ESSA R-L (2)
Last Saved Date: 6/30/2014

Factor of Safety: 2.22

Name: Sherack Formation (Drained)   Unit Weight: 115 pcf   Cohesion': 0 psf   Phi 1: 28 °   Phi 2: 11 °   Bilinear Normal: 2,000 psf   Unit Wt. Above Water Table: 113.1 pcf
Name: Oxidized Brenna (Drained)   Unit Weight: 108 pcf   Unit Wt. Above Water Table: 107.2 pcf   Strength Function: CIU Triaxial (WRR, 15% Axial Strain Failure Criterion)
Name: Brenna (Drained)   Unit Weight: 106 pcf   Unit Wt. Above Water Table: 105.5 pcf   Strength Function: CIU Triaxial (WRR, 15% Axial Strain Failure Criterion)
Name: Argusville Formation (Drained)   Unit Weight: 110 pcf   Unit Wt. Above Water Table: 109.2 pcf   Strength Function: CIU Triaxial (WRR, 15% Axial Strain Failure Criterion)
Name: Glacial Till (Drained)   Unit Weight: 123 pcf   Cohesion': 225 psf   Unit Wt. Above Water Table: 121.9 pcf   Phi': 22 °
Name: Levee Fill (Ultimate Drained)   Unit Weight: 115 pcf   Cohesion': 150 psf   Phi 1: 24 °   Phi 2: 11 °   Bilinear Normal: 1,500 psf



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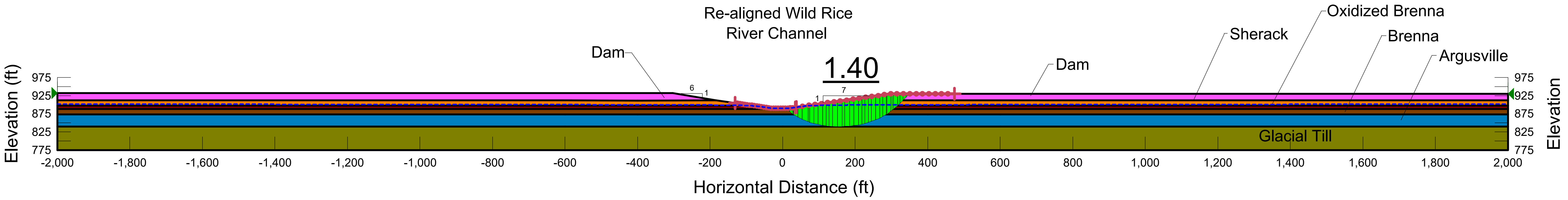
Fargo-Moorhead Metropolitan Area Flood Risk Management Project
Geotechnical Analysis 3 - 0' Offset_20' EMB
Case: Slope Stability USSA R-L
Last Saved Date: 6/30/2014

Factor of Safety: 1.40

Name: Sherack Formation (Undrained) Unit Weight: 115 pcf Cohesion': 900 psf Phi': 0 ° Unit Wt. Above Water Table: 113.1 pcf
Name: Oxidized Brenna (Undrained) Unit Weight: 108 pcf Cohesion': 900 psf Phi': 0 ° Unit Wt. Above Water Table: 107.2 pcf
Name: Brenna (Undrained) Unit Weight: 106 pcf Cohesion': 575 psf Phi': 0 ° Unit Wt. Above Water Table: 105.5 pcf
Name: Argusville Formation (Undrained) Unit Weight: 110 pcf Unit Wt. Above Water Table: 109.2 pcf C-Top of Layer: 575 psf C-Rate of Change: 10 psf/ft
Name: Glacial Till (Undrained) Unit Weight: 123 pcf Cohesion': 1,900 psf Unit Wt. Above Water Table: 121.9 pcf
Name: Levee Fill (Ultimate Undrained) Unit Weight: 115 pcf Cohesion': 900 psf Phi': 0 °



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Attachment C

I-29 Settlement Analysis

Settlement Analysis

Project: FMMFS (34091004.13, Task Order 14, FY2013)

Wild Rice River Structure

Barr Engineering Company
Settlement Analysis Spreadsheet
June 16, 2014

References:

Advanced Soil Mechanics, 1st Edition. Das, Braja M. Hemisphere Publish Corporation, 1983, page 186.

Principles of Foundation Engineering, 6th Edition. Das, Braja M. Thomson, 2007, page 253.

Principles of Geotechnical Engineering, 5th Edition. Das, Braja M. Brooks/Cole, 2002, page 280.

Naval Facilities Engineering Command Design Manual 7.01, 1986, page 77.1.237.

Engineering and Design Manual 1110-1-1904. Settlement Analysis. September 30, 1990.

Assumptions:

1. No consolidation occurs above the water table
2. Input values determined from site-specific laboratory consolidation and Proctor tests
3. Unit weights are based on the specific gravity and void ratios determined from laboratory testing
4. Groundwater table depth was assumed 10' below ground surface

Inputs:

Inputs
Calculated

Depth to ground water =	10	ft
Top Width of Embankment =	150	ft
b =	75	ft
Slope of sides (xH:1V) =	3	H:1V
Height of Embankment =	15	ft
Unit Weight of Embankment =	115	pcf
a =	45	ft
$U_{settlement}$ =	99	%

		Lower Lake Agassiz Clay		
Geologic Units:	Sherack	Ox. Brenna	Brenna	Argusville
Depth to Top of Formation (ft):	0	15	25	38
Depth to Bottom of Formation (ft):	15	25	38	73
Layer Thickness Below Groundwater (ft):	5	10	13	35
γ_{sat} (pcf):	109.7	109.1	112.9	108.6
γ_{dry} (pcf):	75	73	79	73
γ_m (pcf):	109	108	112	108
Moisture Content (%):	44.9	47.3	41.7	47.9
Specifc Gravity, G_s:	2.72	2.75	2.77	2.73
Compression Index, C_c :	0.36	0.57	0.49	0.54
Recompression Index, C_r or C_s :	0.11	0.15	0.12	0.14
Void Ratio, e_o :	1.27	1.34	1.19	1.34
C_v (ft^2/s)* :	6.79E-02	7.07E-02	1.30E-01	7.95E-02
Degree of Saturation (%):	96.46	97.23	97.34	97.82

* Based on anticipated stress under the embankment

Settlement Analysis

Project: FMMFS (34091004.13, Task Order 14, FY2013)

Wild Rice River Structure

Barr Engineering Company
Settlement Analysis Spreadsheet
June 16, 2014

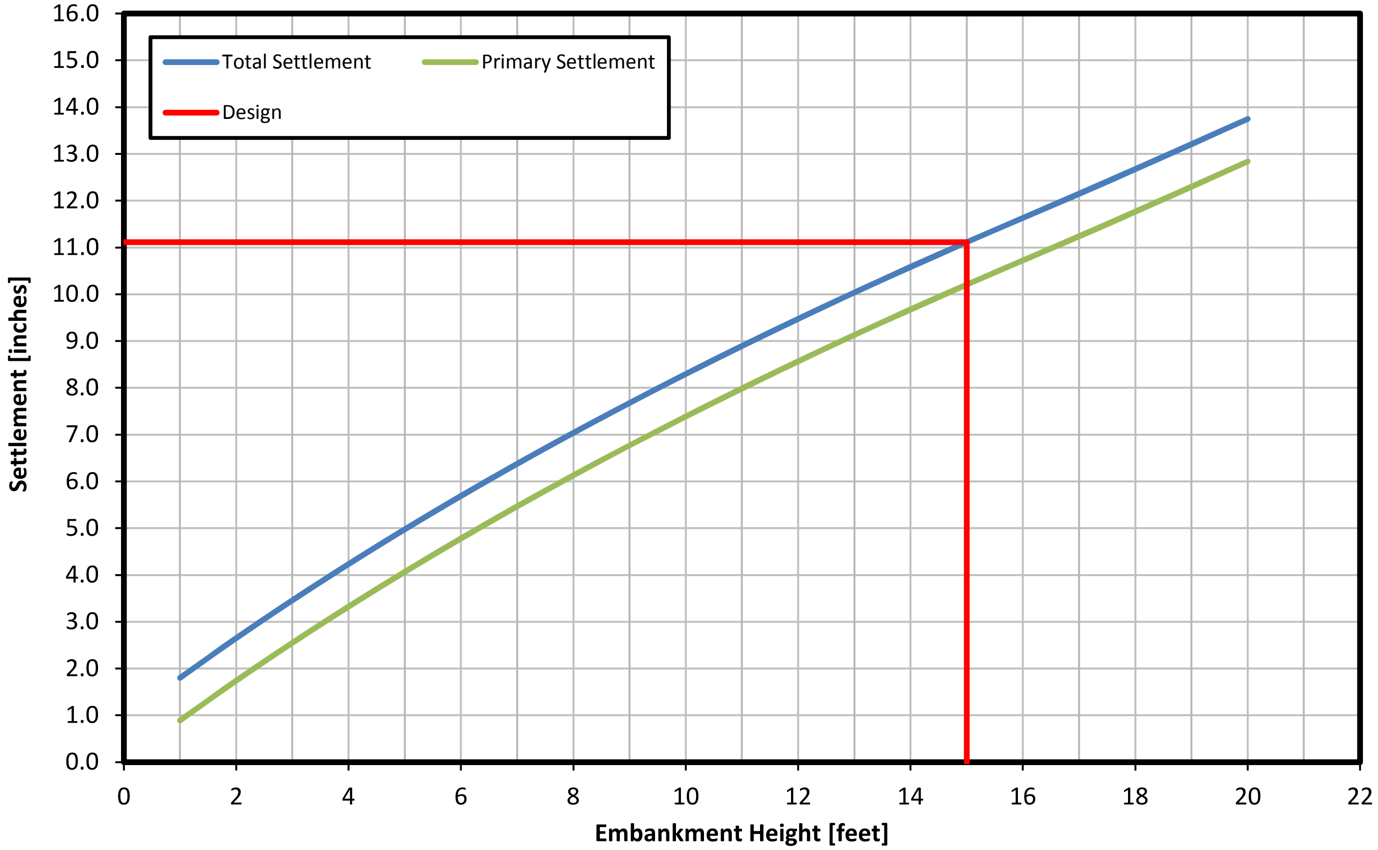
Total Settlement = Primary + Secondary

Primary Settlement = time delayed consolidation as the reduction in volume associated with the dissipation of excess pore pressues due to mechanical loading

Secondary Compression = consolidation of saturated soils due to loading, during the time when the excess pore pressures dissipate.

Design Height: 15 ft

Total Settlement: 11.1 inches



Primary Settlement:

Sherack	1.12	10.2	inches
Lower Lake Agassiz Clay	9.08		

Secondary Compression:

Sherack	0.08	0.9	inches
Lower Lake Agassiz Clay	0.83		

Primary Settlement

Reference: Principles of Foundation Engineering, 6th Ed. Das, Braja M. Thomson, 2007

σ'_{vo} = average effective stress on the clay layer before the construction of the embankment

$\Delta\sigma'$ = average increase in effective stress on the clay layer caused by the construction of the embankment

e_o = initial void ratio of the clay layer

C_c = compression index

C_r = recompression index

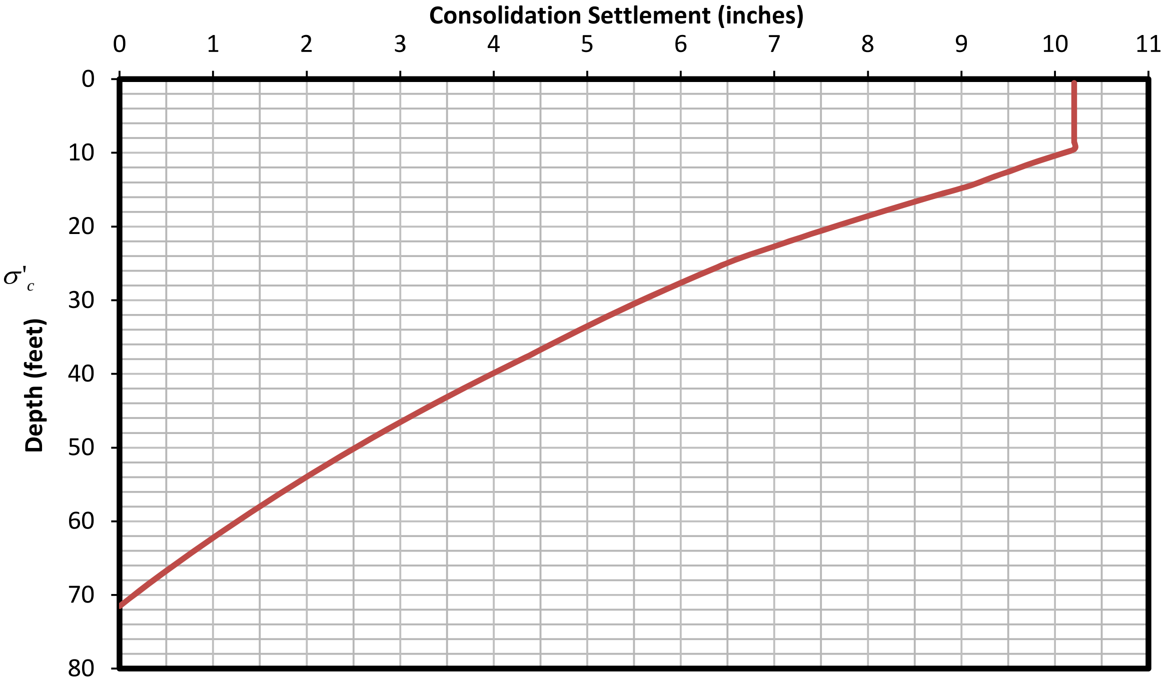
σ_c' = preconsolidation pressure (σ_p')

$$S = \frac{C_c H}{1+e_o}\log\frac{\sigma'_{vo}+\Delta\sigma'}{\sigma'_{vo}} \quad for: NC\ clays$$

$$S = \frac{C_r H}{1+e_o}\log\frac{\sigma'_{vo}+\Delta\sigma'}{\sigma'_{vo}} \quad for: \sigma'_{vo}+\Delta\sigma' \le \sigma'_c$$

$$S = \frac{C_r H}{1+e_o}\log\frac{\sigma'_c}{\sigma'_{vo}} + \frac{C_c H}{1+e_o}\log\frac{\sigma'_{vo}+\Delta\sigma'}{\sigma'_c} \quad for: \sigma'_{vo}+\Delta\sigma' > \sigma'_c$$

Wild Rice River Primary Consolidation Settlement



Primary Settlement																						Design Value
Embankment Height (ft)		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	15
Settlement (in)	Sherack	0.1	0.2	0.3	0.4	0.5	0.6	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.1	1.2	1.2	1.3	1.3	1.4	1.1
	Ox. Brenna	0.2	0.5	0.7	0.9	1.1	1.2	1.4	1.6	1.7	1.9	2.0	2.1	2.3	2.4	2.5	2.6	2.7	2.8	2.9	3.0	2.5
	Brenna	0.2	0.4	0.6	0.7	0.9	1.0	1.2	1.3	1.5	1.6	1.7	1.9	2.0	2.1	2.2	2.3	2.4	2.5	2.6	2.7	2.2
	Argusville	0.3	0.7	1.0	1.3	1.6	1.9	2.2	2.5	2.8	3.1	3.4	3.6	3.9	4.1	4.4	4.6	4.9	5.1	5.4	5.7	4.4
Total Settlement (in)		0.9	1.7	2.5	3.3	4.1	4.8	5.5	6.1	6.8	7.4	8.0	8.6	9.1	9.7	10.2	10.7	11.2	11.8	12.3	12.8	10.2

Time-rate of Consolidation

Assumptions:

1. Materials are fully saturated
2. Compressibility of water is negligible
3. Compressibility of soil grains is negligible

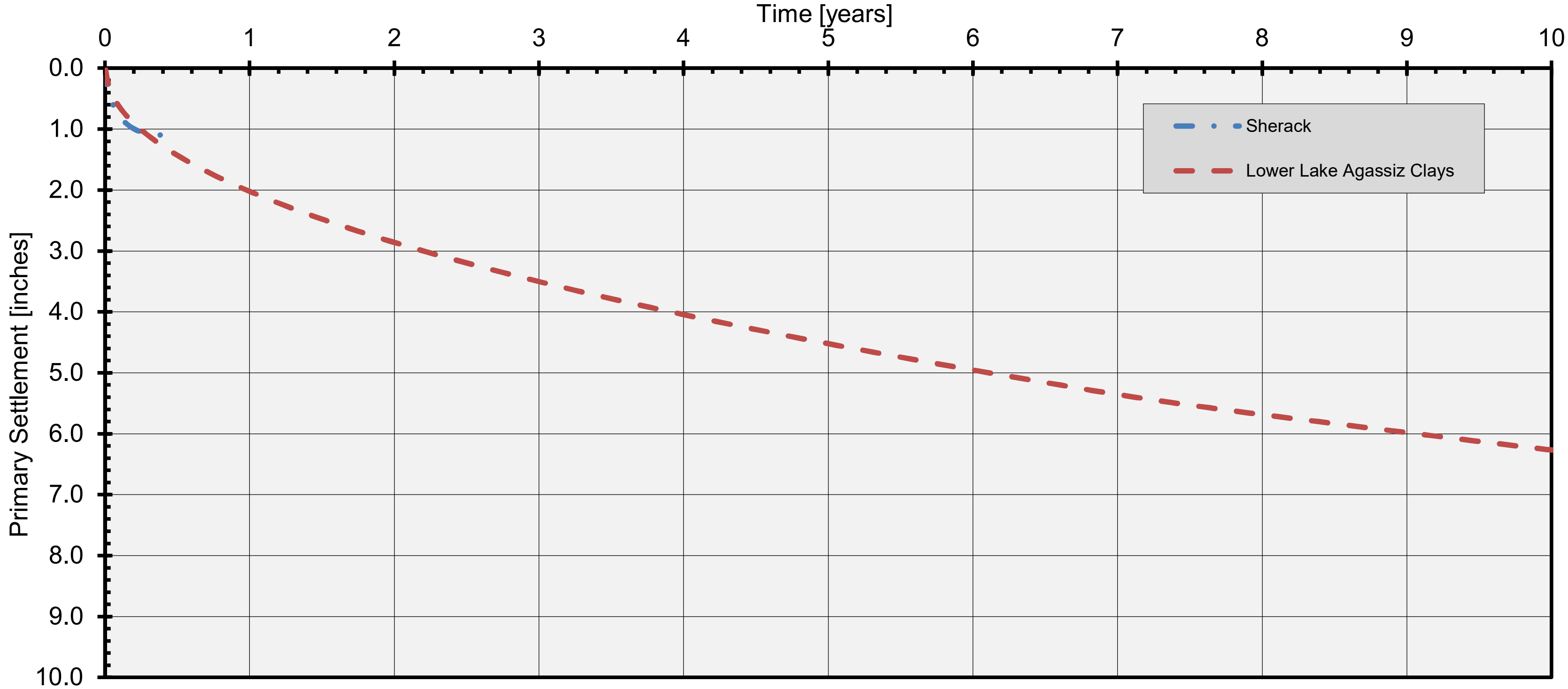
$$T_v = \frac{c_v t}{{H_{dr}}^2}$$

T_v = time factor
c_v = coefficient of consolidation
t = time
H_{dr} = length of maximum drainage path

Inputs:

	Sherack	Lower Lake Agassiz Clays
coefficient of consolidation, c_v (ft^2/day):	0.068	0.089
Drainage:	Double	Double
layer thickness, H_{dr} (ft):	2.5	29
percent consolidation, U (%):	99	99
time factor, T_v:	1.781	1.781
time (days):	164	16763
Time to end of primary consolidation (years):	0.449	45.93

Primary Settlement vs Time Due to Embankment Construction

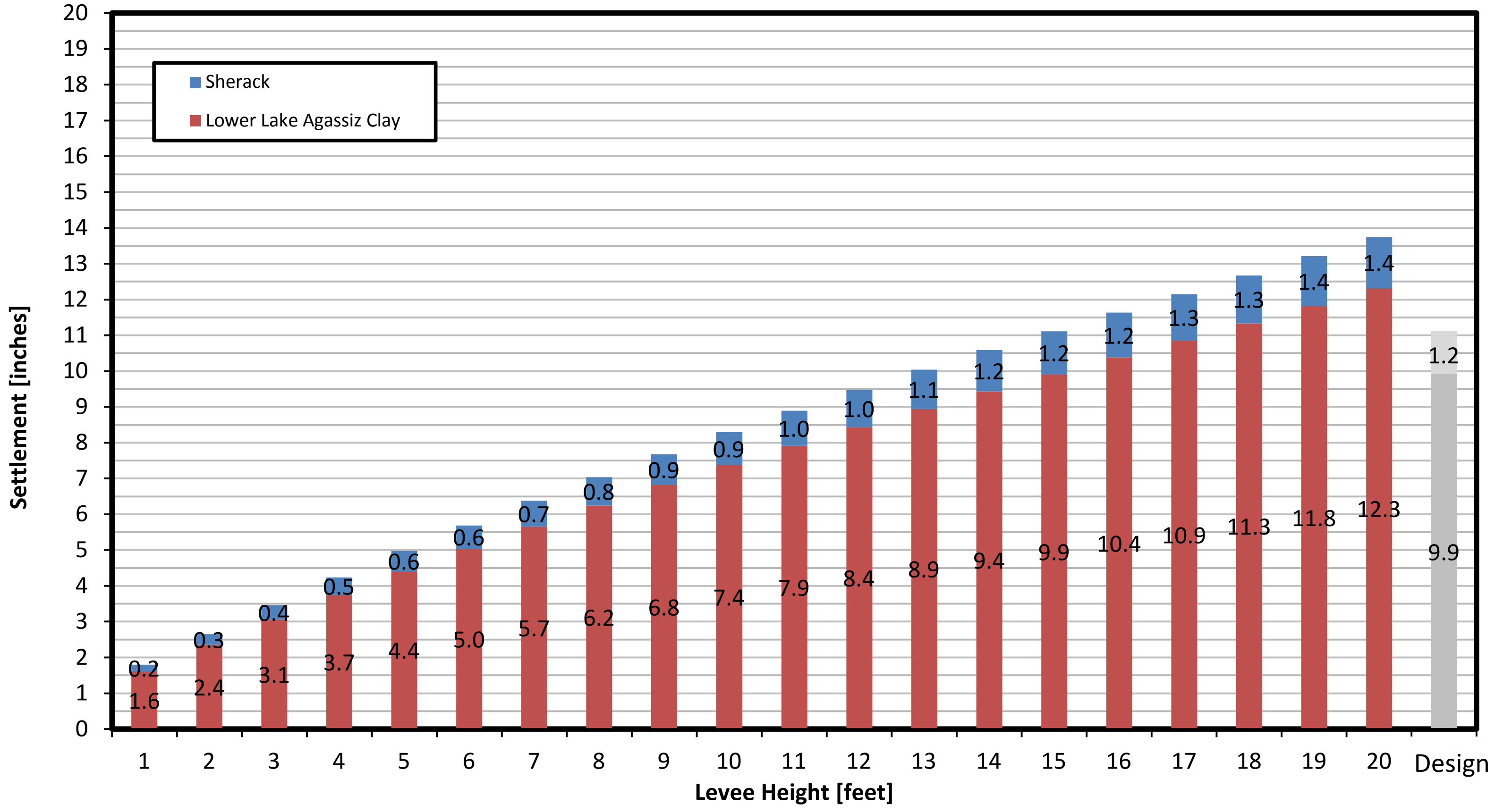


Secondary Compression

	Secondary Compression [in]	Time for end of primary [yr]
Sherack	0.08	0.45
Lower Lake Agassiz Clay	0.83	45.93
Total	0.91	

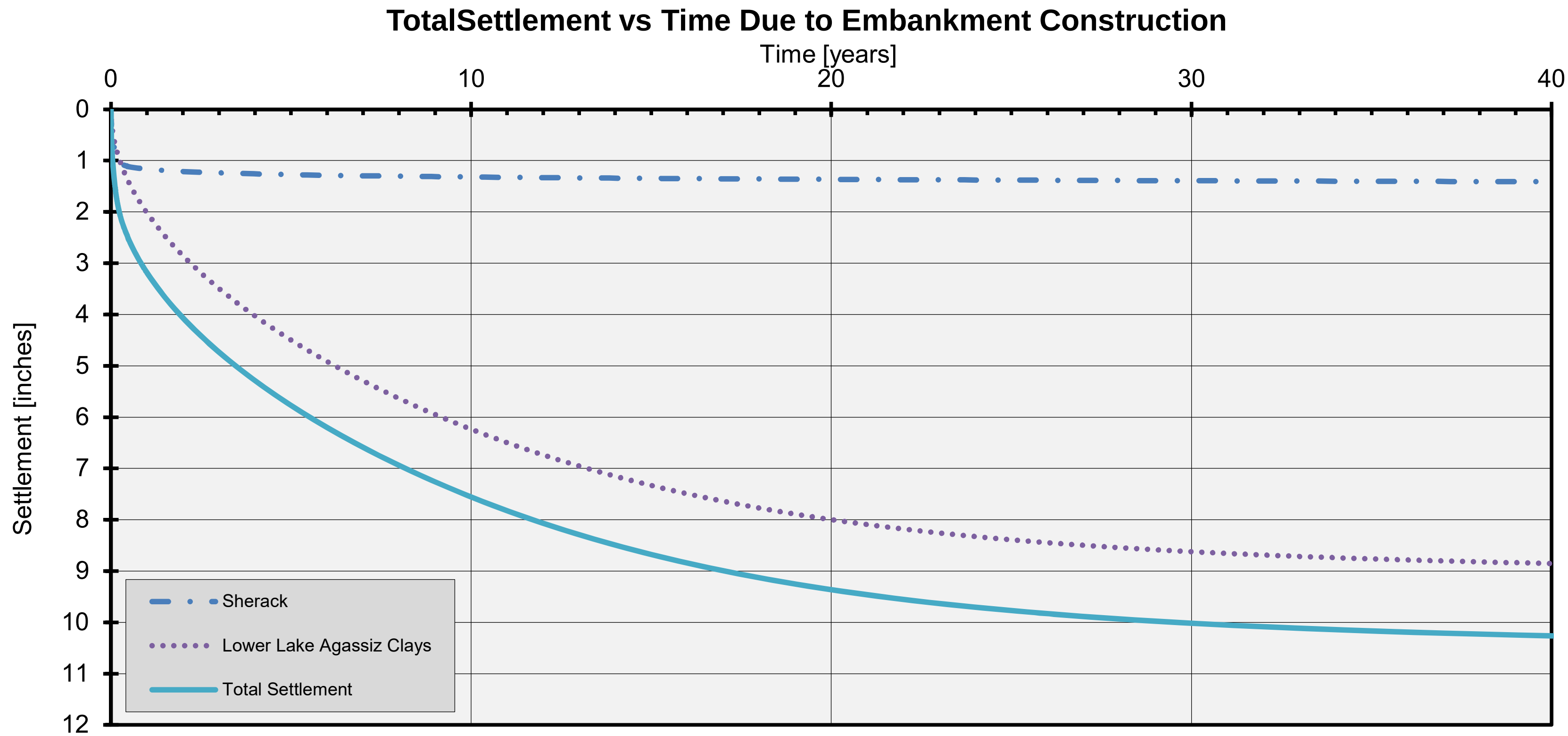
Layer Name	Coefficient of Secondary Compression		$\Delta e = S_c*(1+e_o)/H$	Void Ratio at End of Primary
	C_α	C'_α		e_p
Sherack	0.0045	0.0026	0.510	0.76
Lower Lake Agassiz Clay	0.0046	0.0024	0.360	0.94

Wild Rice River Primary and Secondary Compression per Geologic Unit



Total Settlement

Barr Engineering Company
Settlement Analysis Spreadsheet
June 16, 2014



Primary and Secondary Settlement																						Design Value
Embankment Height (ft)		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	15
Settlement (in)	Sherack	0.2	0.3	0.4	0.5	0.6	0.6	0.7	0.8	0.9	0.9	1.0	1.0	1.1	1.2	1.2	1.2	1.3	1.3	1.4	1.4	1.2
	Lower Lake Agassiz Clays	1.6	2.4	3.1	3.7	4.4	5.0	5.7	6.2	6.8	7.4	7.9	8.4	8.9	9.4	9.9	10.4	10.9	11.3	11.8	12.3	9.9
Total Settlement (in)		1.8	2.6	3.5	4.2	5.0	5.7	6.4	7.0	7.7	8.3	8.9	9.5	10.0	10.6	11.1	11.6	12.1	12.7	13.2	13.7	11.1

Attachment D

Wick Drain Design

Prefabricated Vertical Drain Design

Reference: ***Federal Highway Administration. U.S. Department of Transportation, Volume 1: Engineering Guidelines. Report No. FHWA/RD-86/168, August 1986.***

Barr Engineering Company
Wick Drain Design Spreadsheet

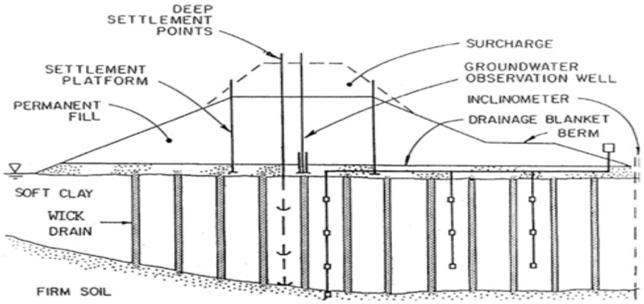
June 17, 2014

Page 1

Assumptions:

1. Radial drainage only
2. No backpressure from horizontal drainage
3. Soil parameters constant in time and position
4. No drain resistance considered
5. No disturbance considered
6. Embankment and surcharge loadings occur instantaneously for purposes of settlement calculations
7. Stability of the embankment is considered in separate analysis not presented here (i.e. staged loading)

Figure 2, page 6.



Inputs:

Depth to Groundwater =	10	ft
Top Width of Embankment =	150	ft
Slope of Sides (xH:1V) =	3	H:1V
Height of Embankment =	15	ft
Unit Weight of Embankment =	115	pcf
$U_{settlement}$ =	99	%
Depth to Bottom of Formation =	73	feet
Construction time =	180	days

Wick Design:

Spacing 1 =	8.00	ft
Spacing 2 =	7.00	ft
Spacing 3 =	6.00	ft
Spacing 4 =	5.00	ft
Spacing 5 =	4.00	ft
Equivalent Drain Diameter =	0.18	ft
Final Pressure, p_f =	1725	psf
$U_{AVERAGE}$ =	70	%

Geologic Units:	Sherack	Ox. Brenna	Brenna	Argusville
Depth to Top of Formation:	0	15	25	38
Depth to Bottom of Formation:	15	25	38	73
Layer Thickness below Ground Water:	**5**	**10**	**13**	**35**
γ_{sat}	**109.7**	**109.1**	**112.9**	**108.6**
γ_{dry}	75	73	79	73
γ_m	**109**	**108**	**112**	**108**
Moisture Content (%)	44.9	47.3	41.7	47.9
Gs	2.72	2.75	2.77	2.73
C_c	0.360	0.570	0.490	0.540
C_r or C_s	0.110	0.150	0.120	0.140
Void Ratio, e_o	1.27	1.34	1.19	1.34
C_v (ft^2/s)* :	6.79E-02	7.07E-02	1.30E-01	7.95E-02
Saturation	**96.46**	**97.23**	**97.34**	**97.82**
C_R =	**0.159**	**0.244**	**0.224**	**0.231**
R_R =	**0.049**	**0.064**	**0.055**	**0.060**

* Based on anticipated stress under the embankment

Prefabricated Vertical Drain Design

Reference: Federal Highway Administration. U.S. Department of Transportation, Volume 1: Engineering Guidelines. Report No.

Barr Engineering Company
Wick Drain DesignSpreadsheet

June 17, 2014

Design Methodology:

Evaluate the effects of the proposed embankment

Calculate effective stress increases under centerline due to stress increases under centerline due to embankment

Develop stress history and stress change profile

Predict total settlement due to embankment loads

Initial undrained settlement does not affect drains

Primary consolidation	10.2	inches	0.85	feet
Secondary compression	0.9	inches	0.08	feet
Total Settlement	11.1	inches	0.93	feet

Evaluate Required Surcharge

Estimate required height of surcharge to remove 99% of Primary Consolidation

In situ Pressure at center of layer, p_o =	2,373	psf
p_f / p_o =	0.73	
p_s / p_f =	0.73	
p_s =	1256	psf
Surcharge Height =	11	ft
New Embankment Height =	26	ft

Predict Primary Consolidation due to Embankment and Surcharge

Design Spacing =	7	feet
Anticipated Settlement to occur in =	165	days
Settlement =	11.1	inches

Wick Spacing [feet]	Consolidation [inches]	Surcharge Height [feet]
10	26	25
9	22	30
8	19	16
7	16	11
6	14	7
5	12	4
4	11.5	3

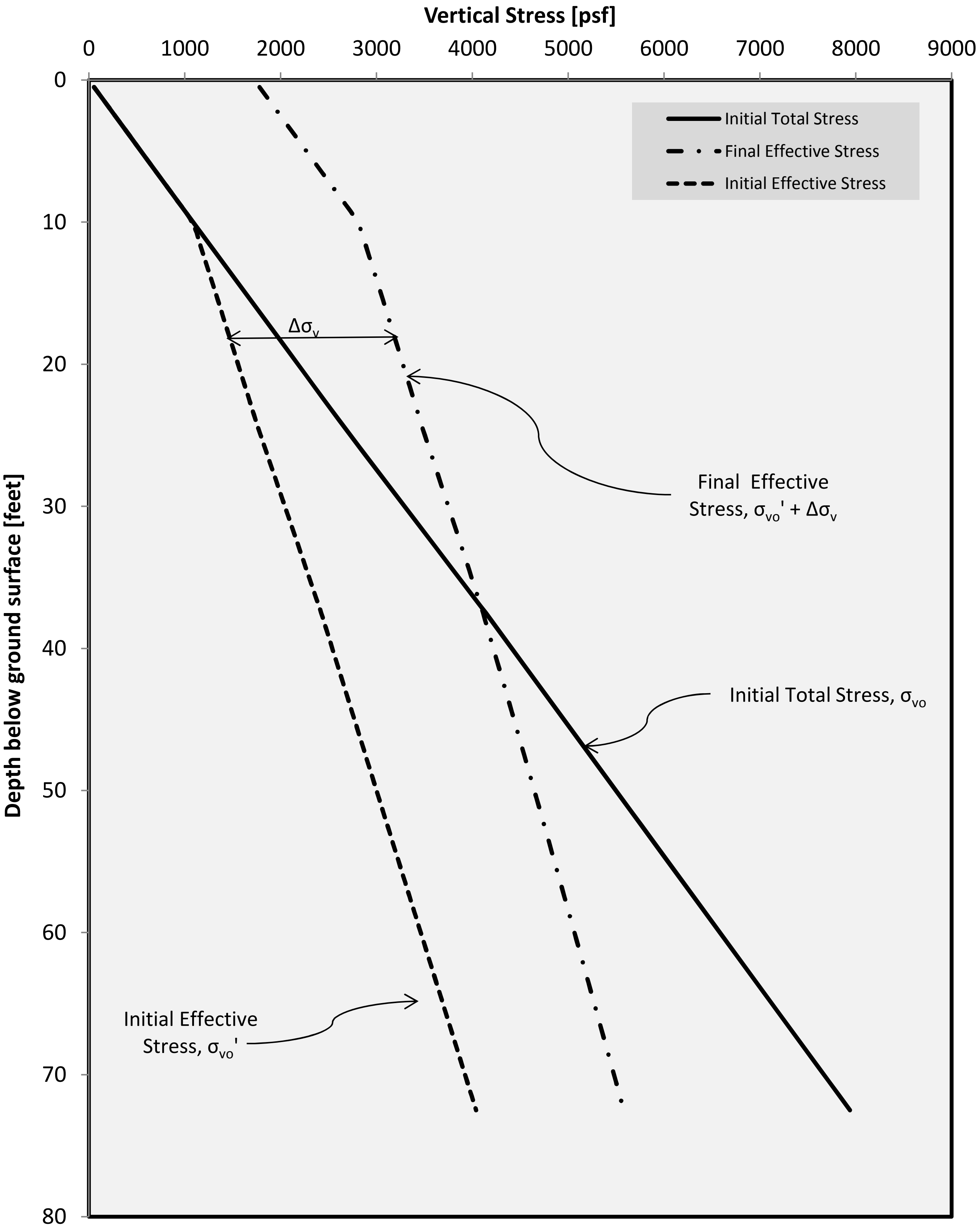


FIGURE 1

Stress History and Stress Change Profile

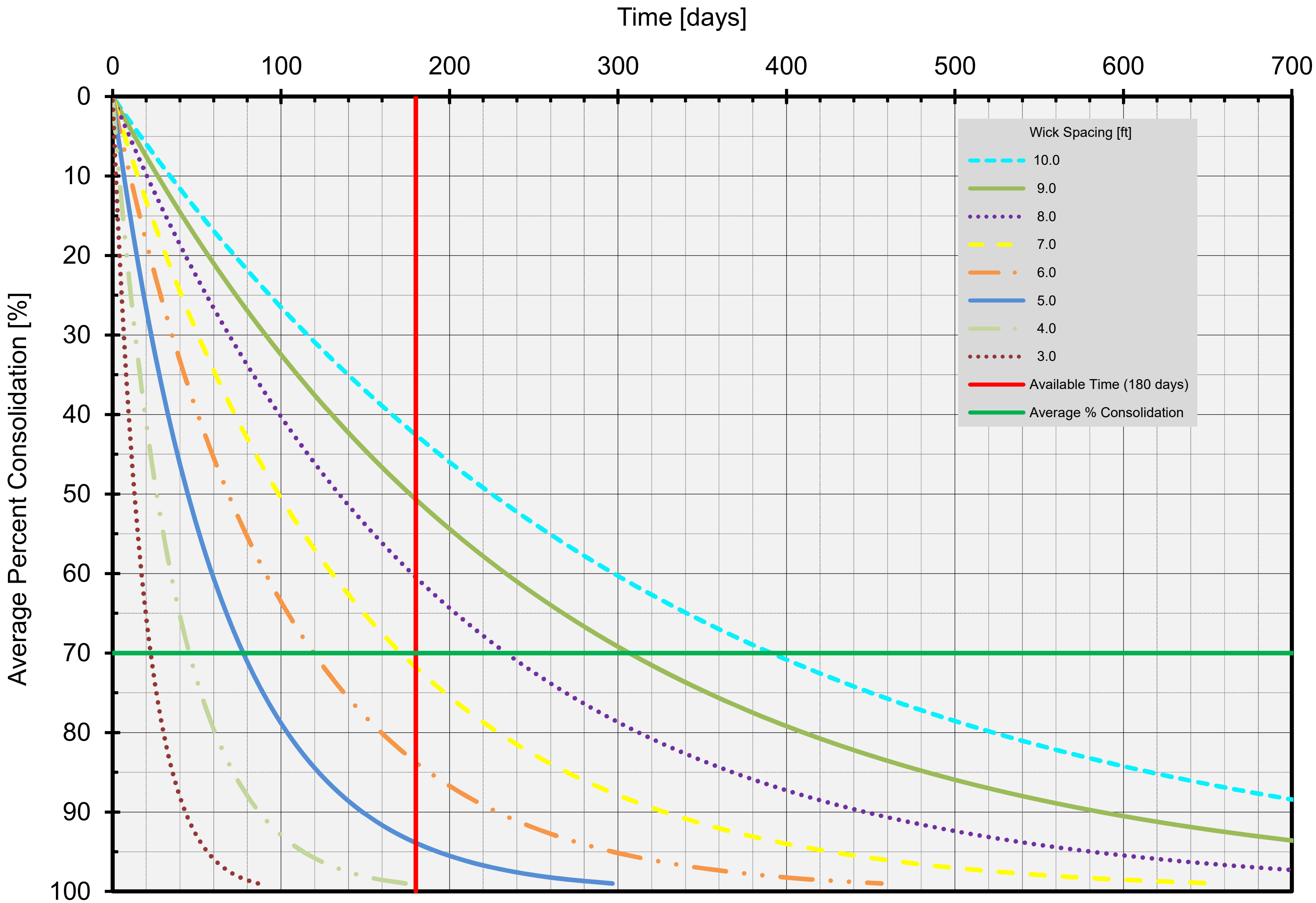


FIGURE 2
Time versus Consolidation Due to 11 Feet of Surcharge Load and Wick Drains
Embankment 7 - 15 feet

Prefabricated Vertical Drain Design

Reference: ***Federal Highway Administration. U.S. Department of Transportation, Volume 1: Engineering Guidelines. Report No. FHWA/RD-86/168, August 1986.***

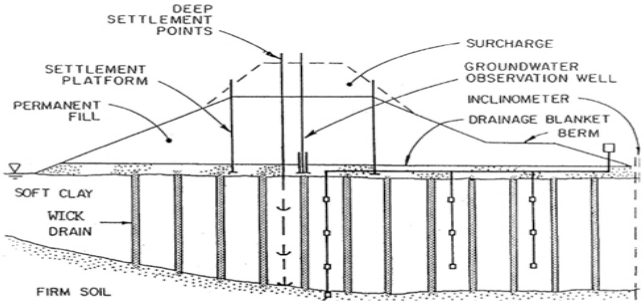
Barr Engineering Company
Wick Drain Design Spreadsheet

June 17, 2014

Assumptions:

1. Radial drainage only
2. No backpressure from horizontal drainage
3. Soil parameters constant in time and position
4. No drain resistance considered
5. No disturbance considered
6. Embankment and surcharge loadings occur instantaneously for purposes of settlement calculations
7. Stability of the embankment is considered in separate analysis not presented here (i.e. staged loading)

Figure 2, page 6.



Inputs:

Depth to Groundwater =	10	ft
Top Width of Embankment =	150	ft
Slope of Sides (xH:1V) =	3	H:1V
Height of Embankment =	7	ft
Unit Weight of Embankment =	115	pcf
$U_{settlement}$ =	99	%
Depth to Bottom of Formation =	73	feet
Construction time =	180	days

Wick Design:

Spacing 1 =	8.00	ft
Spacing 2 =	7.00	ft
Spacing 3 =	6.00	ft
Spacing 4 =	5.00	ft
Spacing 5 =	4.00	ft
Equivalent Drain Diameter =	0.18	ft
Final Pressure, p_f =	805	psf
$U_{AVERAGE}$ =	64	%

Geologic Units:	Sherack	Ox. Brenna	Brenna	Argusville
Depth to Top of Formation:	0	15	25	38
Depth to Bottom of Formation:	15	25	38	73
Layer Thickness below Ground Water:	5	10	13	35
γ_{sat}	109.7	109.1	112.9	108.6
γ_{dry}	75	73	79	73
γ_m	109	108	112	108
Moisture Content (%)	44.9	47.3	41.7	47.9
Gs	2.72	2.75	2.77	2.73
C_c	0.360	0.570	0.490	0.540
C_r or C_s	0.110	0.150	0.120	0.140
Void Ratio, e_o	1.27	1.34	1.19	1.34
C_v (ft^2/s)* :	6.79E-02	7.07E-02	1.30E-01	7.95E-02
Saturation	96.46	97.23	97.34	97.82
C_R =	0.159	0.244	0.224	0.231
R_R =	0.049	0.064	0.055	0.060

* Based on anticipated stress under the embankment

Prefabricated Vertical Drain Design

Reference: Federal Highway Administration. U.S. Department of Transportation, Volume 1: Engineering Guidelines. Report No.

Barr Engineering Company
Wick Drain DesignSpreadsheet

June 17, 2014

Design Methodology:

Evaluate the effects of the proposed embankment

Calculate effective stress increases under centerline due to stress increases under centerline due to embankment

Develop stress history and stress change profile

Predict total settlement due to embankment loads

Initial undrained settlement does not affect drains

Primary consolidation	5.5	inches	0.46	feet
Secondary compression	0.8	inches	0.07	feet
Total Settlement	6.3	inches	0.52	feet

Evaluate Required Surcharge

Estimate required height of surcharge to remove 99% of Primary Consolidation

In situ Pressure at center of layer, p_o = 2,373 psf

p_f / p_o = 0.34

p_s / p_f = 0.88

p_s = 709 psf

Surcharge Height = 6 ft

New Embankment Height = 13 ft

Predict Primary Consolidation due to Embankment and Surcharge

Design Spacing = 7 feet

Anticipated Settlement to occur in = 165 days

Settlement = 6.2 inches

Wick Spacing [feet]	Consolidation [inches]	Surcharge Height [feet]
10	15	17
9	13	13
8	10.5	8.5
7	9	6
6	8	4
5	7	2
4	6.5	1.5

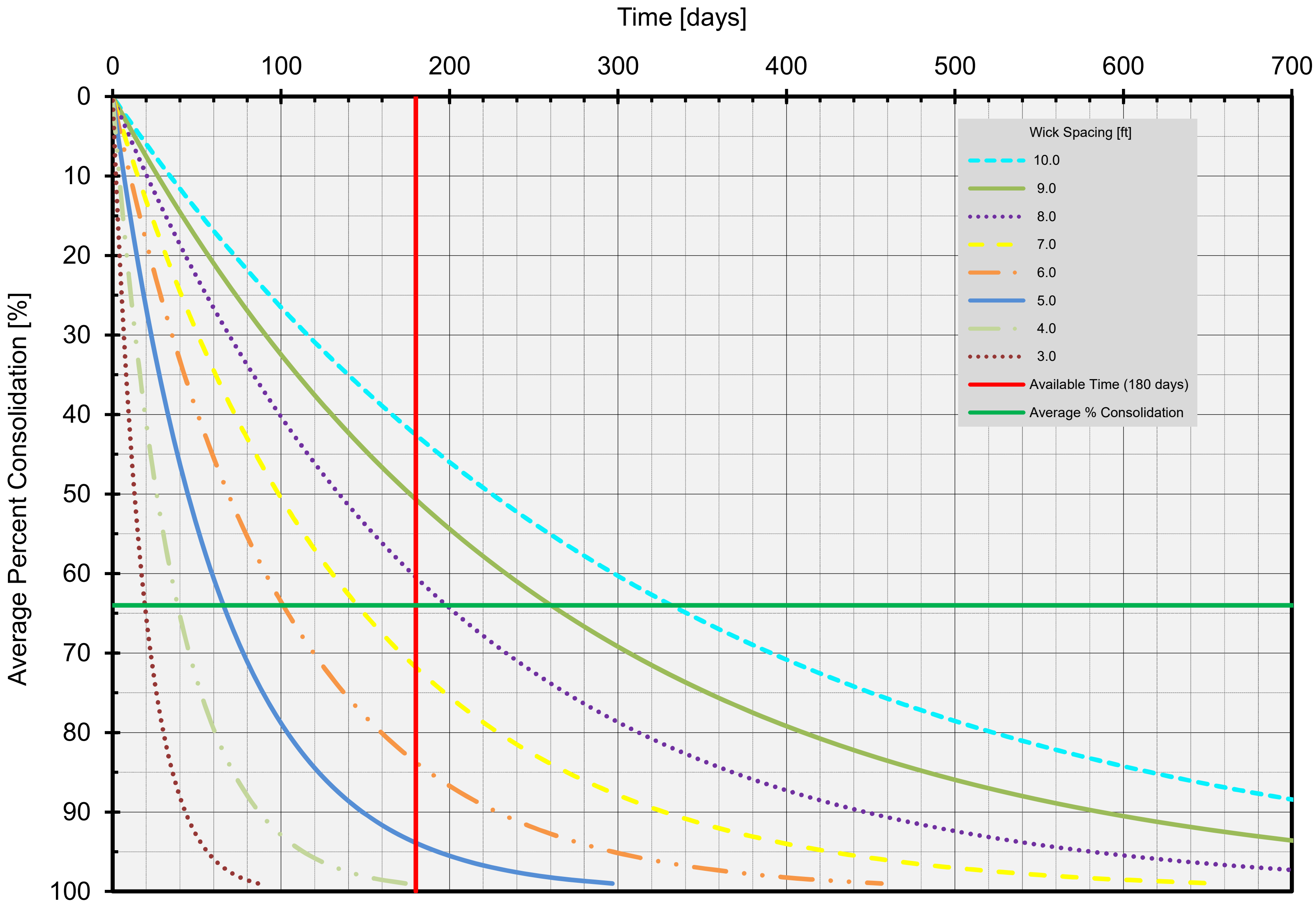


FIGURE 3
Time versus Consolidation Due to 6 Feet of Surcharge Load and Wick Drains
Embankment 0 - 7 feet

APPENDIX C – WILD RICE CONTROL STRUCTURE / CR16 INTERCHANGE OPINION OF COST

Appendix C

Opinion of Probable Cost for CH17 and WRR Relocation

Date: July 25, 2014

Note: see previous cost estimate reports for supporting documentation for full basis of cost estimate

Feature Code	Description	Alt. 0 Project Cost		Alt. 3 Project Cost		Alt. 5 Project Cost	
1	**LANDS & DAMAGES**						
	ROW and Easements	Revise for CH17/I29 Interchange and WRR Micrositing Analysis Extents	14,156,494	Revise for CH17/I29 Interchange and WRR Micrositing Analysis Extents	12,180,263	Revise for CH17/I29 Interchange and WRR Micrositing Analysis Extents	10,859,888
	MITIGATION AREA EASEMENTS						
2	**RELOCATIONS**						
	UTILITY RELOCATIONS						
	ROADWAY BRIDGES, ROAD RAISES RAISES & LOCAL ROAD CONSTRUCTION						
	ROAD RAISES						
		Settlement Mitigation (4800 LF I-29 Grade Raise)	3,846,621	Settlement Mitigation (3700 LF I-29 Grade Raise)	2,657,052	Settlement Mitigation (1900 LF I-29 Grade Raise)	1,546,788
		I-29 Road Raise (4800 LF of the 4.48 miles total)	6,484,545	I-29 Road Raise (3300 LF of the 4.48 miles total)	4,116,813	I-29 Road Raise (1900 LF of the 4.48 miles total)	2,045,094
		I-29 Bridges (2)	3,550,000	I-29 Bridges (2)	3,550,000	I-29 Bridges (2)	3,550,000
		Cass County 16 Road	2,653,643	Temporary Bypass & Temporary Bridge	21,889,058	Temporary Bypass & Temporary Bridge	21,950,817
		Cass County 16 Bridge (1)	1,500,000	Seeding & Erosion Control	320,000	Seeding & Erosion Control	320,000
		Frontage Road	377,986			I-29 Bridges (2) over Wild Rice River	Not Included
		Northbound Entrance Ramp	981,467				
		Northbound Exit Ramp	579,734				
		Southbound Entrance Ramp	600,140				
		Southbound Exit Ramp	847,856				
		Temporary Bypass & Temporary Bridge	21,889,058				
		Seeding & Erosion Control	320,000				
	LOCAL ROAD CONSTRUCTION						
6	**FISH AND WILDLIFE FACILITIES**						
8	**ROADS, RAILROADS AND BRIDGES**						
9	**CHANNELS AND CANALS**						
	Hydraulic Structure at Wild Rice River	Assume no cost change	20,700,000	Height is 1.5 feet lower	20,509,200	Height is 3.0 feet lower	19,746,000
	Site Work - Hydraulic Structure at Wild Rice River	Larger extent (500+ acres)	31,800,000	Larger extent (500+ acres)	35,100,000	Larger extent (500+ acres)	38,600,000
11	**LEVEES AND FLOODWALLS**						
14	**RECREATION FACILITIES**						
30	**PLANNING, ENGINEERING & DESIGN (PED)**						
	PED	Use 15% (Ph4)	14,419,658	Use 15% (Ph4)	13,221,318	Use 15% (Ph4)	13,163,805
31	**CONSTRUCTION MANAGEMENT (CM)**						
	CM	Use 7% (Ph4)	6,729,174	Use 7% (Ph4)	6,169,949	Use 7% (Ph4)	6,143,109
	Total[1][2][3][4][5][6]	**$**	**131,000,000**	**$**	**120,000,000**	**$**	**118,000,000**
		Alt. 0		**Alt. 3**		**Alt. 5**	

[1] Limited Design Work Completed (5%).

[2] Quantities Based on Design Work Completed.

[3] Unit Prices Based on Information Available at This Time.

[4] Limited Soil Boring and Field Investigation Information Available.

[5] Based on Preliminary Project Alignment Definition.

[6] Lands and Easements includes 25% contingency consistent with Task Order #6 Draft Real Estate cost estimate. Construction features, PED and CM includes 26% contingency consistent w/ Phase 4.

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APPENDIX D – TECHNICAL MEMORANDUM: CONCEPT-LEVEL HYDRAULIC EVALUATION OF THE WILD RICE RIVER CONTROL STRUCTURE

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Appendix D

Technical Memorandum DRAFT

To: Gregg Thielman and Lee Beauvais – Houston-Moore Group, LLC
From: Erik McCarthy & Brandon Barnes
Subject: Concept-Level Hydraulic Evaluation of the Wild Rice River Control Structure
Date: October 30, 2013
Project: 34-09-1004.13-001-100

The Wild Rice River Control Structure micrositing alternatives are investigated with the intent of characterizing the hydraulic tradeoffs of each alternative relative to the Recommended Alternative (VE13A) as Presented in the Supplemental EA. The I-29 South interchange conceptual design was included as part of Flood Diversion Authority Task Order 14. As part of the conceptual design, several alternative locations for the Wild Rice River (WRR) control structure were considered, and a Technical Memorandum regarding *Micrositing Alternatives Screening* (HMG, August 16, 2013) provided qualitative comparisons of the alternatives that led to the selection of three locations for further analysis.

- Alternative 0. The conceptual design for Alternative 0 is shown in Figure 1. The location of the control structure on the WRR was generally established based on the VE-13A alignment.
- Alternative 3. The conceptual design for Alternative 3 is shown in Figure 2. The location of the hydraulic structure for Alternative 3 is located south of Alternative 0 approximately 2,300 feet south of the I-29 interchange.
- Alternative 5. The conceptual design for Alternative 5 is shown in Figure 3. The location of the hydraulic structure is located approximately 1,200 feet west of I-29.

The August 16, 2013 *Micrositing Alternatives Screening* memorandum also presents an initial list of design constraints and considerations for the interchanges and WRR control structure. The list includes three design constraints related to the hydraulic design of the WRR control structure and the staging area:

- Maintain the Post Feasibility Southern Alignment Analysis (PFSAA) staging elevation (922.2) within +/- 0.1 feet for the 1-percent and 0.2-percent chance flood events
- Maintain equal pools on both sides of I-29 for the 1-percent chance flood event
- Assume the connecting channel west of the WRR control structure is required and has the same cross section as presented in the PFSAA

This memorandum presents a concept level hydraulic evaluation of each of the three alternatives relative to the hydraulic design constraints set for the WRR control structure. The draft Phase 7.1 HEC-RAS model for the 1-percent event provided by HMG on September 5, 2013 was used along with the conceptual layout of each alternative (Figure 1 through Figure 3). The evaluation of the WRR control structure considered the following items.

1. What impact does the location of the WRR control structure have on maintaining the PFSAA staging elevation?
2. What impact does the location of the WRR control structure have on the height of the gates and top of structure?
3. What opening is required to maintain equal pools east and west of I-29?
4. Does moving the WRR structure west of I-29 impact flows east/west of I-29?

What impact does the location of the Wild Rice River control structure have on maintaining the PFSAA staging elevation?

Alternative 0 is located along the VE-13A alignment, is similar to the conceptual design presented in the PFSAA report, and is the alternative modeled in the Phase 7.1 HEC-RAS model. Therefore, the staging elevation for the 1-percent event for Alternative 0 was taken from the HEC-RAS model. Following input from USACE and HMG, the 0.2-percent event was not considered for this evaluation.

The levee alignment for Alternative 3 and Alternative 5 is relocated to the south. These alternatives were not modeled in HEC-RAS. The impacts to the staging elevation were estimated by computing the storage volume that will be removed from the staging area due to the levee relocation, and then applying that volume equally over the remaining staging area. The underlying assumption is that to maintain acceptable downstream impacts, the volume of water in the staging area will not change due to relocation of the WRR control structure, and that the operation of the gated structures (including not only at the WRR, but also at the Red River of the North (RRN) and the inlet to the diversion) is not modified. Table 1 provides and a summary of impacts to staging elevation for each alternative.

Table 1. Impact to 1-percent Staging Elevation

	Row	Alt. 0	Alt. 3	Alt. 5
Volume Removed from Staging Area (acre-feet)[1]	1	0	3,191	5,249
Surface Area Removed from Staging Area (acre)	2	0	251	419
Remaining Staging Area Surface Area (acre)[1]	3	31,708	31,456	31,289
Impact to Staging Elevation (feet)	*Row1/Row3*	*0.0*	*0.1*	*0.2*

[1] Staging area was estimated for elevation 922.4, which was the staging area from the HEC-RAS model provided by HMG. This elevation is 0.2-feet higher than the elevation listed in the PFSAA report.

Based on the HEC-RAS model provided by HMG, the 1-percent chance peak water surface elevation in the staging area for Alternative 0 is 922.4 ft, which exceeds the design constraint (maximum elevation of 922.2 +/- 0.1-feet) listed in the August 16, 2013 *Micrositing Alternatives Screening* technical memorandum. It is assumed that the discrepancy is due to model updates that have been incorporated since the PFSAA, and the peak staging elevation for Alternative 0 can be reduced if operation of gated structures on the RRN, WRR, and the inlet to the diversion are optimized. At this concept level, the peak water surface elevation will increase an additional 0.1- to 0.2-feet above Alternative 0 for Alternative 3 and Alternative 5, respectively. Again, the optimizing of the control structures may reduce the increase in the peak staging elevation to within the design constraint. However, it is unclear what effect this will have on downstream impacts. Evaluation of downstream impacts were not listed as a design constraint, but should be considered when deciding to remove storage volume from the staging area. It is also unclear how much further the staging area will extend upstream. Evaluation of the staging area extent should also be considered during the next phase of design.

What impact does the location of the Wild Rice River control structure have on the height of the gates and top of structure?

The current design of the WRR control structure includes two gates each 30-feet wide and 25-feet tall. The top of the embankment elevation is 930.0-feet which is approximately equal to the peak water surface elevation of the PMF plus 5-feet of freeboard. The impact to the height of the gates and embankment adjacent to the WRR control structure was evaluated based on available information.

The impacts to the gate and embankment height were estimated by computing the storage volume that will be removed from the staging area due to the WRR control structure relocation, then applying that volume equally over the remaining staging area, and subtracting the change in invert location due to the relocation of the structure. Table 2 provides and a summary of impacts to the height of the embankment and gate for each alternative.

Table 2. Impact to Height of Gates and Embankment

	Row	**Alt. 0**	**Alt. 3**	**Alt. 5**
Volume Removed from Staging Area (acre-feet)[1]	1	0	3,844	6,338
Surface Area Removed from Staging Area (acre)	2	0	251	419
Remaining Staging Area Surface Area (acre)[1]	3	36,388	36,137	35,969
Increase in Water Surface Elevation due to relocating WRR structure	*Row 4 = Row1/Row3*	0.0	0.1	0.2
Gate Invert Elevation (feet)	*5*	887.0	888.6	890.2
Change in Gate Invert (feet)	6	0.00	-1.6	-3.2
Top of Structure Elevation (feet)	*7*	927.8	927.9	928.0
Structure Height (feet)	*8*	40.8	39.3	37.8
Impact to Gate/Embankment Height	*Row4 + Row6*	0.00	-1.5	-3.0

[1] Staging area was estimated as elevation 925.0. This is the approximate elevation of the PMF event in the staging area for Alternative 0.

As summarized in Table 2, relocating the structure could result in a small reduction to the required height of the gates or embankment. However, this should be verified by simulation of the PMF event with the current version of the HEC-RAS model during the next phase of design.

What opening is required to maintain equal pools east and west of I-29?

The HEC-RAS model considers that flow across I-29 is conveyed through the I-29 bridge openings and (2) 5x4 box culverts on the WRR with invert elevations at 906.5, approximately 1.5 miles upstream of the I-29 bridge. These openings provide sufficient capacity to equalize the peak water surface elevation on either side of I-29 during the 1-percent chance event (HMG Technical memorandum, August 2, 2013).

It should be noted however, that while the openings included in the HEC-RAS model equalize the peak staging elevation during the 1-percent chance event, the pool elevations on either side of I-29 may not be equal over the rise and fall of the hydrograph. Indeed, the maximum difference during this event occurs on the receding limb of the hydrograph when the water surface elevation west of I-29 is 1.0-foot higher than the water surface elevation to the east of I-29 (Figure 4). It is recommended that the design team considers what the maximum acceptable difference in water surface elevation is over the duration of the hydrograph (not just the difference at the peak) when sizing the opening below I-29.

Does moving the WRR structure west of I-29 impact flows east/west of I-29?

The WRR control structure is located east of I-29 for Alternative 0 and Alternative 3. For these alternatives, both the WRR and RRN structures are located east of I-29. HEC-RAS simulations of the system have demonstrated that the pool elevation in the staging area may be controlled by operating the gates on the RRN alone. Gates at the WRR structure are lowered at the beginning of the event and remain lowered throughout the simulations. Operating the system in this manner assumes that the opening at I-29 will not be blocked while operating the system. However, if the opening below I-29 is blocked while operating the system, it could be impossible to maintain equal pools east and west of I-29 without diverting additional flow into the diversion channel or without staging additional water.

In Alternative 5, the WRR control structure is located west of the I-29 embankment. This location provides an advantage with respect to the other two alternatives when considering the overall resilience of the diversion system to potential failure modes. If the WRR control structure is located west of I-29, there is less reliance on the I-29 bridge opening to equalize the pool elevations in the staging area. If the bridge opening is plugged while operating the system, the pool elevation west of I-29 may be controlled by operating the WRR gates, while the pool elevation east of I-29 may be controlled by operating the RRN gates. Future construction sequencing/phasing plans should consider if there are advantages or disadvantages associated with constructing the WRR control structure west of I-29.

General Observations

Some general observations and questions can be made regarding the HEC-RAS model following the concept level hydraulic evaluation of the WRR control structure.

- The staging elevation for the 1-percent event is a design constraint defined in the PSFAA, and should not increase or decrease by more than 0.1 feet from 922.2. Based on the HEC-RAS model provided by HMG, the 1-percent chance peak water surface elevation in the staging area for Alternative 0 is 922.4 ft, which exceeds the design constraint (maximum elevation of 922.2 +/- 0.1-feet, HMG, August 2, 2013). As previously indicated, it is assumed that the discrepancy is due to model updates that have been incorporated since the PFSAA.

- The HEC-RAS model includes a set of gate operations that is very different from what was used in feasibility through Phase 4. For instance, the gate operation on the RRN control structure assumes there is an "instantaneous" opening of the gates that allows flow to increase from ~8,500 cfs to ~17,000 cfs into town. While this is possible to simulate in the model, an instantaneous opening when the head differential across the gates exceeds ~20-feet and releases 10,000 cfs seems hard to accomplish, and could lead to increased potential for erosion downstream of the hydraulic structure. In reality, the gates will likely not be operated in this fashion, and so the model results, particularly the staging elevation, may not be accurate.

Figures

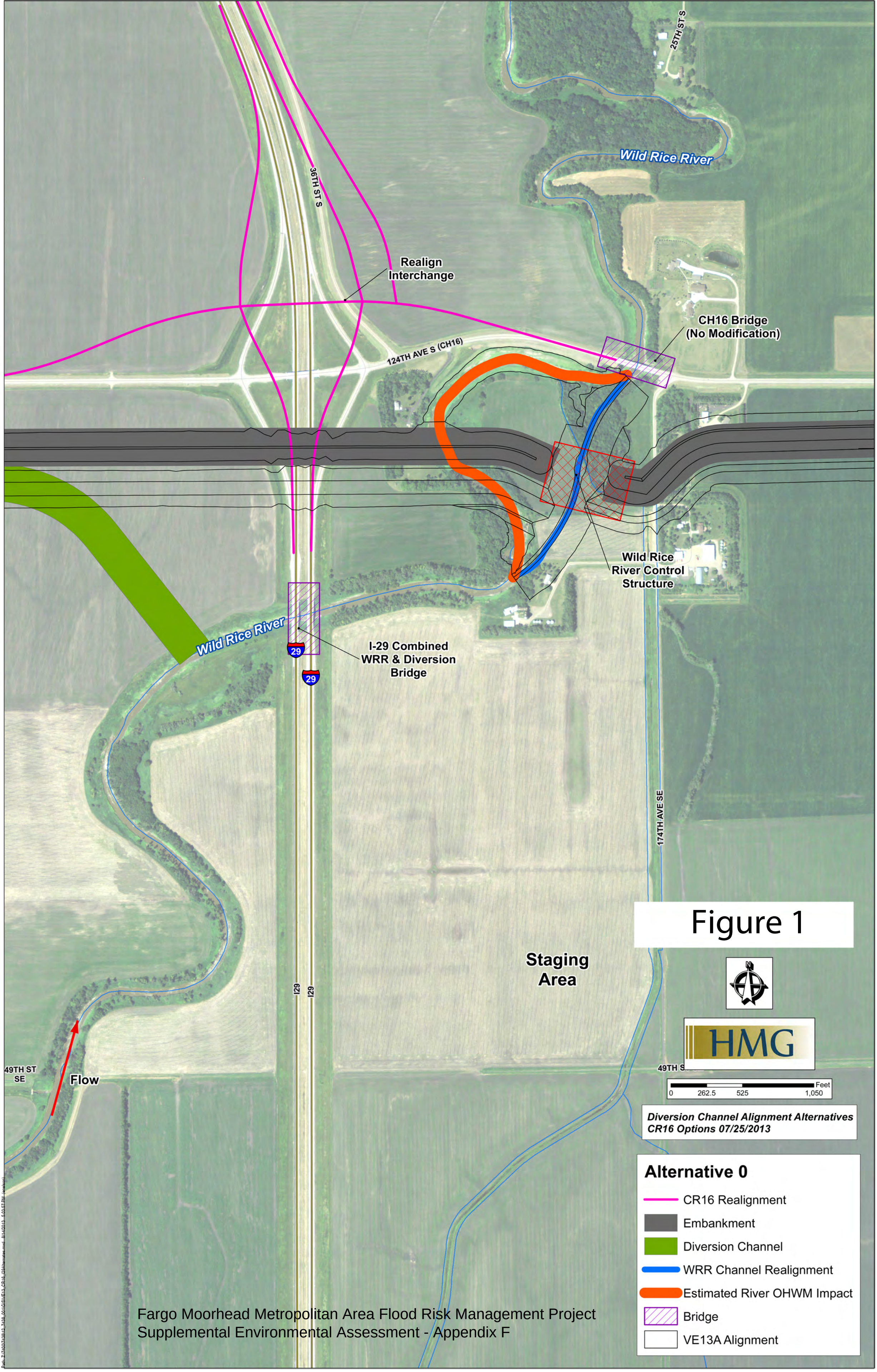

Figure 1

Diversion Channel Alignment Alternatives
CR16 Options 07/25/2013

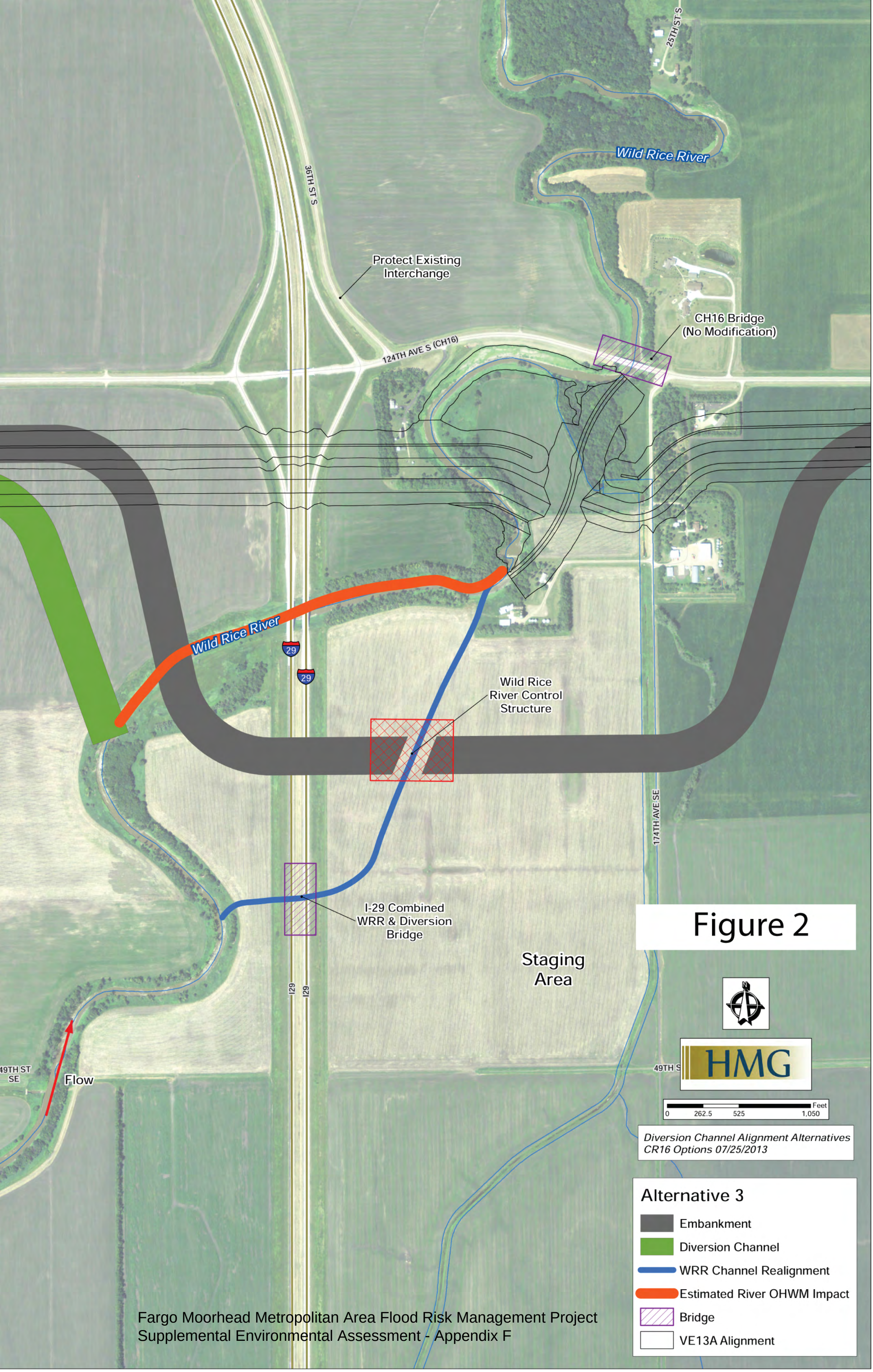

Figure 2

Diversion Channel Alignment Alternatives CR16 Options 07/25/2013

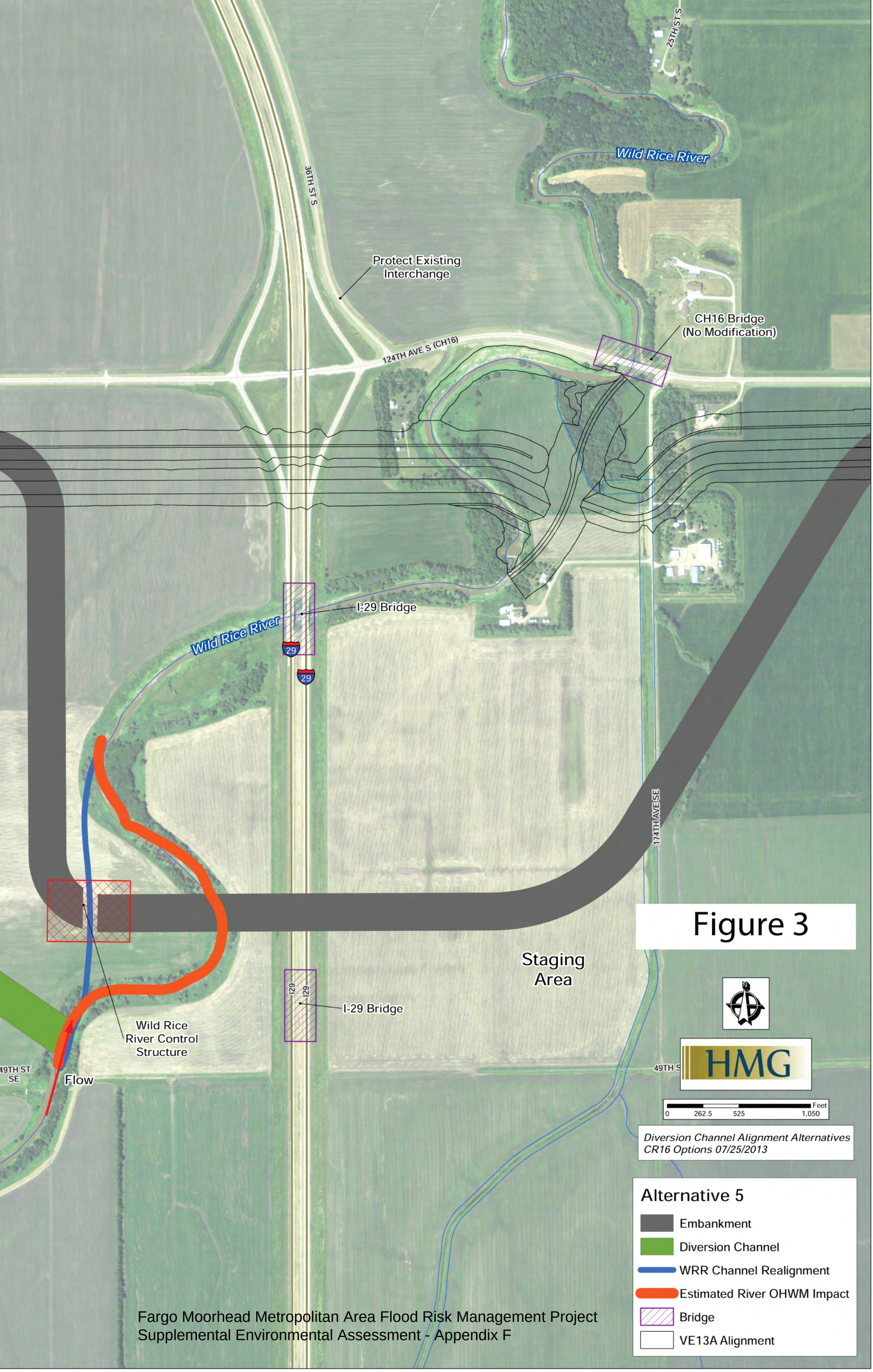

Figure 3

Diversion Channel Alignment Alternatives
CR16 Options 07/25/2013

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Figure 4. Difference in Pool Elevation East and West of I-29 during the 100-Year Event