Attachment I-15: Reviews

Agency Technical Review of Phase 2 and Phase 3 Documents

January 2010

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August 2010

Comment Report: Discipline Specific Comments
Project: **Fargo-Moorhead Metro Feasibility**
Review:**For the ATR - AFB Document**
(sorted by Discipline , ID)
Displaying 29 comments for the criteria specified in this report.

Id	Discipline	Section/Figure	Page Number	Line Number
3094401	Geotechnical	n/a'	I-2	n/a

(Document Reference: Appendix I; Topography)

"The Proposed Project" referred to in paragraph 7 and subsequent discussions in paragraphs 7, 8, and 9 only discuss the in-town levee alternative. Recommend similar overview of topography be included for the diversion alternatives.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0 Evaluation **Concurred**
The topography discussion (page I-2) was revised to indicate the topography for the In-town levee alignments. Paragraphs were added to discus the topography along the ND and MN diversion channel alternatives (page I-3). As additional borings are obtained, the geology discussion will be revised as needed during Phase 3.

Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10

1-1 Backcheck Recommendation **Close Comment**
Closed without comment.

Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10

Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3094405	Geotechnical	n/a'	I-7/8	n/a

(Document Reference: Appendix I; Seismic Risk and Earthquake History)

Seismic Risk and Earthquake History - the relevance of low risk for seismic activity should be expanded to demonstrate weather or not seismic loads will control any design. ER 1110-2-1806 and ER 1110-2-1150 require that preliminary design seismic motions and a preliminary evaluation of key features be addressed at the feasibility stage.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0 Evaluation **Concurred**
A seismic evaluation to determine expected ground motions will be completed and added to the report during Phase 3.

Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10

1-1 Backcheck Recommendation **Close Comment**
Closed without comment.

Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10

Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3094408	Geotechnical	n/a'	I-9	n/a

(Document Reference: Appendix I; Borings)

Clarify the sampling technique and document relevant factors for interpreting field test results – verify that the blows recorded on the logs are only those produced with the standard split spoon and not the modified spoon used for continuous sampling. Also, include information regarding the type of hammer used and its efficiency so adjustments may be made to the field blow counts if needed for future analyses.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0	Evaluation **Concurred** Clarifications have been added to indicate how the sampling was completed using the modified and standard spilt spoons. The blow counts on the drafted logs are from the standard SPT. Autohammers were used during sampling. (pages I-9 & I-10) Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3094412	Geotechnical	n/a'	I-10	n/a

(Document Reference: Appendix I; Selection of Design Parameters)

The validity of using the ultimate strength failure criteria as an indication of a materials "post-creep" strength is not understood since changes in the insitu stress state due to creep effects would occur prior to shear. Reduced strengths due to creep effects would best be determined by back-evaluating full scale slope failures known to have been preceeded by soil creep. However, the rationale for using ultimate strength parameters as a conservative shear strength estimate can be justified on the prudent assumption that significant portions of the critical failure surfaces will be indicative of progressive failure through the Brenna soils and it is therefore unlikely that its peak-strength will be mobilized simultaneously along all points throughout the potential failure surface. Recommend that statement be clarified.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0	Evaluation **Concurred** The first sentence states that the Lake Agassiz soils tend to creep over time when loaded. "Creep" in this context was in reference to the progressive failure of a slope overtime, in which a small amount of movement occurs suddenly and then the slope continues to move at a slower rate over time. The selection of post-peak strength was based on a number of reasons: 1) previous Corps projects which used the same criteria, 2) Experience in the Red River Valley with expected progressive failure, 3) test data indicating a brittle stress-strain response. The paragraph has been revised to more clearly identify the reasons for selecting drained ultimate strength parameters. The word "creep" has also been removed. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10
1-1	Backcheck Recommendation **Close Comment**

	Closed without comment. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3094421	Geotechnical	n/a'	I-12	n/a

(Document Reference: Appendix I; In-Town Levee Alternative, Features)

A short descriptive of the conceptual levee being analyzed (materials cross section, side slopes, crest width, range of heights, etc) and some discussion identifying areas along the alignments where suspected ground modifications (cut or fills) could require extensive evaluations should be included in the final Phase 3 report. Phase 3 refinements should also include short descriptions and relevant information pertaining to conceptual loading and elevations (sizes) for any proposed floodwalls, pump stations, closure structures, or major drainage structures.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0	Evaluation **Concurred** A short description of a typical levee will be included (10' top width, 1V on 3H side slopes, 6' deep inspection trench). Discussion on other features will also be included. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10
1-1	Backcheck Recommendation **Close Comment** Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10
1-2	Backcheck Recommendation **Close Comment** Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3094425	Geotechnical	n/a'	I-12/17	n/a

(Document Reference: Appendix I; Geotechnical Design of Feasibility Alternatives; In-Town Levee Alternative)

A brief discussion should be included in Phase 3 to addresses the long-term steady-state seepage and rapid drawdown stability requirements, the necessity (or lack thereof) for seepage cutoff or seepage control, adequacy of proposed embankment materials and their source, settlement considerations, and any assumed constructability issues in order to identify potential unconventional cost issues and demonstrate that these issues have been considered in the study and will not govern levee design.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0	Evaluation **Concurred** These issues would not likely be a concern in the consideration of a levee alternative but a brief discussion can be included to address the long-term steady-state seepage & rapid drawdown requirements, seepage control measures, embankment fill, consolidation, and constructability. Verbiage along these lines will be added: Due to the relatively low permeability of the soils in the Red River Valley and duration of the flood events, long-term steady-state seepage is not

expected to develop. In the same token, stability of levees due to rapid drawdown has not been seen as an issue in the Red Rive after flooding. Based on this, these conditions do not control the levee design. In addition, seepage control/cutoffs are not generally required unless there is some pervious material near the ground surface. The exploration program did not reveal any pervious materials.

Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10

1-1 Backcheck Recommendation **Close Comment**
Closed without comment.

Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10

Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3094427	Geotechnical	n/a'	I-12/17	n/a

(Document Reference: Appendix I; Geotechnical Design of Feasibility Alternatives; In-Town Levee Alternative)

Commentary should be included to discuss the relevant geotechnical considerations for potential seepage issues around, and the foundations supporting floodwalls, closure structures, pumping plants and major drainage structures. A preliminary structure foundation type and potential need for foundation treatment should be discussed to support the rough cost basis for these structures.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0 Evaluation **Concurred**
The discussion concerning seepage issues and foundation treatments will be included and discussed in the paragraphs describing these structures. Due to the relative impermeable nature of the materials generally encountered in the Red River Valley, seepage is not a major concern.

Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10

1-1 Backcheck Recommendation **Close Comment**
Closed without comment.

Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10

Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3094428	Geotechnical	n/a'	I-18	n/a

(Document Reference: Appendix I; Geotechnical Design of Feasibility Alternatives; Minnesota Diversion Channel Alternatives; Design Sections)

Figures detailing the assumed strata thicknesses and excavation depths at each of the four sections would be helpful.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0
Evaluation **Concurred**
A generalized stratigraphy of the four MN Diversion Channel sections will be included in

	Attachment I-8. It will be similar to the assumed stratigraphy for the preliminary geotech analysis that was shown in the report, Figure I-1. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10
1-1	Backcheck Recommendation **Close Comment** Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3094431	Geotechnical	n/a'	I-18	n/a

(Document Reference: Appendix I; Geotechnical Design of Feasibility Alternatives; Minnesota Diversion Channel Alternatives; Design Sections) [**This item is flagged as a critical issue.**]

The discussion on the initial side slope selection should be expanded. What is the depth of the West Fargo Diversion Channel and how long has it been in service? What specific performance observations lead to this conclusion (i.e. was it dictated by performance during construction or long term maintenance needs, etc.). Also, what is the justification for going to steeper slopes at bridge locations.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0	Evaluation **Concurred** More details will be included on selection of side slopes: The depth of the WFDC is on the order of 10 feet, placing the bottom of the excavation in the Sherack formation or just into the Brenna formation. Stability analyses for the WFDC indicated that steeper slopes 1V on 5H would be acceptable. 1V on 7H slopes were selected to allow the side slopes to be mowed with standard farm equipment. Erosion of the WFDC at the toe of the slopes have lead to slope instability that had to be fixed. The steepness of the slopes at the bridge locations will be further evaluate during Phase 3. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10
1-1	Backcheck Recommendation **Close Comment** Potential impacts cannot be identified until further evaluation is done as a part of phase 3. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3094435	Geotechnical	n/a'	I-18	n/a

(Document Reference: Appendix I; Geotechnical Design of Feasibility Alternatives; Minnesota Diversion Channel Alternatives; Design Sections)

Clarification is needed on the second sentence: "Moore's analyses indicated that the quantity of excavated material decreased with increasing depth, meaning that the deeper excavated channels would likely be more cost effective because there was less materials to excavate"

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0	Evaluation **Concurred** The sentence has been clarified to indicate that for a given hydraulic capacity, the amount of excavation required decreases with increasing depth. It reads: "Moore's analyses indicated that for a given channel capacity, the quantity of excavated material decreased with increasing depth. This means that for a given channel capacity, the deeper the excavated channel, the more cost effective it is." Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3094436	Geotechnical	n/a'	I-20	n/a

(Document Reference: Appendix I; Geotechnical Design of Feasibility Alternatives; Minnesota Diversion Channel Alternatives; Slope Stability) [**This item is flagged as a critical issue.**]

The slope stability criteria cited in para 81 applies to levees. Since the consequence of failure for a diversion channel is different than a typical levee project, a discussion should be included to define and justify the selected design criteria (i.e. safety factors) and applicable load cases. In areas outside of proposed structures (bridges, diversion structures, major drainage structures), the consequence of failure is minimal and lower FOS targets can easily be justified if it is accepted that the O&M costs over the initial few years will be higher in order to maintain channel conditions until the long term state is reached. However, this is not necessarily the case around proposed structure locations and the potential for instability should be discussed in better detail to identify the need for foundation treatment. Although an effective stress analysis with pore pressures developed through Seep/W would appear to be the most fundamentally correct analysis, there is not enough groundwater information to calibrate the Seep/W model, especially at its most critical state for stability which occurs when pore pressures are at their highest. Additionally, the compacted fills for bridge approaches and spoil piles of excavated materials at the top of the slopes will generate additional pore pressures. It will likely take several years for all pore pressures to stabilize. Due to the uncertainties with the boundary conditions, it is not recommended that the pore pressures from SEEP/W model be used in the stability analyses. Recommend that a staged rapid drawdown stability analysis with a surcharge load be performed to identify the pertinent issues during and in the initial few years after construction. Long-term analyses, with or without a channel flooding, will be less stringent of a load case due to the lower pore pressures and will therefore, not govern design.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0	Evaluation **Concurred** During Phase 2, preliminary analyses were completed. The intent is to further evaluate the stability of the selected diversion channels during Phase 3 as indicated in the "Additional Work" Section. The target FOS will be reevaluated during Phase 3. Both global stability and localized stability will be evaluated. The local stability will be used to evaluated the shallow sloughing failures at the toe of the slope that could be a maintenance concern. The rapid drawdown type loading condition will be investigated in addition to the end-of-construction and long-term conditions that have currently be evaluated. The use of a surcharge load for the spoil material will be investigated. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10
1-1	Backcheck Recommendation **Close Comment** Potential impacts cannot be identified until surcharges are included in the refined analysis identifiied for phase 3. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3094437	Geotechnical	n/a'	I-21	n/a

(Document Reference: Appendix I; Geotechnical Design of Feasibility Alternatives; Minnesota Diversion Channel Alternatives; Results) [**This item is flagged as a critical issue.**]

A settlement analysis should be performed around proposed structure locations to identify potential issues requiring special foundation treatments, staged construction, or other special considerations that will affect foundation performance due to the application of a surcharge load, localized groundwater drawdown, and rebounding of the soils below the base of the channel. Recommend that this analysis be used to justify preliminary selection of pile type and anticipated tip elevations to overcome negative skin friction or require foundation treatment.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0	Evaluation **Concurred** For the second phase of the feasibility study, no detailed geotechnical analyses were completed for the hydraulic structures or bridges crossing the diversion channel. Information from previous projects and studies in the area were instead used to develop a conceptual foundation design for the hydraulic structures. The details of the conceptual design and the costs used are detailed in Appendix L, section 6.0. In the case of the bridges, a review of the past costs for bridges was completed. A unit price per square foot was developed from historical data. Further details can be found in Appendix L, section 4.2. The methodology used to estimate the costs for the hydraulic structures and bridges is thought to be reasonable for the this stage of the study. This methodology of estimating costs will be reevaluated during Phase 3 and refinement of the assumptions and conceptual designs will be made as necessary. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10
1-1	Backcheck Recommendation **Close Comment** Impacts cannot be identified until settlement analysis is performed as a part of phase 3. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3094438	Geotechnical	n/a'	I-23	n/a

(Document Reference: Appendix I; Geotechnical Design of Feasibility Alternatives; Minnesota Diversion Channel Alternatives; Groundwater Considerations) [**This item is flagged as a critical issue.**]

The Buffalo Aquifer is identified as a planning constraint, but little is presented in the report to demonstrate that the MN Diversion will have no impact on the Aquifer. The Aquifer is said to be recharged by the Buffalo River, which empties into the Red River just a few miles upstream of the diversion's terminus. Additionally, the piezometer in the sand and gravels at boring 09-14M suggests piezometric levels that extend as much as 20 feet above the proposed bottom of excavated elevation. A more detailed study of the groundwater from the Buffalo aquifer should be presented in order to demonstrate that this planning constraint is satisfied. Additional information is needed to demonstrate that a mile will provide a reasonable buffer between the aquifer and excavated diversion channel – ecspecially since it is reported that water levels in the aquifer are stated to have risen 15 feet over a 10 year period. This could potentially require additional piezometers, peophysical methods, or tracer testing to better map seepage from the aquifer.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0	Evaluation **Concurred**

	The potential effects that a diversion channel on an aquifer are being evaluated. A document has been prepared to discuss the affects of a diversion channel on ground water, wetlands, and aquifers. Disscussions with other agencies will be done to help determine what is required to further evaluate the aquifer. Additional borings will be taken along the MN Diversion channel and piezometers will be installed in critical locations. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10
1-1	Backcheck Recommendation **Close Comment** Findings of phase 3 work may have significant impact on study IF piez. levels in granular materials have strong correlation with Buffalo Aquifer water levels and remaining natural blanket thickness after excavation is inadeqaute. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3094450	Geotechnical	n/a'	I-22	n/a

(Document Reference: Appendix I; Geotechnical Design of Feasibility Alternatives; Minnesota Diversion Channel Alternatives; Uplift) [**This item is flagged as a critical issue.**]

The relevance of a SEEP/W analysis is questionable. Piping should not be a issue in clay soils due to low exit velocities and cohesion, and the pore pressures generated by the model are not reflective of the a conservative case for slope stability. Additional groundwater information would be needed to better define the boundary conditions if pore pressures need to be defined. Reliable uplift calculations could be performed by simplified analyses provided that the minimum blanket thickness and maximum piezometric pressures are characterized. Discussion should also address the effect that bridge foundations may have on the impervious blanket.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0	Evaluation **Concurred** The SEEP/W analysis was used to estimate pore pressures in the soil. Assumptions had to be made on the total head boundary conditions and the distance they where located from the diversion channel. The boundary conditions will be reevaluated with respect to observed groundwater and artesian pressures during Phase 3. A simplified analysis to determine the required minimum blanket thickness based on piezometric pressures can be completed during Phase 3. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10
1-1	Backcheck Recommendation **Close Comment** Impacts cannot be identified until aquifer effects are characterized and uplift analyized in Phase 3. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3094451	Geotechnical	n/a'	I-24	n/a

(Document Reference: Appendix I; Geotechnical Design of Feasibility Alternatives; Minnesota Diversion Channel Alternatives; Instrumentation)

The statement of a downward gradient of the flow of groundwater into the lower formations is based on the assumption of a continuous vertical flow path through all materials. There is a good potential that portions of the Brenna and

Argusville soils act as aquatards and the PZ observations are not reflective of a single groundwater table. It cannot be concluded that the PZ pressures in the lower granular materials are not hydraulically connected or being driven by the buffalo aquifer with the information presented.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0	Evaluation **Concurred** The piezometric data was included in the report to indicate that measurements are being collected to investigate the fluctuation of the ground water table, other subsurface piezometric levels, and if there was excess pressures in a confined sand layer. The piezometric data only covers about 2 months of data collections. Conclusions were not drawn from this data nor was it used in the analyses. Data is being collected through the use of data loggers. Due to the small number of readings, references to trends have be removed from the text. As more data is obtained, it will be evaluated and included in the Phase 3 writeup. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 17-Mar-10
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3094453	Geotechnical	n/a'	I-12/17	n/a

(Document Reference: Appendix I; Geotechnical Design of Feasibility Alternatives; In-Town Levees Setback Analysis)
[**This item is flagged as a critical issue.**]

The objective of the analysis is essentially to determine the position of the levee so as to not contribute to riverbank instability. However, most of the critical failure surfaces documented do not intersect the proposed levee area, meaning that the analysis is essentially an attempt to model the long-term stability of the existing river banks. The long term stability of the river banks are heavily influenced by the scour and deposition due to river dynamics, groundwater fluctuations and long term creep effects that cannot be accurately anticipated or characterized in the slope stability analyses. The creep, erosion and groundwater effects will take place regardless of the surcharge location and preliminary designs should identify revetments (instead of additional setback) that minimize riverbank erosion. The stability analyses to determine setback distances should tie to the top of the secondary bank. Recommend that a limited number of analyses be presented for composite soil stratigraphies that are considered conservative representations of similar reaches Review of ground surface profiles presented in Attachment I-5 consistently suggest that the average slope between the top of the secondary bank and the toe of the primary bank is about 1V on 7H. These observations suggests that the long-term stability of the riverbank could be modeled as a 1V on 7H slope (which is also significant for the diversion channel alternatives since their maximum depth of excavation is about equal to the height of the secondary bank above the riverbed). Recommend that analysis be replaced with one that determines setback distance by varying the horizontal distance of the surcharge from the top of the secondary bank (projected downward at a 1V on 7H slope) and performing an undrained analysis for the end of construction case. Past experience as well as a cursory analysis on these soils suggest that levees of this height will result in minimal setback distances from the top of the secondary slope and might be dictated more by riverside access for O&M considerations.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 26-Feb-10

1-0	Evaluation **Non-concurred** During Phase 2, the preliminary and revised analyses were completed in order to determine the setback of the levees. These analyses followed the methodology that has been used for other Corps projects within the Red River Valley. These analyses are conservative, but still appropriate to determine the costs associated with the in-town levee alternative which only considered levees. The in-town levees were not recommended for further evaluation during the

	screening process for a number of reasons: 1) top elevation is limited to highest natural ground, which is only to an elevation close to the 1% chance event, 2) due to the constraints of the maximum height there would be unacceptably high residual risks, 3) many structures would need to be removed which would have social impacts. If the in-town levees were retained during the screening process, the geotechnical analyses would have further been refined. In addition, an evaluation would have been completed to see if there were locations in which floodwalls could have been a cost effective solution. Better explanation of the analysis methodology and the screening process and its influence of the geotechnical design process will be included in the report. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 16-Mar-10
1-1	Backcheck Recommendation **Close Comment** Additional explanation on the rationale is needed in order to concur on the adequacy or relevance of the model presented for setback requirements. However, this comment is being closed based on the fact that in spite of favorable B/C ratios, the in-town levee alternative is being disconsidered due to limitations on its potential level of protection and the inherent residual risks. If the B/C ratio becomes a relevant factor, additional refinement or alternate analysis would be needed to accurately establish real estate requirements, delineate structure removals, and identify floodwalls and/or revetment needs. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 18-Mar-10
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3689363	Geotechnical	I.3.4.1	I-14	n/a

Paragraph is somewhat confusing. Suggest clarifying the 6th sentence statement "potential shear surfaces are only able to mobilize the ultimate shear strength". I think the key point is that because of the relatively high degree of strain-softening, it is unlikely that the peak shear strength will be mobilized along all points of the postulated shear surface simultaneously. It is therefore necessary to assume effective shear strength parameters that are based on the ultimate (post-peak) strength failure criteria to satisfy critical assumptions for limit equilibrium methods, as recommended in EM 1110-2-1902.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 05-Jan-11

1-0	Evaluation **Concurred** The paragraphs associated with the selection of design parameters have been rewritten to clarify what criteria was used for the different analyses and why. The rewritten paragraphs are as follows: I.3.4.1 The effective shear strength parameters used for the FMMFS are based on the ultimate (post-peak) strength failure criteria that equated to a strain of 15%. There are a number of reasons for this. First, ultimate strengths have been used for previous St. Paul District (MVP) projects within the Red River Valley. In addition, experience within the Red River Valley indicates that clays within this region are fissured and the weakest of these clays exhibit brittle stress-strain behavior. This can lead to progressive failure of the riverbanks and cut slopes, which is commonly seen. As a result of the brittle stress-strain behavior and progressive failure mechanism, the peak shear strength cannot be mobilized along the potential shear surfaces simultaneously. Also, experience indicates that large amount of strain (more than 10%) may occur in natural or cut slopes during the life time of the project. The effective stress shear strength test data indicates that if the materials exhibit brittle stress-strain response, the peak strength occurs typically between 3 and 8 percent strain. For those materials that do not exhibit a brittle stress-strain response, the maximum stress typically remains constant beyond 10% strain For these reasons, the effective stress shear strength parameters were based on the ultimate (post-peak) strength failure criteria for both the In-town Levee alternative and the Diversion Channel alternatives. Both R-bar and DS test results were used in the determination of the effective stress shear strength parameters. I.3.4.2 In the case of the total stress analyses, different criteria were used for the In-town Levee alternative than for the Diversion Channel alternatives. The peak undrained shear strength parameters were used when analyzing the end-of-levee construction condition. At the end-of-levee-construction, the clay soils will start to consolidate and dissipate excess pore pressures generated from the

	embankment loading. The clay will drain, but very slowly, due to the low hydraulic conductivity associated with clay minerals. In time, the clay soils will drain and all excess pore water pressures will have dissipated. At this time, the soil mass is said to be in a drained condition. During the process of draining, it is thought that the soils will experience strain of less than what is required to reach the peak undrained shear strengths. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 31-Jan-11
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 07-Feb-11
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3689370	Geotechnical	I.3.4.2	I-15	n/a

Not sure of accuracy of the strain estimate or how relevant it is since undrained tests exhibit brittle behavior. Consider deleting last sentence in paragraph to avoid confusion on which strengths were actually adopted for the undrained analysis

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 05-Jan-11

1-0	Evaluation **Concurred** The paragraphs associated with the selection of design parameters have been rewritten to clarify what criteria was used for the different analyses and why. The rewritten paragraphs are as follows: I.3.4.1 The effective shear strength parameters used for the FMMFS are based on the ultimate (post-peak) strength failure criteria that equated to a strain of 15%. There are a number of reasons for this. First, ultimate strengths have been used for previous St. Paul District (MVP) projects within the Red River Valley. In addition, experience within the Red River Valley indicates that clays within this region are fissured and the weakest of these clays exhibit brittle stress-strain behavior. This can lead to progressive failure of the riverbanks and cut slopes, which is commonly seen. As a result of the brittle stress-strain behavior and progressive failure mechanism, the peak shear strength cannot be mobilized along the potential shear surfaces simultaneously. Also, experience indicates that large amount of strain (more than 10%) may occur in natural or cut slopes during the life time of the project. The effective stress shear strength test data indicates that if the materials exhibit brittle stress-strain response, the peak strength occurs typically between 3 and 8 percent strain. For those materials that do not exhibit a brittle stress-strain response, the maximum stress typically remains constant beyond 10% strain For these reasons, the effective stress shear strength parameters were based on the ultimate (post-peak) strength failure criteria for both the In-town Levee alternative and the Diversion Channel alternatives. Both R-bar and DS test results were used in the determination of the effective stress shear strength parameters. I.3.4.2 In the case of the total stress analyses, different criteria were used for the In-town Levee alternative than for the Diversion Channel alternatives. The peak undrained shear strength parameters were used when analyzing the end-of-levee construction condition. At the end-of-levee-construction, the clay soils will start to consolidate and dissipate excess pore pressures generated from the embankment loading. The clay will drain, but very slowly, due to the low hydraulic conductivity associated with clay minerals. In time, the clay soils will drain and all excess pore water pressures will have dissipated. At this time, the soil mass is said to be in a drained condition. During the process of draining, it is thought that the soils will experience strain of less than what is required to reach the peak undrained shear strengths. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 31-Jan-11
1-1	Backcheck Recommendation **Close Comment** Closed without comment.

	Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 07-Feb-11
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3689378	Geotechnical	Appendix I-5	13-20	n/a

UU Strength Data plotted on pages 13-20 in appendix I-5; undrained shear stress is labeled as units of tsf instead of psf.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 05-Jan-11

1-0	Evaluation **Concurred** The indicated units of undrained shear stress have been changed to psf to match the plotted data and will be replotted for the March 2011 submittal. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 31-Jan-11
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 07-Feb-11
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3689382	Geotechnical	I.5.2.1.3	I-20	n/a

The first sentence is not accurate. EM 1110-2-1913 does address foundation stability due to new "flood barrier" loads (Section 6-5; Case I - End of Construction). Suggest editing this paragraph to point out that the design methodology developed by St Paul is based on a conservative pre-construction stability evaluation in order to develop confidence in a minimum setback distance.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 05-Jan-11

1-0	Evaluation **Concurred** The drained loading case addressed by EM 1110-2-1913 is Case 3, Steady seepage from full flood stage, which assesses the stability of the landside slope during a flood. The St. Paul District, through experience, has found that slope stability of the natural slope and determining a stable and reliable zone in which to construction the flood barrier is more critical. Both effective and total stress analyses are completed to determine the stability of the natural slope and flood barrier. In the case of the effective stress analysis, a low river water condition is used as natural slope failures tend to occur during periods of low water when the stabilizing force of the water on the natural bank is at its lowest. Section I.5.2.1 has been revised to better discuss the design philosophy used during the evaluation of the In-Town Levee alternative. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 31-Jan-11
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 07-Feb-11
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3689389	Geotechnical	I.5.2.2.3	I-21	n/a

What was the rationale for not extending the search riverward past the wet-side toe if the minimum factor of safety produced was at the search limits. It would be interesting to see what minimum factor of safety would be produced if the search on entry points extended further riverward of the wet-side toe to the secondary bank. This may help validate the limits assumed for the residual strength zone noted in paragraph I.5.6.2.2 if safety factors remain above 1.0, or it may help illustrate the level of conservatism in the analysis if such a search produces safety factors significantly less than 1.0. In either case, it would help frame the discussion for the adopted minimum target factor of safety (FS=1.2) stated in paragraph I.5.6.4.3.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 05-Jan-11

1-0	Evaluation **Concurred** The In-Town Levee alternative was evaluated during Phase 2 of the project. This evaluation was completed to get an understanding of what the potential setback distances could be for the levee alternative. It is recognized that the stability analysis completed during Phase 2 was the first effort in determining the setback requirements and that additional refinement would be required as this alternative moved forward. The decision was made at the end of Phase 2 that the In-Town Levee alternative would not be pursued any further during Phase 3 and thus no additional refinements were warranted. The rationale used to select the extents of the "entry" search limits within the footprint of the levee was to determine the slope stability factor of safety for the levee in respect to sliding down towards the river. The slope stability analysis required that minimum FS be obtained to ensure that the levee could be constructed in a location that remained stable both during construction and also long term. It is recognized that potential shear surfaces riverward of the wet-side toe of the levee would produce lower factors of safety. Shear surfaces riverward of the wet-side toe with lower factors of safety was deemed acceptable. It is recognized that shifting the "entry" search limits riverward would produce lower factors of safety and would provide insight into the conservatism of the stability analysis based on the assumed residual zone. During Phase 3 of the project, the levee alternative would have been refined. The refinements would have included actual slide locations instead of an assumed location. With actual known slide locations, back-analysis would have been completed to estimate the residual shear strength of the soils. The estimated residual shear strength would have then been used in the slope stability analysis used to determine the required setback distances. Paragraph I.5.2.2.3 was rewritten. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 31-Jan-11
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 07-Feb-11
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3689395	Geotechnical	1.5.9.2(3)	I-29	n/a

Suggest eliminating point 3 since the removal of structures could be minimized with a more refined setback analyses or with floodwalls supported by deep foundation systems.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 05-Jan-11

1-0	Evaluation **Non-concurred** A large number of structures would have to be removed to facilitate the construction of the levees. Even with a substantial shift of the levee alignment riverward, the levee would have an impact to the residents as this would reduce the size of their backyards and also obstruct their

	view of the river. This would be considered a social effect. Floodwalls would also have an impact to the residents along the river. Point (3) has been refined as such. (3) Many structures along the river would be impacted by the levee alternative. These impacts would range from reduction of the size of the back yards and obstruction of the view of the river to complete removal of the structure. This would have a social impact to the communities. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 31-Jan-11
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 07-Feb-11
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3689529	Geotechnical	Section 1.6.3.3; Table I-8;	n/a	n/a

The assumed ratio of kx to ky equal to 1 may be unrealistic. The horizontal saturated permeabilities listed are typical of approximations found in textbooks, however most natural deposits can be up to ½ to 3 orders of magnitude tighter in the vertical direction. Isotropic permeability may lead to nonconservative results by allowing more pore pressure to dissipate vertically than in reality, and result in lower piezometric heads near the slope face.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 05-Jan-11

1-0	Evaluation **Concurred** : It is recognized that for the upper foundation materials (i.e. Alluvium, Sherack, Poplar River) that these natural deposits probably have a horizontal permeability greater than the vertical permeability. For the Brenna and Argusville materials are massive and could be considered homogenous. Because of this the horizontal and vertical permeabilities would likely be very similar. Initially when setting up the seepage model, the permeability parameters were varied in order to obtain a piezometric line that seemed reasonable. Varying the horizontal to vertical permeability did not lead to a piezometric line that seemed reasonable. It was found that if the ky to kx ratio was set to 1 in the materials above the till formation and that the till formation permeability was 2 orders of magnitude greater than the Brenna and Argusville formations, that a higher piezometric line was obtained that was judged to be reasonable. This was carried forward through the Phase 3 design. A sensitivity analysis was conducted to determine what affect a ky:kx ratio would have on the stability results. The sections with the lowest FSs were checked; three sections for each diversion alternative. It was found that the slope stability factors of safety obtained when a ky:kx ratio of 1/5 was slightly lower than when using a ky:kx ratio of 1. Generally, the FSs were reduced by less than 1% but remained above the required FS of 1.4. When a ky:kx ratio of 1/10 was used, the FSs increased above those found using a ky:kx ratio of 1. A portion of Section I.6.3 was rewritten to include additional discussion. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 31-Jan-11
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 07-Feb-11
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3689536	Geotechnical	I.6.4.1.3	n/a	n/a

Although this is not the rapid draw-down condition referenced in EM 1110-2-1902, groundwater effects on a slope face are similar during saturated excavation. The highest pore pressures will occur in the soil immediately after excavation as a result of the localized groundwater drawdown, and from the initial support of the spoil pile load. The report should include some evaluation relating to the constructability of the excavations in order to determine if special phasing or other specialized dewatering methods will be needed to limit pore water pressures near the slope during construction. Finite element procedures in Geostudio might approximate these instantaneous pore pressures for use in an effective stress slope stability analysis at various stages (depths) of excavation. The groundwater lowering process could be modeled through transient seepage analyses, but a sigma analyses may be needed to evaluate the additional pore pressure generated from the weight of the spoil pile. Such a model could also help identify critical areas and pressure thesholds to be monitored with instrumentation during test excavations.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 05-Jan-11

1-0 Evaluation **Concurred**
The drained and undrained analyses completed provide for a ranged of expected performance and is deemed adequate for the design of the channel excavation. A staged excavtion type analysis was contemplated but considered more complex than required for the feasibility study for a number of reasons: (1) It was not felt that slope failures during construction would be an issue due to the flat slopes required based on the undrained and long-term analyses. (2) The spoil pile is located 50 feet from the top of the slope and there would be minimal effect on the pore pressures adjacent to the slope. (3) The factors of safety required for levee design were used as the target FSs in the design of the channel in order to obtain a higher degree of certainty in maintaining stability. (4) The side slopes on the diversions are considerable flat, being 1V on 7H. Section I.6.4.1 was revised to discuss in better details the analyses competed. A recommendation has been included that the staged excavation type analysis be completed during the planning phase.

Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 31-Jan-11

1-1 Backcheck Recommendation **Open Comment**

Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 07-Feb-11

1-2 Backcheck Recommendation **Close Comment**
Closed without comment.

Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 14-Mar-11

Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3689548	Geotechnical	Attachment I-13	n/a	n/a

The bottom of channel elevations used in the uplift analysis are several feet higher than provided in Table I-15 and as shown for the low flow channel in Attachment I-10.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 05-Jan-11

1-0 Evaluation **Concurred**
The bottom elevations in Table I-15 for the Sections 4A through 8 are incorrect. The elevations reported are based on the original invert elevation and did not consider the 4-foot invert raise that was required based on the uplift calculations. The profiles in Attachment I-13 indicate the original invert elevation and did not include the revised invert due to the 4-foot raise. Table I-15 and Attachment I-13 will be revised indicate the correct invert based on the 4-foot raise.

Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 31-Jan-11

1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 07-Feb-11
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3689554	Geotechnical	Para I.6.8.3	n/a	n/a

An increasing trend in aquifer water levels is noted in paragraph I.7.0.2 and is apparent on page 9 of Attachment I-13. The uplift analysis should consider long term piezometric projections. It may also be useful to develop piezometric elevation contours from the available data since the confined nature of the aquifer below the diversion channel may not limit projected heads to less than the current ground surface elevation.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 05-Jan-11

1-0	Evaluation **Concurred** Engineering judgment was used in predicting what the future piezometric elevation could be in the aquifer. It is noted that since 1992, the MN DNR observation wells have shown a significant increase in the piezometric level in the aquifer; up to a 20-foot raise. Most of this occurred between 1992 and 2005, with the rate of increase slowing down after 2005. The piezometer clusters installed by the COE in 2009 and 2010 in areas near the proposed diversion alignment are indicating piezometric levels in the aquifer 9 to 14 feet below the ground surface. There is no current modeling being completed to project what piezometric levels may be in the future. Based on the MN DNR observation well levels and the COE piezometer clusters, a piezometric level of 7.5 feet below the ground surface was deemed appropriate. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 31-Jan-11
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 07-Feb-11
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3690096	Geotechnical	I.6.8.5	n/a	n/a

The materials classified in Boring 10-102M are named as a SP-SM, which is a Poorly graded sand with silt. Materials of this classification have very little fines (less than 12%) and should be considered pervious, which does not make the analysis overly conservative.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 05-Jan-11

1-0	Evaluation **Concurred** I agree that the results of the uplift calculation for boring 10-102M is not overly conservative. The explanation was meant to indicate that a minor change to the alignment could be done to eliminate the concern of uplift and to solely base the design of the MN Diversion on boring 10-102M would be over conservative. Will remove the "overly conservative" statement. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 31-Jan-11
1-1	Backcheck Recommendation **Close Comment**

	Closed without comment. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 07-Feb-11
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3690105	Geotechnical	Appendix I	n/a	n/a

There is no geotechnical discussion of the slurry cutoff wall that is called for between stations 700+00 to 815+00 (Civil Sheets CS104, CS303, and CS304). A cutoff wall may pose a issue with the excavations since it may impede drainage of the upper soils during construction. Suggest consideration be given to relief wells to reduce the pressure at the base of the blanket.

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 05-Jan-11

1-0	Evaluation **Concurred** The slurry cutoff wall was one of the alternatives discussed to deal with the uplift problems caused by the aquifer. The slurry cutoff wall was required to isolate a small portion of the aquifer so that relief wells could be installed in the vicinity of the channel bottom to relief the pressure. The aquifer needed to be isolated to allow for the installation of the relief wells, otherwise the relief wells could potentially have a drastic affect on the entire aquifer. The slurry cutoff wall alternative was replaced with the invert raise alternative and will be indicated on the drawings. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 31-Jan-11
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 07-Feb-11
	Current Comment Status: **Comment Closed**

Id	Discipline	Section/Figure	Page Number	Line Number
3690113	Geotechnical	Appendix I	n/a	n/a

Boring logs 10-97M; 10-99M, 10-100M, 10-101M and 10-104M are not provided

Submitted By: David Sobczyk ((402) 995-2249). Submitted On: 05-Jan-11

1-0	Evaluation **Concurred** Boring logs will be included. Submitted By: Kurt Heckendorf (651-290-5411) Submitted On: 31-Jan-11
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: David Sobczyk ((402) 995-2249) Submitted On: 07-Feb-11
	Current Comment Status: **Comment Closed**

Report Complete

Independent External Peer Review Comments

March 26, 2010 to August 31, 2010

Comment Report: Discipline Specific Comments
Project: **Fargo-Moorhead Metro Feasibility**
Review:**For the FMM - IEPR**
(sorted by Discipline , ID)
Displaying 2 comments for the criteria specified in this report.

Id	Discipline	DocType	Spec	Sheet	Detail
3276729	Geotechnical	Other	High	n/a	n/a

(Document Reference: Comment #1)

There are insufficient geotechnical analyses to justify the proposed channel slopes, channel depth, spoil pile configuration, cost estimates, and real estate requirements for the North Dakota Diversion Alternative.

(Attachment: Fargo-Moorhead_IEPR_Comment_1.doc)

Submitted By: Julian Digialleonardo (561-656-6303). Submitted On: 19-May-10

1-0 Evaluation **Concurred**

Concur - adopted The USACE recognizes the fact that minimal geotechnical analyses were completed for the MN diversion alternative during Phase 2 and these analyses were presented in the geotechnical appendix. No geotechnical analyses were completed for the ND Diversion alternative during Phase 2. Further, the USACE realized that additional evaluations would be performed during Phase 3 on the NED plan and/or LPP. Following completion of the Phase 2 draft report, additional geotechnical analyses were completed for the ND Diversion channel alignment along with revisions to the MN Diversion channel alternative. These analyses and results have been coordinated with the hydraulics, structural, and cost estimating disciplines. The diversion channels and hydraulic structures are analyzed hydraulically taking into account the geotechnical requirements of the diversion channels. The excavation quantities will be based on the geotechnical/hydraulic analyses and the cost estimate revised. The final report will include the additional geotechnical slope stability analyses. ND Diversion Channel: Nine reaches along the ND Diversion Channel were analyzed. The geotechnical analyses indicated that to obtain adequate factors of safety for slope stability, the invert of the diversion channel needed to be raised 3 feet and a bench included in the channel slope. The bench is required to be 10 feet high above the bottom of the channel with a 1V on 10H side slope to the channel bottom, and a minimum of 50 feet in width. The spoil piles are setback 50 feet from the top of the diversion slope. The results of the ND Diversion Channel analyses are summarized in Attachment 1. MN Diversion Channel: During Phase 2, the MN Diversion Channel was separated into four reaches and geotechnical analyses completed. During Phase 3, the geotechnical analyses were revised. The MN Diversion Channel stratigraphy and ground surface profile were reviewed and eleven separate reaches were determined. The geotechnical analyses indicated that benching of the slope was required to obtain adequate factors of safety along a majority of the diversion channel. The benching requirement for the MN Diversion Channel was set as follows: 7 feet high above the bottom of the channel with a 1V on 10H side slope to the channel bottom, and a minimum of 70 feet in width, the spoil piles setback 50 feet from the top of the diversion slope. The results of the MN Diversion Channel analyses are summarized in Attachment 1.

Submitted By: Aaron Snyder (651-290-5489) Submitted On: 21-Jun-10 (Attachment: FMMFS_IEPR_Comment_Responses_Attach_1.pdf)

1-1 Backcheck Recommendation **Close Comment**

Concur. In all of the design cross-sections it appears that the spoil piles are located a distance of 50 feet away from the top of the channel excavation. This appears to be an arbitrary assumption that is suitable for feasibility level analysis but could be refined during the final design. It is suggested that during final design a parametric analysis of the setback distance for the spoil piles in combination with the depth of the spoil piles be conducted to optimize the overall configuration. This analysis should include evaluation of stability and unit costs to arrive at the optimal configuration of the spoil pile setbacks and depth of spoil. This analysis should consider the cost impacts of the transportation of the spoil away from the excavation, the additional real estate requirements for spoil pile width and the need to meet required factors of safety. This parametric analysis should be conducted for each design cross-section to ensure that the most economical configuration is obtained based on all relevant costs and stability configuration.

	Submitted By: Julian Digialleonardo (561-656-6303) Submitted On: 06-Jul-10
	Current Comment Status: **Comment Closed**

Id	Discipline	DocType	Spec	Sheet	Detail
3276734	Geotechnical	Other	High	n/a	n/a

(Document Reference: Comment #2)

The stability of the channel slopes, foundation deposits, and related spoil piles should be evaluated using ultimate or near ultimate soil strength values for the End of Construction (EOC) condition.

(Attachment: Fargo-Moorhead_IEPR_Comment_2.doc)

Submitted By: Julian Digialleonardo (561-656-6303). Submitted On: 19-May-10

1-0	Evaluation **Concurred** Concur - Adopting in part - See attached. Submitted By: Aaron Snyder (651-290-5489) Submitted On: 21-Jun-10 (Attachment: Comment_2_response.docx)
1-1	Backcheck Recommendation **Close Comment** Concur Submitted By: Julian Digialleonardo (561-656-6303) Submitted On: 06-Jul-10
	Current Comment Status: **Comment Closed**

Report Complete

Comment 1:
There are insufficient geotechnical analyses to justify the proposed channel slopes, channel depth, spoil pile configuration, cost estimates, and real estate requirements for the North Dakota Diversion Alternative.
Basis for Comment:
The geotechnical design evaluation in Appendix I does not include analyses to support the proposed channel slopes or spoil pile configuration for the North Dakota Alternative. It appears to the Panel that the evaluation of the North Dakota Alternative was not completed. From the information provided in the Fargo-Moorhead DFR/EIS, it also appears that an assumption was made that the typical channel cross-section for the North Dakota Alternative would be similar to that analyzed for the Minnesota Diversion Alternative. However, this may not be the case. A review of the nine borings available for the North Dakota Alternative indicates subsurface profiles that are different in character from the four stability cross-sections evaluated for the Minnesota Diversion Alternatives. The differences in the subsurface profiles may result in potentially different channel configurations and earthwork quantities. The lack of emphasis on the North Dakota Alternative is also illustrated by the fact that 85 borings were taken for the levee alternatives, 40 borings were taken for the Minnesota Diversion Alternative, but only 9 borings were taken for the North Dakota Alternative. The level of geotechnical analysis and evaluation is not sufficient to support an accurate feasibility cost estimate for the North Dakota Alternative. The lack of geotechnical continuity is further illustrated by the inconsistencies between the Fargo-Moorhead DFR/EIS and the Geotechnical Appendix I. • The configuration of both the Minnesota Diversion and North Dakota Diversion cross-sections are described in the Fargo-Moorhead DFR/EIS (pages 42 and 48); however, these descriptions do not correspond to the final geotechnical cross-sections found in geotechnical analysis for the Minnesota Diversion Alternative contained in Appendix I (page I-17). Geotechnical analysis for the Minnesota Diversion stated that in order to achieve adequate factors of safety, the channel would require a slope of 1V to 10H. This proposed slope is not consistent with the Fargo-Moorhead DFR/EIS description or as shown on the cross-section (Figure 13) or the cross-sections shown in Appendix K. • On page 48, paragraph 3.3.4.1 of the Fargo-Moorhead DFR/EIS states that the Minnesota Diversion Alternative channel was limited to a depth of 30 feet based upon the results of a preliminary analysis of slope stability. The Fargo-Moorhead DFR/EIS also states (first paragraph, page 48) that the maximum depth of 32 feet was used for the North Dakota plan; however, there is no geotechnical analysis or stability evaluation to justify the use of a 32-foot channel depth for the North Dakota Alternative Overall, it appears that the geotechnical analysis for both Diversion Alternatives was not developed completely and/or was not accurately incorporated into the project cost estimates.
Significance – High:
Without a consistent level of geotechnical analysis between alternatives, it is not possible to develop accurate comparative costs for the North Dakota Alternative and the Minnesota Diversion Alternative.
Recommendations for Resolution:

To resolve these concerns the report should be expanded to include:

1. A geotechnical evaluation for the North Dakota Diversion Alternative developed to the same level of detail as that used for the Minnesota Diversion Alternative
2. Revised descriptions to reflect a consistent geotechnical design that provides adequate factors of safety for each alternative (after completing the geotechnical analysis for Diversion Alternatives, Appendix I, the Appendix K cross sections)
3. Recomputed quantity estimates for the channel excavation, and revised cost estimates for all alternatives based on the updated analysis and design
4. A design review of the various hydraulic structures to ensure that the proposed designs are compatible with the final channel configurations based on stability evaluations.

Fargo-Moorhead Metro Feasibility Study
Requirements for the North Dakota Diversion Channel
based on Preliminary Phase 3 Geotechnical analyses

Compiled By: KAH
Date: 5/11/2010

Revised By: KAH
Date: 5/12/2010

Section	Location	Reach Start	Reach End	Distance (feet)	Distance (miles)	Percent	Modifications: Excavation Increase	Modifications: Sand Trench Drain	Modifications: Slope Stabilization	Modifications: Tributary Structures		
1	STA 120+00	0+00	390+00	39,000	7.4	20%	15%					
2	STA 545+00	390+00	660+00	27,000	5.1	14%	15%					
3	STA 940+00	660+00	1000+00	34,000	6.4	18%	16%	$45				
4	STA 1080+00	1000+00	1150+00	15,000	2.8	8%	15%					
5	STA 1225+00	1150+00	1300+00	15,000	2.8	8%	15%					
5B	STA 1445+00	1300+00	1500+00	20,000	3.8	10%	17%		$17			
6	STA 1550+00	1500+00	1770+00	27,000	5.1	14%						
6B	STA 1550+00	1500+00	1670+00	17,000	3.2	9%						
6C	STA 1720+00	1670+00	1770+00	10,000	1.9	5%						
7	STA 1810+00	1770+00	1922+00	15,200	2.9	8%						
				192,200	36.4		112%					
Original Cost	346.9					Cost Increase	42.5	45.0	17.0	12.6	464.0	33.8%
Original Cost w/ Contingency	36%	471.8				Cost Increase w/Contingency	57.9	61.2	23.1	17.1	631.1	33.8%

REQUIREMENTS

Filename	Configuration: Bottom Width	Analyzed Bottom Elev.	**Selected Bench Width**	AnalyzedBench Width	**Bench Height**	**Bench Slope**	Ground Surface	Channel Slope	Surcharge Location	Depth	**Selected X-Sectional Area**	Difference	Analyzed X-Sectional Area	Original X-Sectional Area	Difference	Top of slope, from Centerline	Original Top of Slope	Difference
FM_P3_ND_Div_Sect-01_ALT2	100	857	**50**	40	**10**	**10**	882	7	50	25	**9641**	15%	9341	8354	12%	295	246	20%
FM_P3_ND_Div_Sect-02_ALT2	100	865	**50**	50	**10**	**10**	890	7	50	25	**9641**	15%	9641	8354	15%	305	246	24%
FM_P3_ND_Div_Sect-03_ALT2	100	873	50	80	10	10	900	7	50	27	10889	16%	11909	9366	27%	349	260	34%
FM_P3_ND_Div_Sect-03_ALT2_ModTrench	100	873	**50**	50	**10**	**10**	900	7	50	27	**10889**	16%	10889	9366	3%	319	260	23%
FM_P3_ND_Div_Sect-04_ALT2	100	876	**50**	30	**10**	**10**	900	7	50	24	**9038**	15%	8478	7869	8%	278	239	16%
FM_P3_ND_Div_Sect-05_ALT2	100	879	**50**	50	**10**	**10**	903	7	50	24	**9038**	15%	9038	7869	15%	298	239	25%
FM_P3_ND_Div_Sect-05B_ALT2	100	883	50	40	10	10	913	7	50	30	12866	17%	12466	10989	13%	330	281	17%
FM_P3_ND_Div_Sect-05B_ALT2_mod	100	883	**50**	50	**10**	**10**	913	7	50	30	**12866**	17%	12866	10989	17%	340	281	21%
FM_P3_ND_Div_Sect-06_ALT2	100	885	50	30	10	10	920	7	50	35	16441	18%	15441	13974	10%	355	316	12%
FM_P3_ND_Div_Sect-06B_ALT2	100	894	**50**	50	**5**	**10**	920	7	50	26	**10203**	-27%	10203	13974	-27%	297	316	-6%
FM_P3_ND_Div_Sect-06C_ALT2	100	897	**0**	0	0	0	913	7	50	16	**3458**	-56%	3458	7869	-56%	162	239	-32%
FM_P3_ND_Div_Sect-07_ALT1	125	896	**0**	0	0	0	912	7	0	16	**3858**	0%	3858	3858	0%	174.5	174.5	0%

NOTES:
1) Original cross sectional area based on constant 1V:7H channel side slope (no benching)
2) All cross sections analyzed with a low flow channel: 3 feet deep, 10 feet wide, 1V:4H side slopes, with riprap

Sand Trench

Depth	40	ft	
Width	6	ft	
X-Sect Area (per side)	240	SF	
Sand Cost	$50	per ton	(cost based on riprap cost)
	1.5	ton/CY	
Sand Cost	$75	per CY	

Slope Stabilization

X-Sect Area (per side)	150	SF	
Riprap Cost	$50.00	per ton	(cost based on bid prices for Fargo-Ridgewood project)
	1.5	ton/CY	
	$75	per CY	

Fargo-Moorhead Metro Feasibility Study

Summary of the Phase 3 Preliminary Geotechnical Analyses of ND Diversion Alternatives

Compiled By: KAH Date: 5/6/2010

Revised By: KAH Date: 5/12/2010

Section	Location	Reach Start	Reach End	Distance (feet)	Distance (miles)	Percent
1	STA 120+00	0+00	390+00	39,000	7.4	20%
2	STA 545+00	390+00	660+00	27,000	5.1	14%
3	STA 940+00	660+00	1000+00	34,000	6.4	18%
4	STA 1080+00	1000+00	1150+00	15,000	2.8	8%
5	STA 1225+00	1150+00	1300+00	15,000	2.8	8%
5B	STA 1445+00	1300+00	1500+00	20,000	3.8	10%
6	STA 1550+00	1500+00	1770+00	27,000	5.1	14%
6B	STA 1550+00	1500+00	1670+00	17,000	3.2	9%
6C	STA 1720+00	1670+00	1770+00	10,000	1.9	5%
7	STA 1810+00	1770+00	1922+00	15,200	2.9	8%
				192,200	36.4	

	Location
Outfall	STA 0+00
I-29	STA 300+00
Rush River	STA 480+00
Lower Rush	STA 600+00
Maple River	STA 730+00
CR 10	STA 900+00
I-94	STA 920+00
Sheyenne Ri	STA 1500+00
Wild Rice	STA 1780+00
I-29	STA 1820+00
Red River	SRA 1920+00

Alternative 1: Current invert elevation, 10' high bench, 1V:10H slope on bench, 1V:7H channel slope

Filename	Configuration: Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	Top of slope, from Centerline	Original Top of Slope	Difference	X-Sectional Area	Original X-Sectional Area	% Increase	Global FS	Local FS	Water Exits Slope	Height Below Ground Surface	Height above Bottom
FM_P3_ND_Div_Sect-01_ALT1	100	854	50	10	10	882	7	50	28	326	246	33%	11534	8354	38%	1.433	1.587	864.5	17.5	10.5
FM_P3_ND_Div_Sect-02_ALT1	100	862	80	10	10	890	7	50	28	356	246	45%	12614	8354	51%	1.406	1.408	876	14	14
FM_P3_ND_Div_Sect-03_ALT1	100	870	125	10	10	900	7	50	30	415	260	60%	15866	9366	69%	1.417	1.106	888	12	18
FM_P3_ND_Div_Sect-04_ALT1	100	873	60	10	10	900	7	50	27	329	239	38%	11229	7869	43%	1.416	1.266	884	16	11
FM_P3_ND_Div_Sect-05_ALT1	100	876	80	10	10	903	7	50	27	349	239	46%	11909	7869	51%	1.405	1.402	889	14	13
FM_P3_ND_Div_Sect-06_ALT1	100	882	30	10	10	920	7	50	38	376	316	19%	17634	13974	26%	1.425	0.614	896	24	14
FM_P3_ND_Div_Sect-07_ALT1	125	896	0	0	0	912	7	0	16	174.5	174.5	0%	3858	3858	0%	1.598	1.616	899	13	3

Filename	Stability Analysis: Min FS: Global		Wedge		Localized		Localized 2		Undrained		Localized	
Minimum FS	1.4		1.4		1.2		1.2		1.3		1.3	
FM_P3_ND_Div_Sect-01_ALT1	**1.449**	0	**1.433**		**1.588**	0	**1.587**	0	**1.431**	0	**4.762**	0
FM_P3_ND_Div_Sect-02_ALT1	**1.464**	0	**1.406**		**1.408**	0	**1.433**	0	**1.418**	0	**4.226**	0
FM_P3_ND_Div_Sect-03_ALT1	**1.446**	0	**1.417**		**1.109**	0	**1.106**	0	**1.979**	0	**4.515**	0
FM_P3_ND_Div_Sect-04_ALT1	**1.419**	0	**1.416**		**1.266**	0	**1.267**	0	**1.855**	0	**4.48**	0
FM_P3_ND_Div_Sect-05_ALT1	**1.44**	0	**1.405**		**1.402**	0	**1.427**	0	**1.648**	0	**4.905**	0
FM_P3_ND_Div_Sect-06_ALT1	**1.425**	0	**1.432**		**0.672**	0	**0.614**	0	**1.678**	0	**6.33**	0
FM_P3_ND_Div_Sect-07_ALT1	**1.598**	0	**1.604**		**1.881**	0	**1.616**	0	**2.719**	0	**2.713**	0

Alternative 2: Invert elevation raised 3 feet, 10' high bench, 1V:10H slope on bench, 1V:7H channel slope

Filename	Configuration: Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	Top of slope, from Centerline	Original Top of Slope	Difference	X-Sectional Area	Original X-Sectional Area	% Increase	Global FS	Local FS	Water Exits Slope	Height Below Ground Surface	Height above Bottom
FM_P3_ND_Div_Sect-01_ALT2	100	857	40	10	10	882	7	50	25	295	246	20%	9341	8354	12%	1.451	1.659	867	15	10
FM_P3_ND_Div_Sect-02_ALT2	100	865	50	10	10	890	7	50	25	305	246	24%	9641	8354	15%	1.412	1.53	876	14	11
FM_P3_ND_Div_Sect-03_ALT2	100	873	80	10	10	900	7	50	27	349	260	34%	11909	9366	27%	1.413	1.315	888	12	15
FM_P3_ND_Div_Sect-03_ALT2_ModTrench	100	873	50	10	10	900	7	50	27	319	260	23%	10889	9366	16%	1.416	1.446	0	900	-873
FM_P3_ND_Div_Sect-04_ALT2	100	876	30	10	10	900	7	50	24	278	239	16%	8478	7869	8%	1.436	1.495	886	14	10
FM_P3_ND_Div_Sect-05_ALT2	100	879	50	10	10	903	7	50	24	298	239	25%	9038	7869	15%	1.415	1.534	890	13	11
FM_P3_ND_Div_Sect-05B_ALT2	100	883	40	10	10	913	7	50	30	330	281	17%	12466	10989	13%	1.399	0.64	895	18	12
FM_P3_ND_Div_Sect-05B_ALT2_mod	100	883	50	10	10	913	7	50	30	340	281	21%	12866	10989	17%	1.551	1.302	895	18	12
FM_P3_ND_Div_Sect-06_ALT2	100	885	30	10	10	920	7	50	35	355	316	12%	15441	13974	10%	1.421	0.437	897	23	12
FM_P3_ND_Div_Sect-06B_ALT2	100	894	50	5	10	920	7	50	26	297	316	-6%	10203	13974	-27%	1.405	1.425	904	16	10
FM_P3_ND_Div_Sect-06C_ALT2	100	897	0	0	0	913	7	50	16	162	239	-32%	3458	7869	-56%	1.619	1.614	901	12	4

Filename	Stability Analysis: Min FS: Global		Wedge		Localized		Localized 2		Undrained		Localized	
FM_P3_ND_Div_Sect-01_ALT2	**1.452**	0	**1.451**		**1.659**	0	**1.664**	0	**1.45**	0	**4.693**	0
FM_P3_ND_Div_Sect-02_ALT2	**1.412**	0	**1.413**		**1.531**	0	**1.53**	0	**1.402**	0	**4.208**	0
FM_P3_ND_Div_Sect-03_ALT2	**1.419**	0	**1.413**		**1.317**	0	**1.315**	*	**1.694**	*	**4.364**	0
FM_P3_ND_Div_Sect-03_ALT2_ModTrench	**1.446**	0	**1.416**		**1.447**	0	**1.446**	0	**1.693**	0	**4.364**	0
FM_P3_ND_Div_Sect-04_ALT2	**1.45**	0	**1.436**		**1.495**	0	**1.496**	0	**1.892**	0	**4.237**	0
FM_P3_ND_Div_Sect-05_ALT2	**1.451**	0	**1.415**		**1.534**	0	**1.534**	0	**1.62**	0	**5.139**	0
FM_P3_ND_Div_Sect-05B_ALT2	**1.523**		**1.399**		**0.641**		**0.64**		**1.867**		**5.907**	
FM_P3_ND_Div_Sect-05B_ALT2_mod	**1.629**	0	**1.551**		**1.302**	0	**1.304**	0	**1.764**	0	**6.12**	0
FM_P3_ND_Div_Sect-06_ALT2	**1.432**		**1.421**		**0.489**		**0.437**		**1.594**		**5.913**	
FM_P3_ND_Div_Sect-06B_ALT2	**1.425**	0	**1.405**		**1.425**	0	**1.43**	0	**1.687**	0	**3.628**	0
FM_P3_ND_Div_Sect-06C_ALT2	**1.619**	0	**1.626**		**1.791**	0	**1.614**	0	**1.855**	0	**4.24**	0

Alternative 3: Invert elevation raised 5 feet, 10' high bench, 1V:10H slope on bench, 1V:7H channel slope

Filename	Configuration: Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	Top of slope, from Centerline	Original Top of Slope	Difference	X-Sectional Area	Original X-Sectional Area	% Increase	Global FS	Local FS	Water Exits Slope	Height Below Ground Surface	Height above Bottom
FM_P3_ND_Div_Sect-01_ALT3	100	859	25	10	10	882	7	50	23	266	246	8%	7799	8354	-7%	1.453	1.735	866.5	15.5	7.5
FM_P3_ND_Div_Sect-02_ALT3	100	867	30	10	10	890	7	50	23	271	246	10%	7929	8354	-5%	1.401	1.585	877	13	10
FM_P3_ND_Div_Sect-03_ALT3	100	875	50	10	10	900	7	50	25	305	260	17%	9641	9366	3%	1.416	1.437	888	12	13
FM_P3_ND_Div_Sect-04_ALT3	100	878	10	10	10	900	7	50	22	244	239	2%	6914	7869	-12%	1.424	1.6	887	13	9
FM_P3_ND_Div_Sect-05_ALT3	100	881	20	10	10	903	7	50	22	254	239	6%	7154	7869	-9%	1.412	1.587	891	12	10
FM_P3_ND_Div_Sect-06_ALT3	100	887	50	10	10	920	7	50	33	361	316	14%	14969	13974	7%	1.501	0.518	900	20	13

Filename	Stability Analysis: Min FS: Global		Wedge		Localized		Localized 2		Undrained		Localized	
FM_P3_ND_Div_Sect-01_ALT3	**1.453**	0	**1.453**		**1.735**	0	**1.739**	0	**1.442**	0	**4.711**	0
FM_P3_ND_Div_Sect-02_ALT3	**1.401**	0	**1.42**		**1.586**	0	**1.585**	0	**1.386**	0	**4.15**	0
FM_P3_ND_Div_Sect-03_ALT3	**1.42**	0	**1.416**		**1.437**	0	**1.438**	0	**1.826**	0	**4.28**	0
FM_P3_ND_Div_Sect-04_ALT3	**1.424**	0	**1.424**		**1.601**	0	**1.6**	0	**1.904**	0	**4.145**	0
FM_P3_ND_Div_Sect-05_ALT3	**1.412**	0	**1.412**		**1.587**	0	**1.593**	0	**1.587**	0	**1.589**	0
FM_P3_ND_Div_Sect-06_ALT3	**1.503**	0	**1.501**		**0.537**	0	**0.518**	0	**1.729**	0	**6.195**	0

Other Alternatives:

Filename	Configuration: Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	Top of slope, from Centerline	Original Top of Slope	Difference	X-Sectional Area	Original X-Sectional Area	% Increase	Global FS	Local FS	Water Exits Slope	Height Below Ground Surface	Height above Bottom
FM_P3_ND_Div_Sect-06B_ALT2	100	894	50	5	10	920	7	50	26	297	316	-6%	10203	13974	-27%	1.405	1.425	904	16	10
FM_P3_ND_Div_Sect-06C_ALT2	100	897	0	0	0	913	7	50	16	162	239	-32%	3458	7869	-56%	1.619	1.614	901	12	4

Filename	Stability Analysis: Min FS: Global		Wedge		Localized		Localized 2		Undrained		Localized	
FM_P3_ND_Div_Sect-06B_ALT2	**1.425**	0	**1.405**		**1.425**	0	**1.43**	0	**1.687**	0	**3.628**	0
FM_P3_ND_Div_Sect-06C_ALT2	**1.619**	0	**1.626**		**1.791**	0	**1.614**	0	**1.855**	0	**4.24**	0

* Original X-sectional area based on constant 1V:7H channel side slope (no benching)

** All alternatives have a low flow channel: 3 feet deep, 10 feet wide, 1V:4H side slopes, with riprap

Fargo-Moorhead Metro Feasibility Study
Summary of the Phase 3 Preliminary Geotechnical Analyses of MN Diversion Alternatives

Compiled By: KAH Date: 5/17/2010
Revised By: KAH Date: 5/19/2010

Section	Location	Reach Start	Reach End	Distance (feet)	Distance (miles)	Percent	Requires Modification	
1	STA 20+00	0+00	70+00	7,000	1.3	5%	NO	0
2	STA 175+00	70+00	220+00	15,000	2.8	11%	NO	0
2B	STA 355+00	220+00	440+00	22,000	4.2	16%	NO	0
3	STA 515+00	440+00	540+00	10,000	1.9	7%	YES	10,000
4	STA 635+00	540+00	665+00	12,500	2.4	9%	Probably	12,500
4B	STA 700+00	665+00	750+00	8,500	1.6	6%	Probably	8,500
5	STA 805+00	750+00	900+00	15,000	2.8	11%	Probably	15,000
6	STA 980+00	900+00	1110+00	21,000	4.0	15%	YES	21,000
7	STA 1160+00	1110+00	1190+00	8,000	1.5	6%	YES	8,000
7B	STA 1235+00	1190+00	1280+00	9,000	1.7	7%	YES	9,000
8	STA 1325+00	1280+00	1363+00	8,300	1.6	6%	YES	8,300
				136,300	22.6			92,300

Phase 2 Analysis Modifications

Section	Modification 2 Start	Modification 2 End	Modification 2 Length	Modification 3 Start	Modification 3 End	Modification 3 Length	
1							
2							
2B	35500	44000	8,500			0	
3	44000	54000	10,000			0	
4	54000	65000	11,000	65000	66500	1,500	
4B			0	66500	75000	8,500	
5	81000	90000	9,000	75000	81000	6,000	
6	90000	111000	21,000			0	
7	111000	119000	8,000			0	
7B	119000	125000	6,000			0	
8			0			0	
			73,500			16,000	89,500

Requirements:

Filename	Configuration: Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	X-Sectional Area	Original X-Sectional Area	Differece	Top of slope	Original Top of Slope	Difference	Global FS	Local FS
FM_P3_MN_Div_01_20k	175	875	0	0	0	895	7	50	20	6366	6366	0%	227.5	227.5	0%	1.517	1.887
FM_P3_MN_Div_02_20k	175	876	0	0	0	894	7	50	18	5484	5484	0%	213.5	213.5	0%	1.859	2.846
FM_P3_MN_Div_02B_20k	175	878	0	0	0	896	7	50	18	5484	5484	0%	213.5	213.5	0%	1.616	1.928
FM_P3_MN_Div_03_35k_mod	260	880	25	7	10	905	7	50	25	12744	13441	-5%	351	355	-1%	1.488	2.141
FM_P3_MN_Div_06_35k_mod	260	886	70	7	10	914	7	50	28	16803	15634	7%	417	376	11%	1.419	1.585
FM_P3_MN_Div_07_35k_mod	260	890	30	7	10	914	7	0	24	12219	12738	-4%	349	348	0%	1.409	1.786
FM_P3_MN_Div_07B_35k_mod	260	891	10	7	10	910	7	50	19	8424	9433	-11%	294	313	-6%	1.442	1.512
FM_P3_MN_Div_08_35k_mod	260	893	40	7	10	912	7	50	19	9144	9433	-3%	324	313	4%	1.405	1.46

Fargo-Moorhead Metro Feasibility Study
Phase 3 Preliminary Geotechnical Analyses of MN Diversion Alternatives

St. Paul District GEOLOGY GEOTECHNICAL US Army Corps of Engineers

Compiled By: KAH Date: 5/3/2010 Revised By: KAH Date: 5/12/2010

Minnesota Diversion Channel, Section 1 STA 20+00 0+00 70+00 7,000 1.3

	Configuration										Stability Analysis: Min FS																	
Filename	Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	Other Changes	Global		Wedge		Localized		Localized 2		Undrained		Undrained Localized		X-Sectional Area	% Increase	Top of slope	Water Exits Slope	Height Below Ground Surface	Height above Bottom
Original MN 01_20k	175	875	0	0	0	895	7		20														6366		227.5			
FM_P3_MN_Div_01_20k	**175**	**875**	**0**	**0**	**0**	**895**	**7**	**50**	**20**		**1.517**	1	**1.518**		**2.068**	2	**1.887**	2	**1.331**	*	**4.288**	5	**6366**	**0%**	**227.5**	**876.5**	**18.5**	**1.5**
FM_P3_MN_Div_01_b_20k										no riprap LF	1.509		5.1		2.065		1.872		1.338		4.288					876.5		

Minnesota Diversion Channel, Section 2 STA 175+00 70+00 220+00 15,000 2.8

	Configuration										Stability Analysis: Min FS																	
Filename	Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	Other Changes	Global		Wedge		Localized		Localized 2		Undrained		Localized		X-Sectional Area	% Increase	Top of slope	Water Exits Slope	Height Below Ground Surface	Height above Bottom
Original MN 02_20k	175	876	0	0	0	894	7		18													6	5484		213.5			
FM_P3_MN_Div_02_20k	**175**	**876**	**0**	**0**	**0**	**894**	**7**	**50**	**18**		**1.859**	*	**1.875**		**2.85**	3	**2.846**	3	**1.42**	*	**6.692**		**5484**	**0%**	**213.5**	**876**	**18**	**0**
FM_P3_MN_Div_02_b_20k											1.833	*	1.85		2.682		2.657		1.433	*	6.692		66	-99%	0		0	0
																							66	-99%	0		0	0
																							66	-99%	0		0	0

Minnesota Diversion Channel, Section 2B STA 355+00 220+00 440+00 22,000 4.2

	Configuration										Stability Analysis: Min FS																	
Filename	Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	Other Changes	Global		Wedge		Localized		Localized 2		Undrained		Localized		X-Sectional Area	% Increase	Top of slope	Water Exits Slope	Height Below Ground Surface	Height above Bottom
Original MN 02B_20k	175	878	0	0	0	896	7		18													6	5484		213.5			
FM_P3_MN_Div_02B_20k	**175**	**878**	**0**	**0**	**0**	**896**	**7**	**50**	**18**		**1.622**		**1.616**		**1.946**		**1.928**		**1.425**				**5484**	**0%**	**213.5**	**878**	**18**	**0**
																							66	-99%	0		0	0
																							66	-99%	0		0	0
																							66	-99%	0		0	0

Minnesota Diversion Channel, Section 3 STA 515+00 440+00 540+00 10,000 1.9

	Configuration										Stability Analysis: Min FS																	
Filename	Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	Other Changes	Global		Wedge		Localized		Localized 2		Undrained		Localized		X-Sectional Area	% Increase	Top of slope	Water Exits Slope	Height Below Ground Surface	Height above Bottom
Original MN 03_20k	175	880	0	0	0	905	7		25														8816		262.5			
FM_P3_MN_Div_03_20k	175	880	0	0	0	905	7	50	25		1.347		1.346		1.503		1.699		1.222				8816	0%	262.5	881	24	1
Original MN 03_35k	360	880	0	0	0	905	7	50	25														13441		355			
FM_P3_MN_Div_03_35k	360	880	0	0	0	905	7	50	25		1.346		1.363		1.709		1.703		1.247				13441	0%	355	881	24	1
FM_P3_MN_Div_03_35k_mod	**260**	**880**	**25**	**7**	**10**	**905**	**7**	**50**	**25**		**1.488**		**1.499**		**2.141**		**2.293**		**1.297**				**12744**	**-5%**	**351**	**882**	**23**	**2**
																							66	-100%	0		0	0

Minnesota Diversion Channel, Section 4 STA 635+00 540+00 665+00 12,500 2.4

	Configuration										Stability Analysis: Min FS																	
Filename	Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	Other Changes	Global		Wedge		Localized		Localized 2		Undrained		Localized		X-Sectional Area	% Increase	Top of slope	Water Exits Slope	Height Below Ground Surface	Height above Bottom
Original MN 04_20k	175	881	0	0	0	912	7		31														12218		304.5			
FM_P3_MN_Div_04_20k	175	881	0	0	0	912	7	50	31		1.098		1.099		1.1175		1.184		1.175				12218	0%	304.5	890	22	9
Original MN 04_35k	360	881	0	0	0	912	7	50	31														17953		397			
FM_P3_MN_Div_04_35k	360	881	0	0	0	912	7	50	31		1.119								1.179				17953	0%	397	890	22	9
FM_P3_MN_Div_04_35k_mod	260	881	75	7	10	912	7	50	31		1.36								1.256				19608	9%	443	890	22	9
																							66	-100%	0		0	0

Fargo-Moorhead Metro Feasibility Study
Phase 3 Preliminary Geotechnical Analyses of MN Diversion Alternatives

St. Paul District GEOLOGY GEOTECHNICAL US Army Corps of Engineers

Compiled By: KAH Date: 5/3/2010
Revised By: KAH Date: 5/12/2010

Minnesota Diversion Channel, Section 4B STA 700+00 665+00 750+00 8,500 1.6

	Configuration										Stability Analysis: Min FS																	
Filename	Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	Other Changes	Global		Wedge		Localized		Localized 2		Undrained		Localized		X-Sectional Area	% Increase	Top of slope	Water Exits Slope	Height Below Ground Surface	Height above Bottom
Original MN 04B_20k									0														66		0			
FM_P3_MN_Div_04B_20k									0														66	0%	0		0	0
																							66	0%	0		0	0
																							66	0%	0		0	0
																							66	0%	0		0	0

Minnesota Diversion Channel, Section 5 STA 805+00 750+00 900+00 15,000 2.8

	Configuration										Stability Analysis: Min FS																	
Filename	Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	Other Changes	Global		Wedge		Localized		Localized 2		Undrained		Localized		X-Sectional Area	% Increase	Top of slope	Water Exits Slope	Height Below Ground Surface	Height above Bottom
Original MN 05_20k									0														66		0			
FM_P3_MN_Div_05_20k									0														66	0%	0		0	0
																							66	0%	0		0	0
																							66	0%	0		0	0
																							66	0%	0		0	0

Minnesota Diversion Channel, Section 6 STA 980+00 900+00 1110+00 21,000 4.0

	Configuration										Stability Analysis: Min FS																	
Filename	Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	Other Changes	Global		Wedge		Localized		Localized 2		Undrained		Localized		X-Sectional Area	% Increase	Top of slope	Water Exits Slope	Height Below Ground Surface	Height above Bottom
Original MN 06_20k																							66		0			
FM_P3_MN_Div_06_20k																							66	0%	0		0	0
Original MN 06_35k	360	886	0	0	0	914	7	0	28														15634		376			
FM_P3_MN_Div_06_35k	360	886	0	0	0	914	7	50	28		1.107		1.215		1.129		1.121		1.329				15634	0%	376	900	14	14
FM_P3_MN_Div_06_35k_mod	**260**	**886**	**70**	**7**	**10**	**914**	**7**	**50**	**28**		**1.441**		**1.419**		**1.586**		**1.585**		**1.463**				**16803**	**7%**	**417**	**900**	**14**	**14**
																							66	-100%	0		0	0
																							66	-100%	0		0	0

Minnesota Diversion Channel, Section 7 STA 1160+00 1110+00 1190+00 8,000 1.5

	Configuration										Stability Analysis: Min FS																	
Filename	Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	Other Changes	Global		Wedge		Localized		Localized 2		Undrained		Localized		X-Sectional Area	% Increase	Top of slope	Water Exits Slope	Height Below Ground Surface	Height above Bottom
Original MN 07_20k	175	890	0	0	0	914	7	0	24														8298		255.5			
FM_P3_MN_Div_07_20k	175	890	0	0	0	914	7	0	24		1.173	1	1.33		1.232	1	1.214	1	1.357	1		4	8298	0%	255.5	899	15	9
Original MN 07_35k	360	890	0	0	0	914	7	0	24														12738		348			
FM_P3_MN_Div_07_35k	360	890	0	0	0	914	7	0	24		1.212		1.277		1.297		1.267		1.358				12738	0%	348	898	16	8
FM_P3_MN_Div_07_35k_mod	**260**	**890**	**30**	**7**	**10**	**914**	**7**	**0**	**24**		**1.411**		**1.409**		**1.787**		**1.786**		**1.427**				**12219**	**-4%**	**349**	**898**	**16**	**8**
	300	890	20	7	10	914	7		24														12839	1%	359		914	-890
																							66	-99%	0		0	0

Minnesota Diversion Channel, Section 7B STA 1235+00 1190+00 1280+00 9,000 1.7

	Configuration										Stability Analysis: Min FS																	
Filename	Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	Other Changes	Global		Wedge		Localized		Localized 2		Undrained		Localized		X-Sectional Area	% Increase	Top of slope	Water Exits Slope	Height Below Ground Surface	Height above Bottom
Original MN 07B_20k	175	891	0	0	0	910	7	0	19														5918		220.5			
FM_P3_MN_Div_07B_20k	175	891	0	0	0	910	7	50	19		1.25	1	1.251		1.372	1	1.286	1	1.567	1		4	5918	0%	220.5	900	10	9
Original MN 07B_35k	360	891	0	0	0	910	7	50	19														9433		313		910	-891
FM_P3_MN_Div_07B_35k	360	891	0	0	0	910	7	50	19		1.299		1.337		1.449		1.339		1.562				9433	0%	313	900	10	9
FM_P3_MN_Div_07B_35k_mod	**260**	**891**	**10**	**7**	**10**	**910**	**7**	**50**	**19**		**1.442**		**1.452**		**1.827**		**1.512**		**1.621**				**8424**	**-11%**	**294**		**910**	**-891**
																							66	-99%	0		0	0

Fargo-Moorhead Metro Feasibility Study
Phase 3 Preliminary Geotechnical Analyses of MN Diversion Alternatives

US Army Corps of Engineers® St. Paul District Geology Geotechnical

Compiled By: KAH Revised By: KAH
Date: 5/3/2010 Date: 5/12/2010

Minnesota Diversion Channel, Section 8 STA 1325+00 1280+00 1363+00 8,300 1.6

	Configuration										Stability Analysis: Min FS											
Filename	Bottom Width	Bottom Elev.	Bench Width	Bench Height	Bench Slope	Ground Surface	Channel Slope	Surcharge Location	Depth	Other Changes	Global	Wedge	Localized	Localized 2	Undrained	Localized	X-Sectional Area	% Increase	Top of slope	Water Exits Slope	Height Below Ground Surface	Height above Bottom
Original MN 08_20k	175	893	0	0	0	912	7	0	19								5918		220.5			
FM_P3_MN_Div_08_20k	175	893	0	0	0	912	7	50	19		1.057	1.058	1.091	1.069	1.763	4	5918	0%	220.5	902	10	9
Original MN 08_35k	360	893	0	0	0	912	7	50	19								9433		313			
FM_P3_MN_Div_08_35k	360	893	0	0	0	912	7	50	19		1.072	1.162	1.11	1.083	1.763		9433	0%	313	902	10	9
FM_P3_MN_Div_08_35k_mod	**260**	**893**	**40**	**7**	**10**	**912**	**7**	**50**	**19**		**1.407**	**1.405**	**1.46**	**1.46**	**1.956**		**9144**	**-3%**	**324**	**902**	**10**	**9**
									0								66	-99%	0		0	0
									0								66	-99%	0		0	0

Comment 2:
The stability of the channel slopes, foundation deposits, and related spoil piles should be evaluated using ultimate or near ultimate soil strength values for the End of Construction (EOC) condition.
Basis for Comment:
The stability analyses shown in Appendix I (page I-12) indicate that long-term stability will be the controlling load condition in determining the slope configuration of the proposed diversion channel and spoil piles. This is questionable based on the history of many projects in the Red River Valley that have experienced failure or near failure during construction. The failure or near failure during the "End of Construction (EOC) condition" has occurred on many projects within the Red River Valley including the VA Hospital levee failure in Fargo (1948), the Pembina levee project (1978), the Grand Forks levee project (1953), the Fargo Grain Elevator collapse (1955), the Hartsville Pumping Station levee, Grand Forks (2005), and the I-94 Interstate Highway interchange in Fargo (2008). These failures demonstrated that the EOC is a potentially critical failure mode for any excavation or fill slope in the Red River Valley. Furthermore, the use of peak values of Unconsolidated–Undrained (UU) soil strengths to evaluate the EOC conditions appears to be un-conservative. In Appendix I, the use of peak UU soil strengths in the stability analyses was justified by the following statement (paragraph 46): "During the process of draining, it can be expected that the soils will experience strains less than 5% to 8% which is a strain at which undrained shear strength occurs." Appendix I of the Fargo-Moorhead DFR/EIS does not clarify the basis of this statement and does not contain analysis or justification to identify the level of strains that may occur during the EOC condition. Experience and laboratory testing indicate that the Brenna formation, which underlies much of the project area and the Red River Valley, is the weakest and most unreliable lacustrine unit. This is composed of highly active clay minerals with high void ratios, water contents, and liquid limits. Laboratory testing of samples obtained throughout the Red River Valley indicates a brittle stress strain curve that achieves high peak strengths at low values of strain and then drops to much lower values of strength at higher strains. The statement described above regarding the 5 to 8% strains at which the peak undrained strength occurs may be untrue in many cases. The tabulation for the Brenna formation (UU) laboratory testing for the recent Hartsville Pumping Station levee failure in Grand Forks indicated that 35 out of 50 shear strength samples failed in the laboratory at peak strengths less than 5%. If the sliding mass reaches strains greater than 5%, it is likely that the mobilized strength will be significantly less than the peak strength values. It should also be noted that the back calculation of strengths for the Hartsville failure indicated values close to the ultimate strengths as determined by UU testing.
Significance – High:
Using un-conservative strength assumptions could affect the channel design, real estate requirements, and estimated project costs.
Recommendations for Resolution:
To resolve these concerns, the report should be expanded to use data from the recent failure of the I-94 Interchange to back calculate the actual UU strengths mobilized at failure. This

information is available from local engineering firms and would provide a realistic basis to assess the methods and test results to evaluate the stability of the proposed channel for the EOC condition.

Concur - Adopting in part:

The USACE recognizes the differences in soil behavior for different material types/formations. The laboratory test data was reviewed and ultimate undrained shear strength parameters were selected based on shear strengths at 15% strain. It was found that there was approximately a 10%, 19%, and 27% reduction in undrained shear strength for the Oxidized Brenna, Brenna, and Argusville formations, respectively. The end-of-construction stability of the diversion channels was checked using the ultimate undrained shear strength parameters and a spoil pile setback distance of 50 feet. It was found that for most cases the minimum factors of safety were still met using the ultimate undrained shear strength parameters. In a few cases, the factors of safety fell below the minimum required. In these instances, the spoil pile height adjacent to the top of the diversion was reduced in order to meet the minimum required factors of safety. This reduced spoil pile height extended out until it could be increase back to 15 feet while maintaining stability.

Not Adopting: The USACE reviewed the I-94 Interchange embankment report completed by Braun Intertec for SRF Consulting Group. The analyses completed for the I-94 Interchange embankment were forensic type analyses completed to evaluate what caused the failure of the embankment during construction. The analyses used lab and field data that was collected during construction and post-failure to calibrate the models to match observed conditions. The reconstruction of the embankment was based on these calibrated models. The I-94 Interchange was a loading only project, whereas the diversion channels will involve loading and unloading along potential slip surfaces due to spoil pile placement as well as channel excavation.

The USACE recreated the undrained model that Braun reported in Figure 1. The USACE results indicated a factor of safety slightly below 1.0 for a wedge type failure search where as Braun reported a factor of safety of 1.168. The USACE also ran the model using the ultimate undrained shear strength parameters selected for the Fargo-Moorhead project along with a 25-foot, water filled crack (the crack depth was selected to eliminate tension in the fill). The results indicated a factor of safety of 1.015. Based on these analyses, the USACE feels that the method being used to model the end-of-construction case for the Fargo-Moorhead diversions is appropriate. The USACE does not feel that including a reference to the I-94 Interchange report would provide any benefit to the review and may even cause confusion.

In addition, the USACE recognizes that a coupled Sigma/W and Slope/W analysis would be another means of evaluating the stability of the excavated channel, like the I-94 failure. The Sigma/W analysis would estimate pore pressures during excavation of the channel and placement of the spoil material and then an effective stress limit equilibrium slope stability analysis could be completed using Slope/W and effective stress parameters. Due to the lack of actual field data the Sigma/W and Slope/W models cannot be calibrated as in the instance of the I-94 Interchange embankment. A number of assumptions would be required for these analyses and could lead to more uncertainty in the results. It is felt that using ultimate undrained shear strength parameters to evaluate the EOC condition is adequate for the feasibility stage of the project. Thought will be given in using a coupled Sigma/W and Slope/W analysis for future refinement of the project. See the embedded PDF attachment below.

Attachment #2.

Formation	Total Stress Shear Strength Parameters		
	Peak Values c (psf)	Ultimate c (psf)	Reduction %
Sherack	1400	900	36%
Poplar River - West Fargo	1900	1900	0%
Poplar River - Harwood	1450	1200	17%
Poplar River, All (5)	1700	1700	0%
Oxidized Brenna (6)	1000	900	10%
Brenna (6)	650	525	19%
Argusville (6)	825	600	27%
Till (7)	1900	1900	0%

St. Paul District Internal Peer Review

August 2010
&
February 2011

CEMVP-EC-D

MEMORANDUM FOR THE RECORD

SUBJECT: Responses to the Peer Review of the Fargo – Moorhead Metro Feasibility Study, Geotechnical Design and Geology Appendix

A peer review of the Geotechnical Design and Geology Appendix for the Fargo-Moorhead Metro Feasibility Study was completed by Chris Behling, St. Paul District EC-D, on August 26, 2010. Mr. Behling provided some minor grammatical edits to the report using the "track changes" feature. In addition to the grammatical edits, Mr. Behling's review prompted 28 comments or questions concerning the report. The undersigned has reviewed and evaluated Mr. Behling's comments/questions and has responded to them. A summary of the evaluation and changes are indicated following the comments.

1. How were the extent of the failed riverbank (areas of residual shear strengths) determined for the slope stability analyses completed for the in-town levees alternative? Was the line simply drawn at the secondary riverbank? Some description of how this was determined would be helpful.

   The extent of the failed riverbank was drawn to coincide with the secondary riverbank as this would provide the most conservative assumption.

2. Were any instrumented cross-sections available to compare to analysis results? Cross-sections where inclinometer data indicated location of the shear zone, perhaps the location of the head or toe of the landslide was known, and water levels from piezometers were available. This would provide some measure of confidence in the limit equilibrium slope stability results for the in-town levees alternative. Since the in-town levee plan is not the design alternative being pursued, this comment is not considered critical.

   Two slope inclinometer installed during the soil exploration program in Spring 2009. Only a few readings have been taken since the installation. The indicated slip plane is near the bottom of the Brenna formation, which coincides with other slides in the Red River Valley.

   The instrumentation data was not used to complete a back calculation stability analysis. The main reason for not completing the back calculation analysis was the level of detail for this analysis was beyond what was needed in the preliminary stages of defining the needs of the in-town levee alternatives. It was decided that some stability analyses be completed based on conservative assumptions to determine preliminary setback distances. As the project evolved, the assumptions would be reevaluated and more detailed analyses completed. It was found during Phase 2 that the in-town levee alternative could only provide a reasonable flood risk reduction level to around a 100-year event due to the required height of the levees and lack of “high ground” in which to tie the levees into. Due to this, there was no need to evaluate the in-town levee alternative further and complete more detailed analyses.

3. Paragraph I.2.1.3 states, “The slides may extend for several hundred feet along the river bank”. The reviewer’s experience is that landslides in the F/M area can extend for longer distances along the riverbank, perhaps even a few thousand feet.

   Agree that there could possibly be some areas in which a slide could occur over a longer distance.

4. In Paragraph I.2.1.5, the reviewer wasn’t sure what “this relatively steady slope to the north falters very little” meant.

   There is no abrupt changes in elevation.

5. In Paragraph I.2.2.1, the last sentence indicates the riverbank slope is too steep to obtain a mid-slope boring. This seems to contradict para. I.2.1.2 under Topography where the ground slope is described as gentle, flat, or somewhat hummocky.

   The paragraph was rewritten and last sentence was removed.

6. In Paragraph I.2.2.2, under the bulleted NDGS references, should Survey Report of Investigation No. 60 have its own bullet?

   Yes, the NDGS No. 60 report should have its own bullet. This has been changed.

7. Since paragraph I.2.2.3 is one sentence, should it be shown as the last sentence in the previous paragraph?

   To direct the reader to the stratigraphy discussion, this sentence is needed. It does not fit well in the previous paragraph, so it needs to be by itself.

8. Page 11 in Attachment I-1 seems to be repeated and inadvertently included.

   Yes, it was inadvertently included. Page 11 has been removed.

9. In Paragraph I.2.5.2, the second to the last sentence in the paragraph, refers to no known reports of disturbances from either of these events. The reviewer was unsure which events were being referred to.

   The paragraph has been rearranged to clarify what events the statement is referring to. It now states the following:

The nearest continental basement fault to the west is the Thompson Boundary fault, which extends from the approximate Saskatchewan - Manitoba boundary southward through North Dakota, about 200 miles west of the Red River Valley. The fault separates the stable Wyoming and Superior Cratons of the tectonically-inactive Canadian Shield. An earthquake occurred along this fault near Huff, North Dakota, south of Bismarck, in 1968. It had a magnitude of 4.4 on the Richter Scale (IV-V Mercalli Intensity). This has been the largest and also the nearest (less than 200 miles west) recorded earthquake in North Dakota (North Dakota Geological Survey, Geologic Investigations No. 94). Northwest of the Fargo-Moorhead metro area, an earthquake with an epicenter located in southeast Saskatchewan, Canada, had a Mercalli Intensity of VI. No known reports of disturbances near the proposed project area resulted from either of these events. Additional earthquakes have been recorded west and northwest of the Fargo-Moorhead area near Goodrich, Hebron, Williston, and Grenora, North Dakota. These earthquakes have recorded or estimated to be between 1.5 to 3.7 magnitude. Included in Attachment I-1 is a map indicated the "Earthquakes in North Dakota", and was obtained from the North Dakota Geological Survey, *Geologic Investigation No. 94 (Reference I.12.3).*

10. In Paragraph I.3.0.2, the last two sentences in the paragraph, does this mean 17 CPT total were done next to existing borings? Also, could the CPT determine stratigraphy that correlated with the borings? Some discussion of the success or lack of success of the CPT soundings in determining stratigraphy would be helpful.

The paragraph was reworded to better indicate the number of borings used for correlation. The sentence reads as follows:

To better understand the CPT sounding results, 17 soundings were off-set from machine borings. In addition, at 11 of these locations, undisturbed samples were obtained.

A paragraph was added to the section I.3.2 Cone Pentration Tests to discuss the correlation between the CPT and machine borings

As mentioned previously, 17 of the CPT soundings were conducted off-set from machine boring locations. This was completed in order to correlate the CPT sounding and resulting "soil behavior type" (SBT) with the geologic formations indicated in the machine boring. It was found that the results of the SBT could not readily distinguish the contacts between the different upper foundation materials such as the Alluvium, Sherack, and Poplar River formations. It was discovered that there was a distinct change between the upper foundation materials and the Brenna formation and was readily apparent. It was also found that the CPT data and SBT could not be used to distinguish between the Brenna and Argusville materials, nor was there a parameter that could be used to distinguish between the two formations.

11. In Table I-2, it shows zero water content tests were completed. Assuming wc were determined for all the specimens tested, is there any value to listing the test? Also, does Rbar Disturbed refer to an Rbar conducted on a remolded sample? Suggest a different a name for this test.

    The water content was determined on the specimen as part of the triaxial and DS testing. Therefore water contents were not specifically obtained on the large, tube samples. The WC was removed from the table.

    The "R-bar Disturbed" Test is referring to a remolded sample. This test was intended to test compacted levee fill, but none were completed. This test was removed from the table.

12. It appeared the residual direct shear tests were direct shear tests conducted on pre-cut specimens. It appeared the residual shear stress may not have been reached with this type of testing. Were any of the residual shear strengths compared with values selected from correlations to Atterberg limits, effective normal stress, and clay fraction (Stark and Eid)? Also, why did the shear testing show the Oxidized Brenna and P.L. Sherack have lower residual friction angles than the Brenna?

    The residual direct shear tests were conducted on pre-cut samples and sheared until reaching 15% strain. The laboratory reported these results to the Corps. Upon review of the test results, the Corps asked that the laboratory conduct additional direct shear tests on two samples, Brenna (Fargo 09-26MU Sample #3) and Argusville (Fargo 09-27MU Sample #4), and complete three cycles of shear. The laboratory was able to do this by re-setting the sample and shear box after reaching 15% strain. The test results on these two samples showed that the shear stress at 15% strain was similar for all three cycles, indicating that for pre-cut specimens, the residual shear strength is likely attained at 15% strain.

    The residual shear strength parameters were not compared to correlations during Phase 2.

    The results of the residual shear strength tests for the Oxidized Brenna and PL Sherack were not reviewed in depth to determine why the residual shear strength was lower than the Brenna. This can be investigated if a need arises to reevaluate the In-Town Levee alternative or residual shear strength parameters.

13. For the case of the Oxidized Brenna, Brenna, and Argusville formations non-linear effective shear strengths envelopes were selected and used for design. What was the basis for this? Some discussion on why this was done would be quite helpful.

    The following discussion concerning the curvilinear envelope has been included in Section I.3.4

    The curvilinear shear strength envelope was developed for the Oxidized Brenna, Brenna, and Argusville formations for use in the effective stress analysis of the diversion channel excavated slope. The excavation of the diversion channel and the steady state seepage into the channel reduced the effective normal stresses in the Oxidized Brenna, Brenna, and Argusville. For example, the effective normal stresses in the Brenna ranged from 0.5 to 1.5 tons per square foot (tsf). The Mohr-Coulomb effective stress envelope used for the In-Town Levee alternative underestimated the available shear strength of the materials in this range of low confining stresses. So to appropriately represent the shear strength of the materials at the lower stresses, curvilinear shear strength envelopes were developed. The fact that the Oxidized Brenna, Brenna, and Argusville can be expected to be slightly overconsolidated, a small cohesion intercept was used.

14. Peak values of Su were used for EOC analysis for the in-town levees, while ultimate values of Su were used for the EOC analyses for the diversion alternatives. I know why this was done; the average reader doesn't. I'm not sure if the reasoning presented supports why one value is used for the levees and a different one for the cut channels. Perhaps a better argument to use ultimate Su values for the cut channels could be a slower loading rate (produced by slower construction) would mobilize a lower value of Su (although I'm not sure about this) or Su Ultimate represents the average Su (based on shear mode) along a shear surface better than Su Peak. I think some additional discussion/reasoning should be included in the write-up.

    A discussion on the reasoning for selecting ultimate undrained shear strength parameters was included in the report and is indicated below:

    In the case of analyzing the excavated slopes for the diversion channels, ultimate undrained shear strength parameters were used when analyzing the end-of-excavation condition of the diversion channel excavated slopes during Phase 3. The preliminary analyses completed during Phase 2 used peak undrained shear strength parameters. There are a few reasons why the use of ultimate, undrained shear strength parameters were used during Phase 3: 1) The excavation of the channel and the placement of the spoil piles, which are substantially higher than the levees (15 feet high) and extend for a considerable distance, influences the pore pressures over a larger area than just the placement of a levee; 2) The clays in the area are fissured and localized softening can occur along the fissures; the sample size does not capture a representative sampling of the fissure, therefore possibly indicating higher strengths then what would occur in the field; the use of the ultimate undrained strength is an reasonable way to address these differences; 3) An independent external peer review (IEPR) suggested that ultimate undrained shear strength parameters be used; review of the undrained shear strengths indicated a 10% to 30% reduction in strength from peak strengths to ultimate; selection of ultimate undrained shear strengths adds conservatism into the stability model and decreases the potential of failure during construction, resulting in a difficult and expensive fix. For either the peak or ultimate criteria, the selection of the undrained shear strength (c_u) was based on the results of the Q tests

15. Under paragraph I.5.1.2.1, it states only levees were assumed for Phase 2 design. If one assumed T-walls could be built to save expensive real estate could the in-town levees become a viable alternative?

    Assuming that T-walls could be built and be more cost effectively than levees would not change the fact that the In-Town Levee Alternative is not the best implementable plan. The reasoning for not considering the In-Town levee alternative can be found in Section I.5.9.

16. Under paragraph I.5.1.3.1, one reason why the pump stations and drainage structures are deep in the RRV are because the storm sewer outlets at the river end up being very deep because of the flat topography.

    The reasoning why the pump stations are deep was included in the paragraph.

17. In Table I-7, why do the setback distances used differ from those determined from the stability analyses?

    The speed of the project required that the layout be completed during the same time that the geotechnical analyses were being completed. There was not enough time during Phase 2 to revise the layout for the Phase 2 submittal. This the reason for the discrepancy and would have been resolved during Phase 3 if the In-Town Levee alternative was revised during this stage. The changes to the paragraph to discuss the differences is below.

    The setback distances determined during the preliminary and revised geotechnical analyses are summarized in Table I-7. The levee setback distances that were used in the layout of the In-Town Levee alternative are also indicated, which were based primarily on "preliminary analysis" using Houston Engineering's 0.1% annual chance event. The layout of the project proceeded faster than the geotechnical analyses could be revised. Therefore there are some reaches in which the layout setback distances are less than the required setback distances. These discrepancies are not considered to be substantially and would have a small effect on the evaluation of the In-Town Levee as an implementable plan.

18. Under paragraph I.5.8.1, does the generalized riprap cross-section referred to that was 3 feet wide mean the riprap was placed in a 3 foot thickness or layer?

    The riprap cross section was a 3-foot wide top width and 1V:3H slope. A figure was included in the report to depict this.

19. Under paragraph I.6.4.3.1, consider clarifying that non-circular failure surfaces generally produce lower computed FS along the riverbanks where the shear surface passes through a relatively thin weak layer (in effect, these failures can have long neutral blocks). Not sure non-circular shear surfaces would always be critical for the diversion channel. These were critical at

GF/EGF because of the long shear surfaces through a relatively thin, weak soil layer. Along the diversion the full thickness of the lake sediments will generally be present and a circular shear surface might fit that geometry fairly well.

The configuration of the diversion channel is approaching the configuration of the riverbanks along the Red River. The analyses show that a circular failure surface does not represent the critical condition. Slope/W allows one to conduct a “wedge” analysis which is a 3-segment wedge. This 3-segment wedge better represents the failure mechanism. The “optimization” feature in Slope/W was used to fully capture the non-circular type failure surfaces.

The paragraph was revised to clarify that lower FSs are computed for non-circular failures for riverbank stability than circular. The reason being is the long neutral block surface through a weak layer. A similar situation occurs in the diversion channel because the configuration approaches that of the riverbanks.

20. Could a Seep/W model be created to simulate uplift conditions in critical locations, such as the area near Dilworth? This might help refine uplift computations within the various soil layers beneath the diversion channel invert. Also, the heads in the sand were determined from piezometers and available well information. Did any of the CPT soundings penetrate into these sands? This could provide good water level information.

Placing total head boundary conditions on the vertical extent of the seepage models essentially simulates the uplift condition in areas in which there is a clean sand (SP) layer extending across the model. Because the clean sand has such a high permeability when compared to the overlying clay/silt materials, very little head loss occurs through the sand. Placing total head boundary at the interface of the clay and the sand would essentially result in the same results as has been modeled.

The uplift computations takes into consideration the type of soils above the clean sand. If less pervious materials are above the sands and beneath the impervious clay materials of the Brenna and Argusville formations, the thickness of the less pervious materials are transformed into an equivalent thickness of a material with a lower permeability. So in effect, the results from Seep/W will essentially provide similar uplift results as to that which has been presented.

The CPT soundings did penetrate into the sands at a few locations but dissipation tests were not done. So the results from the CPT soundings were not used to get water level information.

21. Sheet 4 of Attachment I-08 shows backcalculated friction angles in the Brenna for six of the design sections. Were these values used in the stability analyses or was a laboratory residual friction angle used?

    The Phase 2 In-Town Levee alternative stability analysis used laboratory residual friction angles. The residual zone was assumed for design purposes and did not represent known and/or existing failures. The residual zone represents a conservative approach. When the back calculations were completed, it was again without the use of a known and existing failure surface. So the back calculated friction angles do not represent what would be calculated using the St. Paul District's methodology and was used as a comparison it to the laboratory residual friction angle.

22. I think something could be learned from a FLAC analysis conducted in stages of excavation and spoil placement (i.e. simulating diversion construction). Neil could do this modeling most efficiently; I wouldn't mind giving it a shot if time permitted.

    I discussed with Neil completing a staged-construction analysis for the diversion channel excavation. He also agreed that something could be learned and could be used to supplement the traditional analyses completed, but not replace them entirely. The intent is to complete this staged-construction in the future and will likely be done using GeoStudio. Additional wording was added to the "Diversion Channel Analysis" under Section I.11.0 Additional Work to indicate that a staged-construction analysis should be completed.

23. In Figure I-6 does the dimension H1 refer to the spoil pile height indicated in Tables I-15 and 16? If so, you might want to add (H1) to the Spoil Pile Height column in the tables.

    The spoil pile height in the tables refers to H1. The tables have been fixed to indicate this.

24. In Tables I-17 and 18 it's not clear which FS values listed are effective stress and which are total stress. It appears they're all effective stress FS except for the last column, undrained global.

    The results of the effective stress analyses are indicated in the first 4 columns and the undrained global is indicated in the last column. The tables have been fixed to indicate the effective stress vs total stress analyses.

25. Were any of the diversion sections checked with another stability program (UT4, Slide, or FLAC/Slope)? If not, consider checking one or two.

    MN Diversion Section 6 and ND Diversion Section 2 were checked using Slide. The drained global stability results indicated similar factors of safety and failure surfaces. These results are included at the end of the respective diversion analysis result attachments.

26. In the future, consider installing a series of VWP near 12th Ave. S. where computed uplift FS values were low.

    It is recognized that additional exploration is needed around the Dilworth area to finalize the design for plans and specifications along with instrumentation. Agree that installing a VWP near 12th Ave S would provide beneficial information. It was not installed in July 2010 due lack of access.

27. Were the artesian conditions assumed for the uplift calculations used in the corresponding seepage and stability calculations? In other words, were head conditions set at the nodes representing the top of the sand in the Seep/W models?

    The artesian conditions assumed for the uplift calculations do not correspond to the seepage and stability calculations. The artesian conditions / piezometric head in the sand was assumed to be 7.5 feet below ground surface and selected base on review of the water levels in the surrounding MN DNR observation wells along with the Corps' instrumentation.

    As previously indicated in the response to Comment 20, the total boundary conditions on the vertical extent(s) of the Seep/W model provided essentially the same results as if the total head boundary conditions were placed on the top of the sand.

28. Some additional discussion of the instrumentation could be helpful, specifically what the piezometers are showing and if anything was or can be learned from them. Some piezometers indicate upward gradient, some a downward gradient or a perched condition, other sets seem to show more or less hydrostatic conditions.

    Some additional discussion on the instrumentation has been included in the report which is indicated below.

    Piezometers P1 through P6 were installed in August 2009 while the reminder of the piezometers, P7 through P23, have recently been installed. P7 though P15 were installed in June 2010 while P16 through P23 were installed in July 2010.

    Piezometers P1, P2, and P3 are located at Gooseberry Park in Moorhead, MN, adjacent to the Red River. During the fall of 2009, the piezometric level of P3 (deepest) was the highest, just slightly above P1 (shallowest), while P2 (middle) was approximately 10 feet lower. During the spring flood in 2010, all instruments indicated an increase in piezometric levels, with P1 (shallowest) piezometric levels essentially following the river stage. P2 (middle) and P3 (deepest) piezometric levels returned to fall 2009 levels. P1 (shallowest) levels have continued to fall and are currently approximately 4 feet above P2 (middle) levels. These trends would indicate that there is a higher pressure at depth.

Piezometers P4, P5, and P6 are located east of the MN Diversion channel alternative at the corner of 28th Ave N and 60th St N. These piezometers were installed in August 2009. The trend is showing that the piezometric level of P4 (shallowest) is the highest and has been fairly constant, varying at the most, 3 feet. The piezometric levels in P5 (middle) have risen slightly (approximate 2.5 feet) since installation, but began to level off in July 2010. P6 (deepest) is located in a sand formation and is reading the lowest piezometric levels, approximately 10 to 12 feet below ground surface. Again, P6 (deepest) has risen almost 3 feet since the installation, but began to level off in July 2010. The piezometric levels for P4, P5, and P6 can be interpreted either two ways: 1) there is a downward gradient through the clay formations into the sand formation because water is being pumped from the sand formation; 2) there are perched water tables within the different clay formations.

Piezometers P7, P8, and P9 were installed at the proposed location of the Red River Control Structure for the MN Diversion channel alternative. P7 (shallowest) and P9 (deepest) are indicating piezometric levels approximately 10 feet below ground surface, with P9 (deepest) piezometric levels being approximate 0.5 feet lower. The piezometric level for P8 (middle) is the lowest, being approximate 12.5 feet BGS. These trends could indicate that there is perched water table near the ground surface but more readings are needed to verify this.

Piezometers P10, P11, and P12 are located east of the MN Diversion channel alternative and 1 mile north of P4, P5, and P6, at the corner of 936rd Ave N and 60th St N. P10 (shallowest) is indicating piezometric levels approximately 10 feet below ground surface. P11 (middle) and P12 (deepest and installed in a sand formation) are indicating piezometric levels approximately 3 feet and 5 feet, lower than P10, respectively. This trend of the shallowest piezometer having the highest piezometric level while the deepest piezometer has the lowest level is similar to that of the instrument cluster of P4, P5, and P6.

Piezometers P13, P14, and P15 were installed at the proposed location of the Wild Rice River hydraulic structure on the ND Diversion channel. The piezometric levels for these piezometers are very different from all other instruments. P13 (shallowest) is indicating a piezometric level 28 feet BGS while P14 (middle) is approximately 34 feet BGS. This is very different from the other instruments, which typically have piezometric levels 10 feet to 15 feet BGS. The piezometric level for P15 (deepest) is approximately 80 feet BGS. Additional readings are needed before a definitive conclusion can be made as to what this trend is indicating as readings have only been collected since the end of June 2010.

Piezometers P17, P18, and P19 were installed at the proposed location of the Red River Control Structure for the ND Diversion channel alternative. P17 (shallowest) is indicating piezometric levels approximately 12 feet below ground surface. P18 (middle) and P19 (deepest) are indicating piezometric levels approximately 1 feet and 2 feet, lower than P17, respectively. This trend of the shallowest piezometer having the highest piezometric level while the deepest piezometer has the lowest level is similar to that of the instrument cluster of P4 – P6 and P10 – P12.

Piezometers P19 through P22 were installed east of the MN Diversion channel along HWY 10, at Dilworth. This instrumentation cluster was installed in July 2010. The data collector that was connected to these instruments was inundated with water so readings are not available for this cluster.

Piezometer P23 was installed at a depth of 43 feet BGS, in a sand formation east of the MN Diversion channel alignment. This piezometer was installed to observe the piezometric levels in the Buffalo aquifer and compare it to other readings in sand formations which are at greater depth. For P23, the piezometric level is approximately 14 feet BGS, which is similar to the readings of the other piezometers in the sand formations.

If any of the responses do not resolve Mr. Behling's questions or comments, please let me know and I'll attempt to clarify them.

Kurt Heckendorf, P.E.
Civil Engineer
Geotechnical and Geology Section

Heckendorf, Kurt A MVP

From: Wachman, Gregory S MVP
Sent: Friday, February 18, 2011 1:23 PM
To: Heckendorf, Kurt A MVP
Subject: RE: FMMFS: Peer Review of Phase 4 (UNCLASSIFIED)

Classification: UNCLASSIFIED
Caveats: NONE

Kurt,

I saved a version of the writeup with my initials at the end and made some suggested changes and comments. The comments and changes were mostly editorial in nature. The seepage and stability work was good as far as I could tell, but I had some comments on the presentation that you can consider. There is a lot of stability work, so I didn't look at it too closely - if you want me to take a closer look at anything in particular just let me know.

Greg

Comments not made in the writeup:

1) Many of the Phase 2 stability plates are cut off, but not magnified enough to show details well. Also, things like model points are shown unnecessarily.

2) Additional shear surfaces are shown on the Phase 2 models without explanation - is there a reason for them to be shown on the plates? In many cases they cover up the final critical surface.

3) Presentation of the Phase 3 and 4 seepage/stability: There seem to be problems with the axes in relation to the area of interest on a number of sections - the vertical axis could be moved further to the right.

4) The seepage plates might benefit from head contour labels and total head boundary condition descriptions. The flow vectors aren't particularly useful.

5) The grids in the wedge analysis of MN div. sec #9a and 9b are too close to each other - doing this can produce unreliable results. Also, they obscure the critical shear surface.

-----Original Message-----
From: Heckendorf, Kurt A MVP
Sent: Wednesday, February 16, 2011 11:13 AM
To: Wachman, Gregory S MVP
Subject: FMMFS: Peer Review of Phase 4 (UNCLASSIFIED)

Classification: UNCLASSIFIED
Caveats: NONE

Greg,

Would you have time in the next week or so to complete a peer review of the Phase 4 Fargo-Moorhead geotechnical documents? This would include review of the write-up and the stability models.

Labor Code: 138D83

The documents are located on the server and can be accessed through the following links.

Phase 4 Report Documents: \\mvd\mvp\PROJECTS\SA\SA Fargo-Moorhead Metro FDR-153866\02FeasibilityFEA\Geotech\Phase 4 Report

Phase 4 slope stability models with invert raise: \\mvd\mvp\PROJECTS\SA\SA Fargo-Moorhead Metro FDR-153866\02FeasibilityFEA\Geotech\Stability\Phase 4\ND Invert Raise\03 P4 invert Vr4

Some additional documents that may help in your review:

Phase 3 slope stability models: \\mvd\mvp\PROJECTS\SA\SA Fargo-Moorhead Metro FDR-153866\02FeasibilityFEA\Geotech\Stability\Phase 3

Shear Strength Parameters: \\mvd\mvp\PROJECTS\SA\SA Fargo-Moorhead Metro FDR-153866\02FeasibilityFEA\Geotech\Parameters

Instrumentation: \\mvd\mvp\PROJECTS\SA\SA Fargo-Moorhead Metro FDR-153866\02FeasibilityFEA\Geotech\Instrumentation

Please let me know if you have any questions.

Respectfully,
Kurt
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