Appendix I
Geotechnical Design and Geology

Fargo-Moorhead Metropolitan Area
Flood Risk Management

Final Feasibility Report
and Environmental Impact Statement

July 2011

US Army Corps
of Engineers ®

Prepared by:
U.S. Army Corps of Engineers
St. Paul District
180 Fifth Street East, Suite 700
St. Paul, Minnesota 55101-1678

Page intentionally left blank

Fargo – Moorhead Metro Feasibility Study
Fargo, North Dakota and Moorhead, Minnesota
Appendix I
Geotechnical Design and Geology

Table of Contents

Index of Tables

Index of Figures

Attachments

Fargo – Moorhead Metro Feasibility Study
Fargo, North Dakota and Moorhead, Minnesota
Appendix I
Geotechnical Design and Geology

I.1.0. PROJECT DESCRIPTION

I.1.0.1. The Fargo-Moorhead Metro Feasibility Study (FMMFS) was initiated in the fall of 2008. The purpose of this study is to identify measures to reduce the flood risk in the Fargo-Moorhead (FM) metro area. The study consists of six major steps that include:

(1) Specification of water and related land resources problems and opportunities;

(2) Inventory, forecast and analysis of water and related land resources conditions within the study area;

(3) Formulation of alternative plans;

(4) Evaluation of the effects of the alternative plans;

(5) Comparison of the alternative plans; and

(6) Selection of the recommended plan based upon the comparison of the alternative plans.

I.1.0.2. The feasibility study was initially broken into three major phases. In September 2010, it was determined that the study would be extended to allow for additional analysis to be completed, creating a fourth phase. The first phase, Phase 1 (September 2008 – May 2009), was to develop low level of detail assessments of the alternatives to determine which concepts seemed feasible. This phase was dominated by hydrologic, hydraulic, and economic analyses. The major geotechnical design effort during this phase was to evaluate the reliability of the existing levees that the cities of Fargo and Moorhead have in place. This effort is documented in Appendix H, Geotechnical Analysis: Credit to Existing Levees and the results used in the economic analyses. The assessment of the alternatives in Phase 1 indicated that the in-town levee alternative had a benefit/cost ratio (BCR) of 1.0 while the preliminary diversion concept had a BCR of 0.65. It was decided that further evaluation of these alternatives was needed.

I.1.0.3. The second phase of the study, Phase 2 (May 2009 – November 2009), required that further evaluation and technical analyses be completed on the most implementable projects. Three alternatives that were likely to be implementable were selected:

(1) In-Town Levee Alternatives

(2) Minnesota Diversion Alternatives

(3) North Dakota Diversion Alternatives

I.1.0.4. During Phase 2, the geotechnical design became a major effort. Initially, a soil exploration and testing program was developed and implemented for the In-Town Levee alternative and the Minnesota (MN) Diversion alternative. This data was used in the geotechnical analyses completed to determine the required setback distances for the In-Town Levee alternatives and complete the preliminary evaluation of slope stability for the MN Diversion channel.

I.1.0.5. During Phase 2, the hydrologic, hydraulic, and economic analyses were refined and preliminary cost estimates developed for the different alternatives. By the end of November 2009, the initial results indicated that the most viable alternatives were the MN and ND Diversion channels. The In-Town Levee alternative was not recommended for further evaluation for a number of reasons, listed below:

(1) Top elevation is limited to highest natural ground, which nearly corresponds with 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.

I.1.0.6. Refinements to both the MN and ND Diversion alternatives were made during the third phase of the study, Phase 3 (December 2009 – August 2010). These refinements were made to the hydrology, hydraulics, geotechnical, and economic analyses in order to determine the National Economic Development (NED) plan.

I.1.0.7. The major geotechnical task for Phase 3 was completing the evaluation of the slope stability for the MN and ND Diversion alternatives. In December 2009, a soil exploration and testing program was initiated and completed along the North Dakota (ND) Diversion alternative. Additional soil exploration was conducted along both alignments from May to July 2010.

I.1.0.8. The fourth phase (September 2010 – February 2011) consisted of completing additional analysis on alternatives and impacts. The majority of the additional analysis involved refinement to the hydrology and hydraulics. The design of the MN Diversion alternative remained unchanged from Phase 3 during this phase. Revisions to the ND Diversion alternative were done which consisted of changing the configuration of the channel, including a large storage cell, and staging water upstream of the project.

I.1.0.9. The geotechnical analyses completed and geological interpretation for the Fargo-Moorhead Metro Feasibility Study are presented in this appendix.

I.2.0. REGIONAL GEOLOGY

I.2.0. Physiography

I.2.0.1. The adjacent cities of Fargo, North Dakota and Moorhead, Minnesota are located within the Red River Valley Division of the Central Lowlands Physiographic Province. The watershed of the Red River of the North encompasses the northeastern corner of South Dakota, much of eastern North Dakota, northwestern Minnesota, and a small portion of the province of Manitoba, Canada. The river is formed by the confluence of the Bois de Sioux and Ottertail rivers at the cities of Breckenridge, Minnesota and Wahpeton, North Dakota. Flowing northward, the Red River of the North forms most of the boundary between North Dakota and Minnesota. Upstream of the proposed project, the river drains an area of about 30,100 square miles including the Devils Lake sub-basin.

I.2.0.2. The river valley consists of a broad, nearly flat plain flanked on either side by gradual hills or higher ground. This plain is not derived from river erosion, but is an ancient glacial lake bed. North-south trending, the plain extends approximately 245 miles within the United States, and is about 15 miles in width on the extreme southern end before rapidly widening to 60-70 miles. The plain is generally inclined northward with an average slope of less than 1 foot per mile. The Red River of the North flows in a tightly meandering course within this plain for about 400 river miles before arriving at the Canadian border, with a river surface elevation drop from approximately 945 feet (msl) to 740 feet. The Red River meander belt may be up to 1.5 miles wide. Ultimately the river flows into Lake Winnipeg, Manitoba, Canada. Some of the principal tributaries of the Red River of the North in the project vicinity include the Sheyenne River, Wild Rice River of ND, Maple River, Rush River, and the Buffalo River.

I.2.1. Topography

I.2.1.1. The upper reaches of the Red River of the North watershed lie in drift prairie plateaus, while the river's main stem flows through an ancient glacial lake bed. The uplands vary in elevation from approximately 2,150 to 1,200 feet (msl), while the elevation of the valley plain varies from about 980 feet near Lake Traverse, Minnesota to 800 feet at the Canadian border. Westward the plain slopes gently nearly to the elevation of the upland. Eastward of the valley plain lies a relatively hilly area that merges into lakes and swamps of the uplands area. Perpendicular to the trend of the main stem of the Red River the valley has an average slope of 2 to 3 feet per mile. The slope of the main stem between Fargo, North Dakota and Grand Forks (154 river miles) averages slightly more than one-half foot per mile. The pools of three low head dams at Fargo/Moorhead on the Red River extend for several miles upstream at each location.

I.2.1.2. Previously proposed in-town levee alignments would have been confined to an area approximately 300-500 feet wide, along the banks of the Red River of the North. Due to modern erosional processes, topographic relief is more pronounced along this narrow zone, than is typical for most of the present day Red River Valley. Nearest the river primary banks exist some 5 to 15 feet above the normal water surface. Above the primary banks the ground slopes in either a gentle, flat or somewhat hummocky fashion to the foot of a secondary bank. The secondary bank usually rises relatively steeply 20 to 25 feet above the primary bank, before flattening out into the ancient glacial lake bottom that forms a majority of the modern watershed. Most of the present-day human activity begins within tens of feet of the top of the secondary bank.

I.2.1.3. Scarps from riverbank slides are typically located in the secondary bank, 100 to 400 feet from the edge of the river. Often the slides progress up slope thereby leading to a hummocky appearance between the tops of the primary and secondary banks. The slides may extend for several hundred feet along the river bank.

I.2.1.4. It has been noted that slides in the Red River Valley are most typically found to exist on the outside of river bends. These slides are likely initiated, in part, by the scouring action of the river on the toe of the primary or lower river bank. In addition to slides in the upper or secondary banks, smaller scale sloughing of the lower river banks is frequently observed.

I.2.1.5. Topography of the Red River Valley, outside of the river channel, is predominately flat. The proposed North Dakota Diversion alternative follows a topographical route that progresses from the south, just north of Oxbow, ND, to the north end of the project where the project channel rejoins the natural channel north of Argusville, ND, and Georgetown, MN. The elevation from south to north drops from about 915 feet to 880 (NAVD 1988 adj). This relatively steady slope to the north falters very little, but is cut by 5 different river channels. The Wild Rice and Sheyenne are the most influential, though their river bottoms drop beneath their banks by only about 20 feet, to elevations of 893 and 902 feet at their respective proposed diversion channel crossings.

I.2.1.6. The proposed Minnesota Diversion alternative follows a topographical route that drops from about 910 feet at the south, where the river is proposed to divert the Red River, just north of confluence of the Wild Rice River, to approximately 885 feet (NAVD 1988 adj) at the north end of the project where the diversion rejoins the natural channel north of Oakport, MN. This relatively steady slope to the north falters very little, but crosses a few eskers that create low-lying ridges. The largest of these, at the north end of the proposed channel, is about 150 feet wide with a relief of approximately 4 feet.

I.2.2. Geology

I.2.2.1. Most of the observed conditions that are the basis of this report are closely related to the geologic setting within the proposed project site. Soil borings were obtained along the river course and diversion alignments in order to more closely define the site specific subsurface conditions.

I.2.2.2. The geology influencing the Red River of the North Valley along the North Dakota / Minnesota border is the legacy of glacial Lake Agassiz and recent fluvial/alluvial processes of the Red River and its tributaries. During the glacial period, the entire watershed of the present day Red River of the North was covered by a continental glacier. Periodically, as the glacial ice melted and retreated northward, huge ice dams were formed which blocked the natural northerly drainage pattern. Glacial Lake Agassiz, which covered approximately 200,000 square miles, resulted from the ice damming and subsequent ponding of meltwaters. The lake is believed to have existed from approximately 13,800 to 9,000 years before present (B.P.), during the Late Wisconsin Glacial Episode of the Pleistocene Epoch. At its maximum extent, Lake Agassiz is believed to have been approximately 150 feet deep in the vicinity of Fargo. As the glacier receded and advanced, fluctuations of the lake levels resulted in corresponding variations of the sediment types. After the glacial lake drained for the final time, the relatively youthful drainage pattern of the present Red River of the North established itself on top of the lake sediments. A useful analogy may be to consider the river course to be little more than a scratch in a broad table top. The basis for most of the stability analysis prepared for this report is a direct result of the geologic setting of the present day Red River Valley. This brief history of the Pleistocene Epoch and related stratigraphy is presented to establish background for discussions of the engineering characteristics of the various soil units. Much of this information has been previously detailed in:

- <u>North Dakota Geological Survey Bulletin No. 47</u> (Klausing, 1968),
- <u>North Dakota Geological Survey Miscellaneous Series 52</u> (Harris, Moran, & Clayton, 1974),
- <u>North Dakota Geological Survey Report of Investigation No. 60</u> (Arndt, 1977),
- <u>General Design Memorandum for Flood Control-East Grand Forks</u> (Corps of Engineers, 1986).

I.2.2.3. The stratigraphic units will be discussed from bottom-most to ground surface.

I.2.2.4. **<u>Bedrock</u>**. Bedrock lies at an estimated depth greater than 300 feet beneath the glacial sediments in the region. The bedrock is likely composed of Cretaceous Period, shales and sandstones. The bedrock lies well below the influence of the proposed project.

I.2.2.5. **<u>Undifferentiated Glacial Sediment</u>**. Up to 400 feet of till may overlie the bedrock surface. An unknown amount of glacial drift is included in the estimated thickness. The till surface generally slopes from south to north at a slightly greater rate than the ground surface. The till ranges from a gravelly, sandy clay to a gravelly, silty, clayey sand.

I.2.2.6. **<u>Argusville Formation (Unit A)</u>**. The lowest foundation unit of interest is the Argusville Formation (Unit A). In the project location, the depositional period for this formation likely straddled the Pre-Caledonian Advance of the Lostwood Glaciation (Cass Phase) and continued throughout the early portion of the Caledonian Advance approximately 13,500 – 12,800 years BP. This unit provides suitable foundation for nearly all types of pile-founded construction in the Fargo-Moorhead area. ‘Unit A” is composed mostly of gray to dark gray silty, sandy, pebble-loam (till). Locally the uppermost portion of this formation may be made up of sand and gravel. The Argusville Formation (Unit A) may be characterized as stiff to hard, moist, low plasticity, variably sandy, silty, gray, glacial clay, with a liquid limit generally less than 40. A gritty texture and high SPT values are the distinguishing features of "Unit A". The contact with the overlying Argusville Formation is gradational. The engineering properties are the best of any unit in the project vicinity.

I.2.2.7. **Argusville Formation**. In the project location, the depositional period for this formation likely straddled the Pre-Caledonian Advance of the Lostwood Glaciation (Cass Phase) and continued throughout the early portion of the Caledonian Advance approximately 13,500 – 12,800 years BP. In contrast to the underlying glacial till, this unit has only scattered sand and gravel, and occasional small till inclusions up to 3 inches in diameter. The sand, gravel and till inclusions in the deposit may have been derived from rafts of floating glacial ice. The Argusville Formation may be characterized as massive, soft to medium stiff, wet, highly plastic, slightly sandy or gravelly, dark gray, glacio-lacustrine clay, with a liquid limit generally less than 80. A slightly gritty texture is the most distinctive feature of the Argusville Formation. Along the proposed project alignment, the Argusville Formation has an average thickness of 20-40 feet. The contact with the overlying Brenna Formation is gradational. The engineering properties are very poor and should be considered as poor as, or only slightly better than, the overlying Brenna unit.

I.2.2.8. **Brenna Formation**. The second high-water phase (or Lockhart Phase) of Lake Agassiz occurred from approximately 11,600 to 11,000 years BP and resulted in the deposition of the Brenna Formation. The Brenna Formation is characterized as a uniform, soft to very soft, wet, highly plastic, dark grey, glacio-lacustrine clay, with little or no visible structure and a liquid limit generally greater than 80. The major source of sediment for this formation was eroded Pierre Shale bedrock. Slickensides are commonly observed on shear planes in freshly broken samples. Soft, calcareous silty nodules are common, increasing with depth, and silty laminae are occasionally present in the lower zone of the formation. The Brenna Formation is notoriously unstable as a foundation material throughout the Red River of the North Valley. In the project area, the unit has an average thickness of approximately 20-40 feet. The contact with the overlying Sherack or Poplar River Formations is an erosional unconformity. The upper 3 to 10 feet of the Brenna Formation is variably harder, occasionally oxidized, and more consolidated than the bulk of the Brenna Formation, probably due to desiccation during sub-aerial exposure. It is not thick enough, however, to substantially alter the basic weakness inherent within the formation.

I.2.2.9. **Poplar River Formation**. Between 11,000 to 9,000 years BP (the Moorhead Phase), Glacial Lake Agassiz experienced several water level fluctuations. During periods that portions of the lake bottom were exposed to sub-aerial erosion, a drainage network similar to the modern Red River system developed. The Poplar River Formation is the result of deposition of fluvial channel and overbank sediments during this phase. Along the project alignment the formation is found as two members- the West Fargo member is characterized as a laminated, soft to medium stiff, wet, silty, organic rich clay with sand and silt seams. The Harwood member of the Poplar River unit is a medium stiff, moist, silty clay, brownish gray with a mealy texture. Locally, peat beds up to 3 feet thick may be encountered in this unit. Where present, it typically occurs as trough-shaped features from a few hundred feet to a mile in width that are incised into the top of the Brenna Formation. In the proposed project area, it averages 8 feet in thickness, and is found more abundantly towards the northwest in the project area. The contact with the overlying Sherack Formation is conformable, usually interbedded, and gradational. Locally pervious deposits of the Poplar River Formation can produce substantial amounts of water. If a significant body of this pervious material were encountered during construction, it could, potentially, pose a dewatering problem.

I.2.2.10. **Sherack Formation**. The third and final high-water phase (or Emerson Phase) of Glacial Lake Agassiz occurred from approximately 9,900 to 9,000 years BP and resulted in the deposition of the Sherack Formation. The Sherack Formation is typically characterized as laminated, medium stiff, glacio-lacustrine silty clay and clayey silt with minor amounts of sand, gypsum and calcite crystals, and /or organics. The upper portion of this unit is usually brown to yellow-brown with frequent iron oxide or calcareous concretions but the base is grey. Glacial material from the uplands, instead of shale bedrock, was the major source of sediment for the Sherack Formation. The contact with the overlying present period (Holocene Epoch) sediments is an erosional unconformity.

I.2.2.11. The Sherack Formation has been impacted more than any other unit in the project reach by erosion and flooding near the banks of the Red River. Often, below the secondary (upper) banks, substantial portions have been removed and replaced by relatively recent alluvial and/or fluvial sediments. Slope failures have also displaced the Sherack Formation riverward of the secondary banks. Riverward of the upper bank, average elevation and thickness of formation figures are so variable that the only practical method for evaluation is to reference a specific cross-section before any meaningful analysis may begin. In the project vicinity the formation averages approximately 10 to 20 feet in thickness. The engineering properties are the best of any glacio-lacustrine unit in the project area.

I.2.2.12. **Present period sediments**. As the northeastern outlets for the lake opened for the final time, it is estimated that Glacial Lake Agassiz retreated from North Dakota by about 9,000 years BP, and was wholly gone as a Pleistocene phenomenon by approximately 8,500 years BP. An immature drainage system developed along the floor of the glacial lake bed with tributary streams rising in the high ground to the west and east. The present day Red River of the North watershed is the result of this post-glacial erosional activity. Flood sediments from the Red River blanket the valley ground surface now in a meander belt approximately 1 to 1.5 miles on either side of the existing river. These surface sediments may be characterized generally as soft to medium stiff, fluvial or alluvial, silty clay or clayey silt. Variably, the unit may contain sand or organic matter, including shells, and range from massive to weakly laminated. Adjacent to urban development, fill and rubble are frequently components of the upper sediments. The river exhibits no well defined flood plain. The depth of these surface sediments is highly variable and may range from approximately 1 up to 15 feet in thickness along the proposed project alignment.

I.2.3. Structure

I.2.3.1. Jointing within the glacio-lacustrine deposits at the site has been observed in the boreholes infrequently. These joints had a short vertical magnitude and no attempts were made to determine their orientation. Evidence of sliding is usually most noticeable near the main-stem valley of the Red River or its major tributaries. Obvious surficial evidence of slide activity noted includes crack development in the ground surface, cracking of walls, foundations, and drainage utilities, scarps and hummocky topography within and below the secondary (upper) river bank, leaning trees, utility poles, and structures. The borings revealed evidence of slide activity also. Jumbled, high angle or displaced bedding, soil units out of stratigraphic sequence or at displaced elevations, and older or pre-existing slickensides were all used to identify areas where sliding is occurring now or has occurred within the relatively recent past. Typically, but not exclusively, these areas were also located on the outside of river bends. All of the evidence gathered was used to determine which criteria was appropriate for any slope stability analysis in the project reach. Sliding, as observed in numerous exposures along the Red River of the North, is generally oriented so that the major plane of movement is perpendicular to the trend of the river.

I.2.4. Site Hydrogeology

I.2.4.1. The generally low permeability of the soils within the proposed project boundaries makes determination and prediction of groundwater levels challenging. Occasionally some coarse sediment seams are sufficiently pervious to allow a confident measurement. Earlier efforts to correlate soil color with groundwater conditions are now thought to be unreliable. In an attempt to obtain more useful groundwater information, the drilling method was modified. An offset hole was drilled and allowed to stand open as long as practical so that a water level could be obtained. In essence, 2 holes were drilled at many locations so that little time was wasted waiting on a single borehole to develop a stable water level. Several water level holes were allowed to stand open overnight, although all the water levels obtained by this method were not definitively known to be stable. The information gathered has helped to shed light on this problem; however the results are still not entirely definitive.

I.2.4.2. Groundwater levels in the Fargo area are high. Soil borings taken for this study revealed groundwater to be within an approximate range of 5 to 20 feet below the ground surface. Experience indicates that water levels fluctuate seasonally, with fall /winter conditions exhibiting the lowest measured water levels as might be expected. The water surface profile from the secondary bank riverward varies also, with the flattest profile occurring during the fall/winter months. It is reasonable to expect these same conditions to be reflected in Fargo. Water levels in the banks do fluctuate with the level in the river. Data is not available to ascertain the rate at which the banks become saturated with river water. River banks observed weeks after a high water event reveal that there is usually a seepage point 2 to 3 feet above the river water surface at approximately the same level as the measured water level in the borehole adjacent to the lowest or primary river bank. Water levels are most frequently, but not exclusively, measured in the alluvial and/or fluvial surface deposits. Levels not obtained in the alluvial/fluvial soils (or fill) are found in permeable zones of the upper portions of the Sherack Formation. Dry holes are occasionally encountered, but are the exception rather than the rule.

I.2.5. Seismic Risk and Earthquake History

I.2.5.1. The Fargo-Moorhead metro area in the Red River of the North Valley is one of the least seismically active places in the United States. According to Figure C-1, Seismic Zone Map of the United State from ER 1110-2-1806 (Reference I.13.7), the Fargo-Moorhead metro area is located within earthquake Seismic Zone 0. The Seismic Zone of 0 is associated with the least risk area while a Seismic Zone 4 is associated with the greatest risk. A reproduction of Figure C-1 with the Fargo-Moorhead metro location indicated is included in Attachment I-1.

I.2.5.2. 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 (Reference I.13.2). 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.13.2).

I.2.5.3. To the east, two faults exist in Minnesota that could possibly affect the project. The first is known as the Vermillion fault, which is an inactive Precambrian fault that extends eastward from Northwestern Minnesota near the Twin Lakes area in the Glacial Lake Agassiz plain. The westernmost extent of the fault is approximately 30 miles east of the Red River. It bends southeastward in an arcuate path through the northern part of Minnesota and enters Canada north of Ely, Minnesota. It is approximately 250 miles long. Its surface expression is defined by a narrow, linear topographic depression, which is occasionally occupied by deep, elongate lakes. The second fault, known as the Morris fault, extends diagonally from the town of Morris, Minnesota to the Brainerd area in west-central Minnesota, roughly 100-130 miles east/southeast of Fargo. Like the Vermillion fault, it is confined to the Precambrian bedrock and is not considered tectonically active, although some seismic activity has been associated with the Morris fault. In 1975, an earthquake with a Modified Mercalli Intensity of VI occurred near the town of Morris. This earthquake occurred about 10 miles west-northwest of Morris at a depth of 3-5 miles. It is one of the best documented earthquakes in Minnesota history, and possibly the largest. In Fargo and in Valley City, North Dakota, a Modified Mercalli

Intensity of II (felt by persons at rest, on upper floors, or favorably placed) was assigned for this event. The Modified Mercalli Intensity Scale ranges from I (not felt) to XII (damage nearly total). Five other earthquakes have been linked to the Morris fault since the year 1860. The most recent earthquake in Minnesota occurred along the western edge of the Morris fault in 1993 near the town of Graceville. It had a magnitude of 4.1 on the Richter Scale and a Mercalli Intensity of V. The Graceville earthquake occurred at an estimated depth of 7 miles.

I.2.5.4. Eighteen recorded earthquakes have occurred in Minnesota since 1860. Some are associated with glacial isostatic rebound, particularly in the northeast region of the state near Duluth. No earthquake has exceeded the magnitude or intensity of the Morris event in 1975. An approximate frequency of between 10 and 30 years has been established for minor earthquakes in Minnesota.

I.2.5.5. The peak horizontal ground accelerations (PGA) for various annual exceedance rates were determined using the 2008 NSHMP PSHA Interactive Deaggregation web site (http://eqint.cr.usgs.gov/deaggint/2008/) from the U.S. Geological Survey (USGS). The estimated PGA for the mean return time of 475 years, 2475 years, and 4975 years, was 0.007g, 0.025g, and 0.04g, respectively, which are very small. The PSH deaggregation charts are presented in Attachment I-1.

I.2.5.6. The probability of a magnitude 5 or larger earthquake occurring within 50 km of the Fargo-Moorhead was also determined using the USGS 2009 Earthquake Probability Mapping tool (http://geohazards.usgs.gov/eqprob/2009/index.php). Maps indicating the probability of earthquakes of magnitude 5 or larger occurring within "50 year and 50 KM" and "250 years and 50 KM" are also included in Attachment I-1. For both scenarios, the probability of a magnitude 5 earthquake or larger is between 0.0 and 0.01.

I.2.5.7. A search for earthquakes within a 1000 kilometer circular area of the Fargo-Moorhead metro area was also conducted. Again, the U.S. Geological Survey's "Circular Area Earthquake Search" tool found on their website (http://earthquake.usgs.gov/earthquakes/eqarchives/epic/epic_circ.php) was used. The search resulted in finding 219 earthquakes that have occurred between 1804 and 2010. The earthquakes ranged in magnitude from 2.3 to 6.2. The 6.2 magnitude occurred on November 8, 1882 at a distance of 620 miles from Fargo-Moorhead, near Fort Collins, Colorado. A table that summarizes these earthquakes is presented in Attachment I-1 along with a map that indicates the locations and magnitudes.

I.2.5.8. The seismic risk assessment for the Red River Valley region relies largely on earthquake history. The absence of major or catastrophic earthquakes, coupled with the infrequency of these earthquakes in general, implies an extremely low risk level for seismic activity in the vicinity of Fargo-Moorhead metro area. Based on this low risk, the performance of project features were not assessed using earthquake loading cases.

I.3.0. SUBSURFACE INVESTIGATION

I.3.0.1. The subsurface investigation for the FMMFS was completed at different phases of the study. The first subsurface investigation program was developed and initiated in the Spring/Summer 2009. This program was developed to obtain subsurface information for the In-Town Levee alternative and also the Minnesota Diversion alternative. This program consisted of machine borings and the use of the cone penetration test (CPT). A subsequent subsurface investigation program consisting of only machine borings was completed in December 2009. This program was developed to obtain subsurface information along the North Dakota Diversion alternative. A third subsurface investigation, consisting again of only machine borings, was completed between May and July 2010. This program was used to obtain additional subsurface information along the MN Diversion alignment, especially in the area near Dilworth, MN and at the structure locations. Exploration was also conducted along the ND Diversion alignment, with most being taken near the proposed locations of the structures. The final subsurface investigation was conducted between November 2010 and March 2011. The program was used to gather additional subsurface information along the ND Diversion alignment and at structural locations. Both machine borings and CPTs were used.

I.3.0.2. The entire soil exploration program consisted of 123 machine borings, 22 off-set undisturbed borings, and 101 cone penetration test (CPT) soundings. The off-set undisturbed borings were completed in order to obtain 5-inch undisturbed samples to conduct shear strength and consolidation testing. To better understand the CPT sounding results, 25 soundings were off-set from machine borings. In addition, at 15 of these locations, undisturbed samples were obtained.

I.3.0.3. The number and type of exploratory holes completed for the different alignments varied. Table I - 1 below indicated the type and number of explorations completed for each alignment. For the In-Town Levee alternatives, the exploratory holes were typically located on the outside bends of the river, where stability of the natural bank is of greatest concern. In general, a machine boring was completed near the river's edge on the primary bank while a second exploratory hole was conducted on the secondary bank. The secondary bank hole was generally a CPT sounding, but some were machine borings. For the MN Diversion alignment, the exploratory holes were generally located in a checker board pattern, with exploratory holes being located on alternate sides of the diversion channel alignment. In the area of Dilworth, MN, the exploratory holes were completed along sections running east-west in order to determine the trend of the Buffalo Aquifer. For the ND Diversion alignment, the exploratory holes were generally done near the centerline.

Table I - 1: Summary of Soil Exploration and Instrumentation

Location	Machine Borings	Undisturbed Borings	CPT	Piezometer	Slope Inclinometer
Fargo - In-Town Levees	22	4	19	0	1
Moorhead - In-Town Levees	21	3	16	1	1
ND Diversion Channel	40	12	34	2	0
MN Diversion Channel	40	3	32	5	0
Total	**123**	**22**	**101**	**8**	**2**

I.3.0.4. The locations of the exploratory holes are shown on the figures presented in Attachment I-2. The exploratory holes are numbered in sequential order as they were obtained. Each side of the river has its own sequential order, with the FMMFS exploratory holes for the Fargo side starting at 21 and for the Moorhead side, 11. This is due to the fact that the St. Paul District had previously obtained borings in the area. The exploratory holes are labeled to indicate the year in which the hole was obtained, the sequential order of the hole, and the type (i.e. 09-21M). Machine borings are indicated by “M” (i.e. 09-21M) while off-set undisturbed machine borings are indicated by “MU” (i.e. 09-32MU). CPT soundings are indicated by “C” (i.e. 09-23C). For locations in which multiple types of exploratory holes are off-set from each other, each type of hole will have the same year and sequential order, with the type varying in the name.

I.3.1. Borings

I.3.1.1. Many of the machine borings were conducted using an all-terrain vehicle (ATV) rig. The ATV rig was more maneuverable and minimized the disturbance of the area when compared to a truck mounted rig or the CPT truck mounted rig. Therefore, the ATV rig was used to complete the borings located on the primary bank, near the river’s edge. The truck mounted rig was utilized at locations along the diversion channel alternatives where access and disturbance were not issues.

I.3.1.2. The machine borings were conducted using a continuous sampling method which allowed the soils to be classified in the field by a geologist. The sampling was done in 5 foot flights. The first 3 feet were sampled with a modified 2”x2 ½” split spoon, followed by the 2” standard penetration spoon for the remaining 2 feet. The already sampled 5 feet was then cleaned out with the noted drilling method, and sampling continued. The larger spoon above the standard spoon cleaned the hole out large enough to not affect the SPT blow counts of the standard spoon. The drive of the modified 2”x2 ½” spoon was recorded on the field logs, but not digitally recorded. The standard SPT blows were recorded in the field with a hand held device into PLog software and are the blow counts presented on the drafted logs. SPT blows were performed dropping a 140 pound hammer 30”, unless otherwise stated, with the auto-hammer corresponding to the drill rig performing the boring; either the CME-750, or the Diedrich 120. Disturbed samples were also collected and tested for moisture content, Atterberg limits, and in some cases, grain size distribution.

I.3.1.3. The undisturbed soil borings were located off-set to the machine borings in order to obtain 5-inch undisturbed samples. Generally at each undisturbed machine boring, one undisturbed sample was obtained from each formation. A total of 16 undisturbed borings were completed, obtaining 63 undisturbed samples. These undisturbed borings were distributed throughout the project, with four completed for the in-town levee alignment in Fargo, three for the in-town levee alignment in Moorhead, three along the Minnesota Diversion alternative, and six along the North Dakota Diversion alternative.

I.3.1.4. The boring logs are presented in Attachment I-3.

I.3.2. Cone Penetration Tests

I.3.2.1. The CPT method of soil exploration was implemented during the first and fourth exploration programs. For the first exploration program in the Spring/Summer 2009, the CPT soundings were conducted by the USACE Savannah District using a 20-ton CPT truck mounted rig. The CPT cone was a Hogentogler piezocone that measured U2 pore pressure located at the shoulder of the cone. Due to the large size and weight of the CPT rig, CPT soundings were conducted on the secondary bank at locations which would minimize the disturbance of the area. This typically meant that the CPT soundings were conducted on or near a paved street. USACE Savannah District again conducted the CPT soundings during the fourth exploration, November 2010 through January 2011. For this program, a 20-ton track rig was implemented. With the track rig, a Hogentogler seismic piezocone was used to obtain the shear wave velocity of the soils.

I.3.2.2. As mentioned previously, 16 of the CPT soundings were conducted off-set from machine boring locations during the first exploration program. 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.

I.3.2.3. For the fourth exploration program, the seismic cone was used implemented in order to obtain the shear wave velocity of the soils. It was thought that the shear wave velocity could help distinguish between the Brenna and Argusville formations. It was found that there is a general trend in the shear wave velocity that can be used to help distinguish between the Brenna/Argusville contact. The trend is that the shear wave velocity increases in the Argusville compared to the Brenna formation. This trend is based on only two CPTs off-set from the machine borings. Additional machine borings should be completed adjacent to CPT sounding locations to verify this trend.

I.3.2.4. The CPT soundings are presented in Attachment I-3.

I.3.3. Testing

I.3.3.1. Testing was done on disturbed samples (jar samples) to determine in-situ moisture contents, Atterberg limits, and in some cases grain size distributions. The results of this testing helped to identify the soil characteristics and define the stratigraphy. A summary of the disturbed sample results is in Attachment I-4.

I.3.3.2. Undisturbed testing was requested on 79 of the 81 undisturbed samples. The majority of the laboratory testing performed was done to determine the shear strengths of the soils. The shear strength tests included isotropically consolidated-undrained triaxial compression tests with pore-water pressure measurements (R-Bar), direct shear tests (DS), and unconsolidated-undrained (Q tests). Residual direct shear tests were also run to determine the effective residual shear strength of the soil. In addition, consolidation tests were performed on the samples. Other testing performed on undisturbed samples included: moisture content, unit weight, specific gravity, Atterberg limits, and grain size distributions. These tests helped identify the soil characteristics and define stratigraphy. Table I - 2 shows the undisturbed testing request for Phases 2, 3, and 4 while Table I - 3 shows testing completed on each soil unit. The laboratory test results are presented in Attachment I-4.

Table I - 2: Summary of Undisturbed Testing By Phase

Test	Phase 2	Phase 3	Phase 4	Total
DS	13	4	0	17
Residual	12	0	0	12
UU, Undisturbed	12	12	24	48
R-bar, Undisturbed	28	12	25	65
Consolidation	10	5	19	34
Constant Rate of Strain Consolidation	0	7	2	9
Atterberg Limits	30	15	26	71
Spec. Gravity	5	0	26	31
Hydro & Sieve	10	13	26	49
Sample Extrusion	38	15	26	79

Table I - 3: Summary of Undisturbed Testing by Formation

	Test								
	DS	Residual	UU, Undisturbed	R-bar, Undisturbed	Consolidation	Constant Rate of Strain Consolidation	Atterberg Limits	Spec. Gravity	Hydro & Sieve
Alluvium	0	2	2	7	2	2	5	1	3
Sherack	0	3	2	7	1	0	6	1	0
PL Sherack	0	0	0	0	0	0	0	0	0
Poplar River	0	0	0	1	0	0	1	0	0
PR - Harwood	0	0	2	2	1	0	2	0	2
PR - WF	1	0	2	2	1	0	2	0	2
OX Brenna	0	1	7	9	6	0	8	4	6
Brenna	10	3	16	18	11	3	22	9	15
B/A Trans	1	1	1	2	2	0	2	0	1
Argusville	5	2	14	13	9	4	18	12	15
Till	0	0	2	4	1	0	5	4	5
TOTAL	**17**	**12**	**48**	**65**	**34**	**9**	**71**	**31**	**49**

I.3.4. Selection of Design Parameters

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 can not 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 then 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.

I.3.4.3. 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 which would result 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.

I.3.4.4. The test results from the Fargo-Moorhead samples were entered into the St. Paul District's shear strength calculation spreadsheet. Since the Lake Agassiz soil deposits generally do not vary much, test results from other projects were also all incorporated into the spreadsheet. The other projects included three other MVP projects and consisting of: 1) Fargo Section 205 for the VA Hospital and Ridgewood Area; 2) the Sheyenne River and West Fargo Diversion project; and 3) Oakport Section 205. Data from the City of Fargo Southside Flood Control was also incorporated. It was found that the data from the different projects compared favorably to each other.

I.3.4.5. The shear strength parameters were selected using the 1/3: 2/3 rule, meaning that approximately 1/3 of the data points fell below the failure envelope and 2/3 of the data plotted above it. In the case of the Oxidized Brenna, Brenna, and Argusville formations, a curvilinear shear strength envelope was developed for the effective stress analysis of the diversion channel excavated slope. The curvilinear envelope is one standard deviation less than the most likely value. The most likely value (MLV) was determined by estimating lines that represented the highest and lowest conceivable values for shear strength. It was assumed that there were six standard deviations between the highest and lowest conceivable values, with the MLV envelope being located three standard deviations from either one. The selection of unit weights was based on the average value of the laboratory test results. The selected design parameters can be found in Table I - 4 and the data points for the curvilinear shear strength parameters in Table I - 5. In addition, the Mohr-Coulomb and curvilinear shear strength plots and undrained strength versus elevation plots are presented in Attachment I-5.

Table I - 4: Summary of Selected Soil Parameters

Formation	Unit Weight (1) γ_{sat} (pcf)	Shear Strength Parameters				
		Effective Stress (2)		Total Stress, c (psf)		Residual
		ϕ'	c' (psf)	Peak (3)	Ultimate (4)	$\phi'_{residual}$
Alluvium (5)	119	31	0	assume values of Sherack		20
Sherack	118	28	0	1400	900	13.0
Plastic Laminated Sherack	112	19	0	1150	N/D	6.8
Poplar River - West Fargo	123	34	0	1900	1900	25
Poplar River - Harwood	116	26	0	1450	1200	assume values of West Fargo
Poplar River, All (6)	119	assume values of Harwood		assume values of Harwood		assume values of West Fargo
Oxidized Brenna (7)	111	19	0	1000	900	5.5
Brenna (7)	104	13	0	650	525	9.0
Argusville (7)	106	15	0	825	600	10.5
Till (8)	122	31	0	1900	1900	N/A
Sand (9)	125	32	0	N/A		N/A
Riprap(9)	125	30	0	N/A		N/A

Notes:

(1) The unit weights are taken as the average value of all laboratory test results.

(2) The effective stress parameters are based on the R-Bar triaxial and direct shear tests. The failure criterion is defined as ultimate deviator stress which equates to the deviator stress at 15% or 20% axial strain.

(3) The peak total stress parameters are based on unconsolidated-undrained triaxial shear tests with the failure criterion defined at peak deviator stress. The peak undrained shear strength parameters were used for the end-of-levee-construction condition.

(4) The ultimate total stress parameters are based on unconsolidated-undrained triaxial shear tests with the failure criterion defined at ultimate deviator stress which equates to the deviator stress at 15% axial strain. The ultimate undrained shear strength parameters were used of the end-of-excavation condition when analyzing the diversion channel excavated slopes.

(5) Alluvium undrained shear strength parameters are assumed to be that of Sherack.

(6) Poplar River formation parameters are assumed to be that of the Harwood member.

(7) For the Oxidized Brenna, Brenna, and Argusville formations, a curvilinear shear strength envelope was developed for the effective stress analysis of the diversion channel excavated slope. The curvilinear envelop is one standard deviation less than the most likely value.

(8) Assumed values based on literature review.

(9) Assumed values based judgment.

Table I - 5: Summary of Curvilinear Shear Strength Envelope Points

Oxidized Brenna		Brenna		Argusville	
Effective Normal Stress σ' (psf)	Effective Shear Stress τ' (psf)	Effective Normal Stress σ' (psf)	Effective Shear Stress τ' (psf)	Effective Normal Stress σ' (psf)	Effective Shear Stress τ' (psf)
0	25	0	50	0	50
200	113	200	120	200	127
1000	420	1000	333	1000	413
2000	760	2000	540	2000	653
3000	933	3000	673	3000	893
4000	1073	4000	807	4000	1093
7000	1493	6000	1033	6000	1460

I.4.0. GEOTECHNICAL DESIGN OF ALTERNATIVE

I.4.0.1. The geotechnical design of the alternatives was completed in phases that followed the direction of the feasibility study. As mentioned earlier, the “Credit to Existing Levees” analysis was completed during Phase 1 and is documented in Appendix H. During Phase 2 geotechnical analyses of the most promising alternatives was completed. The alternatives were the In-Town Levee option and the diversion channel options. One major task during Phase 2 was to collect and interpret subsurface information and laboratory test data – the basis for the design analyses. For the In-Town Levee alternative, the minimum setback distances had to be determined. In addition, the MN Diversion channel was analyzed to determine what side slopes would be appropriate and how deep the channel could be excavated. The major geotechnical task for Phase 3 was completing the evaluation of the slope stability for the MN and ND Diversion alternatives. Additional soil exploration was also completed along the MN and ND Diversion channel alignments during Phase 3. For Phase 4, the major task was to reanalyze the ND Diversion channel incorporating the changes that were determined during the Phase 4 hydraulic analysis. The analyses completed for the alternatives during Phase 2, 3, and 4 are presented in the following respective sections.

I.5.0. IN-TOWN LEVEE ALTERNATIVE

I.5.0.1. The In-Town Levee alternative was evaluated in detail during Phase 2. The evaluation was completed to determine the required setback distance for levees such that stability of the natural banks and levees were adequate. At the end of Phase 2, it was recommended not to evaluate the In-Town Levee alternative due to the limitation in the top elevation of the levee that could be tied into high ground, high residual risks due to maximum height constraint, and the requirement to remove a large number of structures. Therefore, no refinement to the In-Town Levee alternative was completed during Phase 3. The following section describes the In-Town Levee Alternative and the analyses completed during Phase 2.

I.5.1. Features

I.5.1.1. Levees

I.5.1.1.1. Levees have been used as the primary feature for many of the projects designed and constructed by the St. Paul District in the Red River Valley. The levees are constructed of clay that is readily available within the valley. The typical configuration of the clay levees includes a 10-foot wide top and 1 vertical on 3 horizontal side slopes. With the foundation being composed of various clay formations, minimal foundation preparation is required. Typically, stripping of the topsoil and clearing and grubbing areas within the footprint of the levees is all that is required. Special seepage control measures are not typically required due to the impermeable nature of the foundation and levee. The only seepage control measure generally required is an inspection trench that is excavated 6 feet below the ground surface along the centerline. This is done to investigate the foundation conditions and intercept any unsuitable materials near the surface. The construction of the levees is straightforward and major constructability issues are rarely encountered.

I.5.1.2. Floodwalls

I.5.1.2.1. Floodwalls have also been incorporated into St. Paul District projects and are generally inverted T-walls. They are used when project constraints limit the location and size of the project footprint. Typically it is where residential homes and other buildings are required to stay in place. The construction cost of floodwalls is generally higher than that of levees, but the real estate cost can be less. Typically, the St. Paul District compares the cost of levees versus floodwalls and makes a determination as to which is the most cost effective solution. In the case of the FMMFS, it was decided that during Phase 2, only levees would be considered for design and cost purposes. It was understood that refinement of this assumption would be required if the In-Town Levee alternative proved to be the best implementable plan.

I.5.1.2.2. For floodwalls, there generally are not many special geotechnical considerations. The floodwall foundations are constructed and bear directly on the impervious foundation to minimize seepage. The depth of the foundation is below the frost line, approximately 6 to 7 feet below the ground surface. Piling is normally not required to support the floodwall. Sheetpile is typically only included on closure structure foundations.

I.5.1.3. Pump Stations and Drainage Structures

I.5.1.3.1. In addition to the levees and floodwalls, pump stations and drainage structures are required. The foundation elevations for these structures are normally deep because the storm sewer outlets at the river are deep because of the flat topography. Deep foundations allow drainage structures to be designed and constructed to bear on the natural clay deposits without piles or foundation treatments.

I.5.2. Design Methodology

I.5.2.1. Background

I.5.2.1.1. Experience and observation within the Red River Basin have shown that the riverbanks are unstable or only marginally stable in many areas. This is mainly the result of having a weak soil formation (Brenna) at depth. Continued erosion of the bank and lower than normal water elevations in the river also lends the natural banks to becoming unstable. Failure of the natural bank has been repeatedly observed during low water conditions, when the stabilizing force of the water on the bank is the lowest, even without an additional load being placed. Construction activities which place loads on the natural banks may also induce slope failures in areas that are unstable or only marginally stable. The fact that the riverbanks are unstable or only marginally stable leads to a critical geotechnical problem when trying to design and construct a flood barrier (i.e. levee or floodwall).

I.5.2.1.2. The flood barrier (levee, wall, or combination of both) represents a large capital investment by the local sponsors and the federal government. The flood barrier needs to perform its intended function (keep water out of the cities) over the life of the project, under both undrained and drained conditions. Foundation movements of the flood barrier resulting from the natural slope failing and sliding towards the river have the potential to render the flood barrier ineffective. Given the overall cost of the flood barrier, the importance of its intended function, and the potential to impact other structures by initiating foundation movements, a proper analysis and design is required that results in construction of a flood barrier in a stable and reliable zone.

I.5.2.1.3. The slope stability criteria and guide, EM 1110-2-1913, "Design and Construction of Levees," (Reference I.13.5) does not cover the aspects of an effective stress analysis analyzing using low water conditions to establish a stable and reliable location of a flood barrier. The St. Paul District, through discussions with HQUSACE, established a design methodology for the Grand Forks, North Dakota and the East Grand Forks, Minnesota flood control projects in the late 1990s (Reference I.13.9). This design methodology has been used on projects within the Red River Basin following the Grand Forks/East Grand Forks projects and is the basis of the geotechnical analyses completed for the FMMFS In-Town Levee alternative.

I.5.2.2. Method of Analysis

I.5.2.2.1. The geotechnical analysis completed for the FMMFS requires that slope stability analyses be completed to determine appropriate setback distances for the In-Town Levee alignments. This analysis is intended to determine the location of the levee such that it is outside a zone that is unstable, marginally stable, or may become unstable. Like traditional stability analyses, a computer program was implemented to calculate the factors of safety against slope stability. The St. Paul District elected to use Slope/W 2007 from Geo-Slope International LTD.

I.5.2.2.2. The St. Paul District's experience in analyzing setback distances for projects within the Red River Basin indicates that non-circular type failures are the most critical. The "Entry and Exit" slip surface option in Slope/W was utilized along with the "optimize" feature. The "Entry and Exit" search routine requires that the ranges for the entry and exit points of potential circular slip surfaces be indicated in the model. The circular surfaces along which the factor of safety (FS) is computed are determined by the entry and exit points, as well as a specified number of radius increments. The search yields a critical slip surface with the lowest factor of safety. With the optimization feature enabled, Slope/W proceeds to divide the critical slip surface into multiple segments and adjusts the location of these segments for the number of iterations specified, while calculating the FS for each iteration. Finally, the most critical "optimized" slip surface, now non-circular in shape, is reported. All stability analyses were completed using the Spencer's Method. In addition, an independent check of the Slope/W results was completed using UTexas4.

I.5.2.2.3. A range of entry points for the slip surface search was placed along the levee, from the wetside toe to the dryside toe. This allows a search to be conducted along the entire footprint of the levee. 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 and natural bank with respect to sliding down towards the river. The slope stability analysis required that minimum FS be obtained within the footprint of the levee to ensure that the levee could be constructed in a location that remained stable both during construction and also long term. In the case of Fargo-Moorhead, the most critical slip surfaces initiated at the wetside toe of the levee. If the entry points were extended towards the river, lower factors of safety would be determined. Shear surfaces riverward of the wet-side toe with lower factors of safety was deemed acceptable for the required level of effort for the Phase 2 analysis and design.

I.5.2.3. <u>Long-Term (Drained/Effective Stress) Design Condition</u>

I.5.2.3.1. The long-term, low water, drained/effective stress design condition has been in most instances, the critical design condition in determining setback distances. The long-term design condition represents a drained slope stability case where the changes in water table and lowering of the river water surface elevation, occur gradually over periods of time which allow for dissipation of excess pore pressures. Therefore, effective stress shear strength parameters are used for all of the soil units in the slope stability analyses. Due to the fact that many of the soil formations exhibit a brittle stress-strain behavior and slope movements are progressive failures, the ultimate (post-peak) shear strength parameters are selected for design.

I.5.2.3.2. The long-term design condition also requires that the groundwater table and river elevation be defined in the model. The groundwater table is known to fluctuate seasonally. The groundwater table was conservatively set at ten feet below the ground surface above the secondary bank, which is similar to the groundwater conditions used for the analysis of the Fargo-Ridgewood project. A low river stage was used in the long-term analysis since this provides for a small stabilizing force at the toe of the slope.

I.5.2.4. <u>End-of-Construction (Undrained/Total Stress) Design Condition</u>

I.5.2.4.1. The end-of-construction, undrained, total stress design condition has generally not been the controlling design condition. Even so, this condition is analyzed as a check. The end-of-construction condition represents the short-term slope stability case where excess pore water pressures develop due to foundation loading (normally from embankment construction). A total stress analysis is assumed, where undrained shear strength parameters are used to characterize the soil formations. For undrained analyses, pore water pressures are assumed to be unknown in clays, so a piezometric line was not used. The hydrostatic pressure due to river loading corresponds with the low river stage used in the long-term design condition.

I.5.2.5. Sudden Drawdown Design Condition

I.5.2.5.1. The sudden drawdown design condition is not considered to be a critical design condition. The low permeability soils and the relatively slow rise/fall of the river should not result in development of a Sudden Drawdown Condition. While a lag between the river level and the adjacent groundwater level is likely as the flood waters recede, a rapid river drawdown to a normal level leaving the levee and river bank saturated is not physically possible.

I.5.3. Preliminary Geotechnical Analysis

I.5.3.1. The geotechnical analysis for the In-Town Levee alternative was started in Phase 2. The purpose of the analysis was to determine the setback distances from the centerline of the Red River for the In-Town Levee alternative. Given the schedule of the project, the alignment for the In-Town Levee alternative was required prior to finishing the collection and interpretation of the subsurface information (i.e. soil exploration and testing) in order to provide enough time to complete the layout of the project and cost estimate. Because of this, many assumptions had to be made in order to perform the preliminary geotechnical analyses. For these preliminary analyses, the stratigraphy of the Fargo-Moorhead area was generalized based on the findings of the St. Paul District's 2007 feasibility study for the Fargo Section 205 Project (Reference I.13.8). Also, the design parameters were selected based on data available at that time.

I.5.3.2. The preliminary geotechnical analyses were completed for two different flood event scenarios. The hydrologic and hydraulic (H&H) analyses had not been completed prior to starting the preliminary geotechnical analyses. Therefore, the first event was based on the top of levee profile for the 1-percent annual chance event plus 4 feet of freeboard as presented in the "North Side Flood Control Evaluation" report dated April 30, 2008, and prepared by Houston Engineering, Inc. for the City of Fargo (Reference I.13.0). During this time, the H&H analyses were completed for the 0.5-percent and 0.2-percent annual chance events and the top of levee profiles were determined. The levee profile developed by Houston Engineering was compared to the St. Paul District's 0.5-percent annual chance event and found to be similar. The top of levee profile the St. Paul District determined for the 0.2-percent annual chance event was found to increase the levee heights between 2 and 4 feet. Geotechnical analyses were completed for this second flood event scenario.

I.5.4. Revised Geotechnical Analysis

I.5.4.1. Typically, interpretation of the soil exploration program and testing results would be completed prior to starting any geotechnical analyses. In this case, due to the project schedule, geotechnical analyses had to be started prior to completion of exploration and testing. When time allowed, the stratigraphy was revised based on site specific data. Also, the additional test data was incorporated and the design parameters were reevaluated. Once this was completed, the initial slope stability models used in the preliminary geotechnical analyses were revised using the site specific stratigraphy and revised design parameters. The long-term and end-of-construction conditions were analyzed for the 0.2-percent annual chance event. The more frequent flood event was not analyzed as the results of the preliminary analysis indicated similar setback distances for the two flood events.

I.5.5. Design Sections

I.5.5.1. There were many design sections selected for analysis of the In-Town Levee alternative. A total of 40 cross sections were analyzed, 20 sections a piece for the Fargo side and Moorhead side. These sections were developed in locations on the outside bend of the river or slightly downstream of the apex of the bend as these areas are the most susceptible to destabilizing toe erosion. The locations of the design sections are shown in Attachment I-2.

I.5.5.2. The ground surface profiles for the design sections were based on LiDAR data gathered in 2008. The LiDAR data was gathered as part of the Red River Basin Mapping Initiative. In addition, hydrographic surveys of the Red River were completed in October 2008 by St. Paul District's Channels and Harbors crew. From this survey, the channel bathymetry was developed and merged with the LiDAR data. The ground surface profile and channel bathymetry for the design sections are based on this merged data. All elevation data is presented in North American Vertical Datum 1988 (NAVD 88).

I.5.6. Slope Stability

I.5.6.1. Stratigraphy

I.5.6.1.1. The slope stability design sections were developed in Slope/W. The ground surface profile was generalized for the design cross sections. For the preliminary geotechnical analysis, the stratigraphy was also generalized based on the findings of the St. Paul District's 2007 feasibility study for the Fargo Section 205 Project (Reference I.13.8). An assumed stratigraphy as indicated below in Figure I - 1 was used. The stratigraphy of the design sections was revised to reflect the foundation conditions as indicated by the soil exploration program. The stratigraphy developed and used for the revised geotechnical analyses is presented in Attachment I-6.

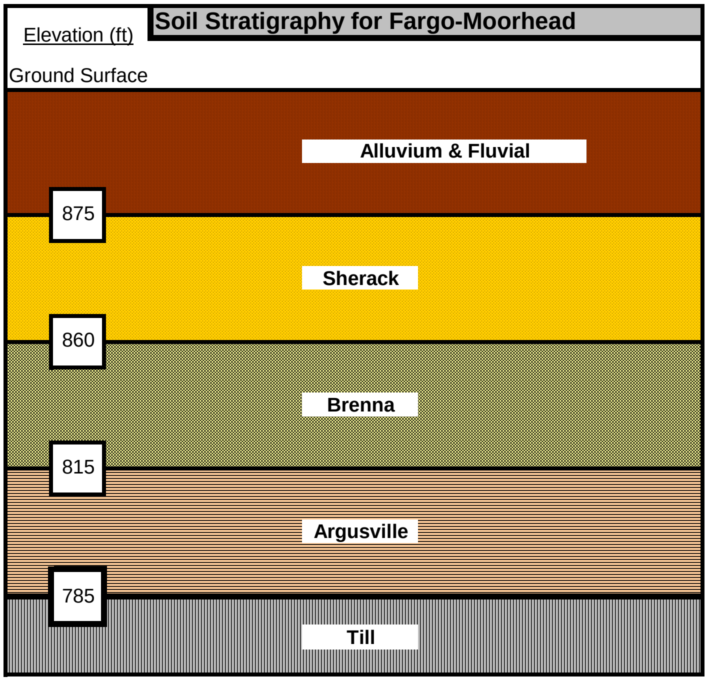

Figure I - 1: Assumed Stratigraphy for the Preliminary Geotechnical Analysis

I.5.6.2. Past Slides

I.5.6.2.1. There is evidence that earth movements (slides) have occurred within the Fargo-Moorhead metro area. Some of these slides have been initiated by human activities such as placing fill on the natural bank. One such occurrence has been documented in the area of the Veterans Administration Hospital in Fargo, ND in which a levee was constructed that caused a slide to form. In Moorhead, MN at Gooseberry Mound Park, there is physical evidence of a slide - a crack has formed in the pavement and there is differential vertical movement on either side of the crack.

I.5.6.2.2. Experience dictates that excess loading in areas of these slides, even if dormant, could cause existing slides to reactivate. For the revised geotechnical analysis, a residual shear strength zone, or failed soil zone, was assumed in the stability models to account for this. The residual shear strength zone was assumed to coincide with the location of the secondary bank slope and extend through the upper materials of alluvium and Sherack into and through the Brenna formation. The location of the residual shear strength zone coinciding with the secondary bank slope is a conservative assumption. It was assumed that no failure of the Argusville formation has occurred and that the formation was intact. The shear strength parameters of the failed soil were based on the results of the residual direct shear tests performed on samples of the respective formations.

I.5.6.3. Groundwater Table and River Elevation

I.5.6.3.1. The groundwater table was input into the stability model using the piezometric line feature in Slope/W. The groundwater table was assumed to be approximately 10 feet below the ground surface. At a location approximately 100 feet back from the start of the secondary bank, the groundwater table sloped linearly down to the assumed river surface elevation. For the preliminary analyses, the river surface elevation that was assumed to be 5 feet above the river bottom. This was based on the low river flow depth used for the of the St. Paul District's Fargo Section 205 project (Reference I.13.8).

I.5.6.3.2. For the revised analyses, a low river surface profile was determined using the Red River HEC-RAS model for a flow of 220 cubic feet per sec (CFS). The 220 CFS flow is the 50-percentile flow for the month of September. The month of September was chosen because the average flow is the lowest of any given month. This resulted in an increase to the river surface elevation between 2 and 11 feet from the originally assumed 5-foot depth used in the preliminary analyses.

I.5.6.4. Required Minimum Factors of Safety

I.5.6.4.1. The goal of the geotechnical analyses was to determine the required setback distance of the levee from the centerline of the Red River. The position of the centerline of the levee on the slope that produced the required factor of safety (FS) was used as the minimum setback distance from the river's edge. There were three minimum FS criteria for slope stability used when determining the minimum setback distance for the in-town levees, depending on existing conditions and loading conditions.

I.5.6.4.2. A minimum FS = 1.4 was required for the preliminary analyses of the effective stress (drained) condition. The preliminary analyses were based on generalized stratigraphy with all soil being intact, an assumed ground water surface, and a river depth of 5 feet.

I.5.6.4.3. The revised geotechnical analyses included both residual shear strength zones and intact soil formations. The shear strength parameters used for the failed soils were based on residual direct shear test results, which is the lowest expected value for the soil. In addition, the failed/intact interface was assumed to coincide with the secondary bank slope and also assumed to be a vertical line. This is a conservative assumption, as it creates the largest extents for the failed soils. Also, ultimate or "fully-softened" shear strength parameters were used for the intact soil formations. Based on this, a minimum FS = 1.2 was required for revised effective stress (drained) conditions with residual shear strengths incorporated into the model. This factor of safety is similar to that used in the design procedure established by the St. Paul District for the Grand Forks / East Grand Forks Flood Control Projects (Reference I.13.9) in which back analysis was used to calculate the residual shear strength parameters.

I.5.6.4.4. The end-of-construction (undrained/total stress) conditions were also analyzed and required a minimum FS of 1.3. The undrained analysis used only intact peak undrained shear strengths. Experience in the Red River Valley indicates that undrained conditions typically do not control the required setback distances. This held true for the preliminary geotechnical analyses, in which the FS determined for end-of-construction condition were greater than the effective stress cases. Following revision and reanalysis of the design sections, two of the forty sections were found to be controlled by the undrained condition. Therefore, in those two sections the setback distance was greater than what was required for the drained condition. A summary of the required FS is presented below in Table I - 6.

Table I - 6: Summary of Requirement Minimum Factors of Safety for Slope Stability

Design Condition	Required FS
Long-Term (Drained/Effective Stress)	1.4
Long-Term (Drained/Effective Stress) with Residual Soil	1.2
End-of-Construction (Undrained/Total Stress)	1.3

I.5.6.5. Results of the Geotechnical Analyses

I.5.6.5.1. The preliminary geotechnical analyses indicated that the required setback distances were similar for both the Houston Engineering 0.1-percent annual chance event plus 4 feet freeboard (St. Paul District's 0.5-percent annual chance event) and the 0.2-percent annual chance event. The required setback distances ranged from 345 feet to 555 feet.

I.5.6.5.2. Because the preliminary analyses indicated that the setback distances were similar for the two different scenarios, only the 0.2-percent annual chance event analyses were revised. The revised geotechnical setback distances varied from the preliminary setback distances. Sixty-five percent (26 out of 40) of the revised setback distances were within 50 feet (plus or minus) the preliminary setback distances. Approximately 28% (11 out of 40) of revised setback distance changed 50 to 100 feet (plus or minus). Only 8% (3 out of 40) of the revised setback distances changed more than 100 feet.

I.5.6.5.3. 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. A summary of the results of the preliminary analyses is presented in Attachment I-7 and the revised geotechnical analyses in Attachment I-8.

Table I - 7: Summary of In-Town Levee Alignment Setback Distances

Fargo, North Dakota					Moorhead, Minnesota				
	Setback Distance [ft]					Setback Distance [ft]			
	Preliminary Analysis		Revised Analysis			Preliminary Analysis		Revised Analysis	
Cross-Section	Houston 0.1% Annual Chance	USACE 0.2% Annual Chance	USACE 0.2% Annual Chance	Setback Used	Cross-Section	Houston 0.1% Annual Chance	USACE 0.2% Annual Chance	USACE 0.2% Annual Chance	Setback Used
FAR-01	520	520	415	520	MOOR-01	470	470	431	415
FAR-02	555	555	545	555	MOOR-02	435	435	442	430
FAR-03	480	480	443	480	MOOR-03	435	435	476	425
FAR-04	495	490	487	495	MOOR-04	445	445	458	450
FAR-05	510	510	454	500	MOOR-05	495	495	442	485
FAR-06	380	385	362	660	MOOR-06	400	415	336	400
FAR-07	345	355	345	380	MOOR-07	480	485	483	660
FAR-08	410	410	415	420	MOOR-08	455	465	462	455
FAR-09	435	450	360	N/A	MOOR-09	435	435	488	430
FAR-10	460	470	453	450	MOOR-10	460	470	498	470
FAR-11	370	365	263	520	MOOR-11	470	485	540	475
FAR-12	475	485	489	485	MOOR-12	415	440	472	570
FAR-13	340	380	516	340	MOOR-13	380	395	475	385
FAR-14	365	410	483	365	MOOR-14	405	420	466	415
FAR-15	410	425	448	420	MOOR-15	320	370	470	380
FAR-16	360	400	440	430	MOOR-16	385	410	471	405
FAR-17	N/D	385	405	505	MOOR-17	430	440	492	435
FAR-18	N/D	445	453	410	MOOR-18	500	515	502	530
FAR-19	N/D	500	458	470	MOOR-19	385	405	446	435
FAR-20	N/D	450	406	415	MOOR-20	425	N/D	N/D	495

I.5.7. Settlement

I.5.7.1. The levee heights for the In-Town Levee alignments ranged from a couple feet to approximately 12 feet in height. No settlement analysis was completed during Phase 2. Based on the St. Paul District's experience in the Red River Valley, expected settlement could be in the range from a few inches to approximately 1 foot. For previous projects, the levees have been constructed higher than the required H&H top of levee profile to account for anticipated settlement. This is typically referred to as overbuild. For the Phase 2 In-Town Levee layout and cost estimate, the overbuild was neglected considering it was a minor amount. It was understood that refinement of this assumption would be required if the In-Town Levee alternative is selected as an implementable plan.

I.5.8. Revetments

I.5.8.1. It is expected that the erosion of the primary bank will continue throughout the lifetime of the project. The erosion process decreases the stability of the bank. It is most prevalent on the outside bends in the river. An evaluation was done and eighteen locations were identified to be critical due to the tightness of the bend and the proximity of structures to the river. It was thought that riprap would likely be required in these critical locations in order to maintain stability over time. A rough estimate of riprap quantities was completed using a generalized cross section that was 3 feet wide at the top and had a 1V on 3H slope. The generalized cross section is shown below in Figure I - 2. It was determined that approximately 125,000 cubic yards could be placed and was included in the cost estimate for the In-Town Levee alternative.

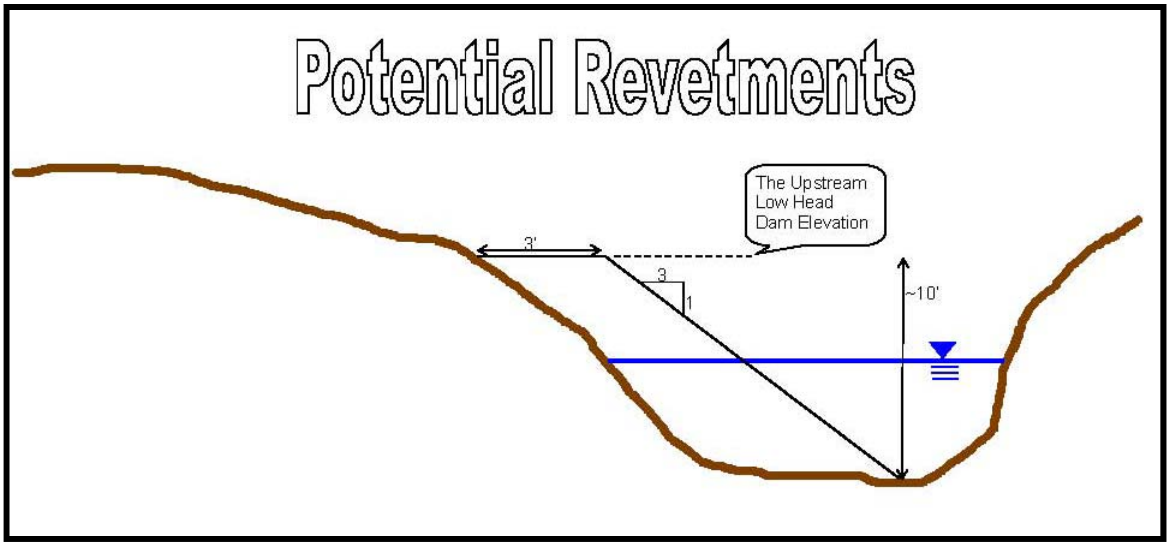

Figure I - 2: Conceptual Revetment Typical Section

I.5.9. Conclusions

I.5.9.1. The stability analyses followed the methodology that has been developed by the St. Paul District for projects within the Red River Basin. These analyses are considered conservative, but still appropriate to determine the costs associated with the In-Town Levee alternative. The level of the geotechnical analysis is comparable to the level of detail used to determine the cost estimates during Phase 2. It is acknowledged that further refinement of the models is required if the In-Town Levee alternative were to be carried forward at any point in the future.

I.5.9.2. The recommendation made at the end of Phase 2 was that no further evaluation of the In-Town Levee alternative would be completed. There were a number of reasons why the In-Town Levee alternative was not considered to be the best implementable plan, as outlined below. No further evaluation of the In-Town Levee alternative is expected at this time.

(1) The top elevation of the levees 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, and

(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.

I.6.0. DIVERSION CHANNEL ALTERNATIVES

I.6.0.1. The evaluation of the Diversion Channel alternatives began during Phase 2 in which preliminary analyses were completed for the MN Diversion channel. During Phase 3, the MN Diversion channel analyses were refined and a more robust evaluation completed. In addition, the ND Diversion channel alternative was evaluated in similar detail. For Phase 4, the ND Diversion alternative was reanalyzed incorporating the required hydraulic changes. The following section describes the evaluation completed for the Diversion Channel Alternatives.

I.6.1. Features

I.6.1.1. <u>Diversion Channel</u>

I.6.1.2. The major feature of the diversion channel alternatives, as the name implies, is an excavated channel. The diversion is used to intercept the Red River at flood events higher than the 20-percent annual chance event and divert the water around the Fargo-Moorhead Metro area, reducing the flood stage within the city limits. The depth of the channel and bottom width varies depending on the capacity of the diversion channel. The side slope of the channel is dependent on the depth of excavation and the strength of the foundation materials.

I.6.1.3. <u>Red River Control Structure</u>

I.6.1.4. A control structure is required on the Red River downstream of the diversion channel inlet for either diversion channel alternative. The purpose of this control structure is to divert the flood waters into the diversion channel and reduce the amount of water that remains in the natural channel during flood events.

I.6.1.5. <u>Inlet Weir</u>

I.6.1.5.1. For the MN Diversion, an inlet weir is required in the diversion channel near the Red River. The purpose of the inlet weir is to limit the flow into the diversion channel until a flood event greater than the 3.6-yr is reached on the Red River. The inlet weir consists of a sheetpile-rockfill weir.

I.6.1.5.2. For the ND Diversion channel, an inlet weir is required. The location of the inlet weir is west of the Wild Rice River and downstream of the Storage Area 1. The purpose is to limit flow into the diversion channel until a flood event greater than the 3.6-

yr is reached. The inlet weir for the ND Diversion channel is an ogee-type concrete spillway.

I.6.1.6. Outlet

I.6.1.6.1. The outlet of the diversion channel to the Red River is simply riprap placed on the excavated channel to reduce the potential for erosion for both the MN and ND Diversion.

I.6.1.7. Wild Rice River Control Structure

I.6.1.7.1. For the ND Diversion channel, there is a control structure required on the Wild Rice River. This control structure is similar in design and purpose of the Red River control structure.

I.6.1.8. Sheyenne River and Maple River Hydraulic Structures

I.6.1.8.1. For the ND Diversion channel, hydraulic structures are required at locations where the diversion channel crosses the Sheyenne and Maple Rivers. These hydraulic structures are similar in design and purpose at both crosses. At the crosses, an aqueduct will be used to pass the flow of water through the diversion channel at the same time the tributary rivers are flowing over top of the diversion channel.

I.6.1.9. Lower Rush and Rush River Drop Structures

I.6.1.9.1. For the ND Diversion, drop structures are required to divert the Lower Rush and Rush Rivers into the diversion channel. These drop structures are stepped, concrete spillways.

I.6.1.10. Tie-Back Levees

I.6.1.10.1. The tie-back levees are needed for either diversion channel alternative. The tie-back levees extend away from the inlet structure to high ground to prevent the flood water from getting into the protected area.

I.6.1.10.2. In the case of the MN Diversion channel alternative, the tie-back levee extends west from the control structure to high ground on the North Dakota side. The ND Diversion channel alternative requires two different tie-back levee alignments. The first alignment extends east from the control structure to high ground on the Minnesota side. The second tie-back levee alignment extends south from the diversion channel and western edge of Storage Area 1 to high ground on the North Dakota side. This tie-back levee contains the flows within the designated staging area.

I.6.1.11. Storage Area 1

I.6.1.11.1. For the ND Diversion alternative, a large storage area is required. The storage area encompasses 4,360 acres and is located on the north side of the ND diversion alternative between the Wild Rice River and the Sheyenne River. The storage area will be formed by constructing approxamtely 12 miles of embankments around the area. The purpose of the storage area is to store water in order to eliminate downstream flood level impacts.

I.6.1.12. The features described above, excluding the diversion channel, were analyzed and designed under an in-kind services contract through the local sponsors. There were four engineering firms that completed this in-kind work: Moore Engineering, Inc; Houston Engineering, Inc; Barr Engineering Company; and HDR, Inc. The work completed by these A/E firms is documented in the "Red River Diversion, Fargo-Moorhead Metro Flood Risk Management Project, Feasibility Study, Phase 3" report (Reference I.13.1). The report will be referred to as the "RRD Report". Appendix F of the RRD Report further explains the function and design of each of the features.

I.6.2. Geotechnical Design Process

I.6.2.1. As with the analysis conducted for the In-Town Levee alternative, the analyses completed for the Diversion Channel alternatives evolved throughout the feasibility study. A description of the evolution of the Diversion Channel alternative analyses is included below.

I.6.2.2. At the beginning of Phase 2, the hydraulic modeling of the MN Diversion Channel alignment was to be completed. Initially, the side slopes of the diversion channel were set at 1V on 7H except at locations of bridge crossings. At these locations, the channel side slopes were steepened up to 1V on 5H. This was provided as input to the A/E firms so the hydraulic modeling of the diversion channels could be started. The selection of the initial side slopes was based on judgment and would be verified and/or modified through geotechnical analyses.

I.6.2.3. The selection of the 1V on 7H side slopes was based on the experience of the St. Paul District in construction of previous diversion channels and most specifically the West Fargo / Horace Diversion (WFHD) project that the St. Paul District had constructed in 1992. The depth of the WFHD was 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 WFHD indicated that slopes as steep as 1V on 5H would be acceptable. The side slopes selected for the WFHD were 1V on 7H to allow the side slopes to be mowed with standard farming equipment. Erosion of the WFHD at the toe of the slopes has led to slope instability that had to be fixed. The design and performance of the WFHD was used as the basis of the initially selected side slopes.

I.6.2.4. The initial hydraulic modeling effort for the MN Diversion channel by Moore Engineering indicated that for a given channel capacity, the quantity of excavated material decreased with increasing depth. This meant that for a given channel capacity, the deeper the excavated channel, the more cost effective it was. The initial recommendation was therefore to use these deep excavated channels. Since the St. Paul District had not expected that deep excavations would be recommended, four sections were analyzed for seepage and stability to verify that these deep excavations would be stable and meet the required design criteria. The initial geotechnical analyses found that the deep excavations would not meet the design criteria. Based on the initial geotechnical findings, the maximum excavation depth was limited to approximately 30 feet below the ground surface.

I.6.2.5. Using the maximum excavation depth restriction as the bottom of the channel, Moore Engineering revised the hydraulic models and optimized the MN Diversion channel width for the 3 different channel capacities. With the revised hydraulic modeling completed, the geotechnical analyses were revised for the MN Diversion channel. The results indicated that modifications to the channel slope were required for approximately 2/3 of the length of the diversion channel. The modification of the slope included the addition of a bench and flattening the bench slope to 1V on 10H.

I.6.2.6. During Phase 2, the majority of the hydraulic modeling effort was placed on the MN Diversion channel. The hydraulic modeling effort of the ND Diversion channel was kept to a minimum, as at the time the likelihood that the ND Diversion channel alternative would be selected was small. Therefore, no geotechnical effort was put forth on the ND Diversion channel. It was acknowledged that geotechnical evaluation would be required of the ND Diversion channel if this alternative was to be refined in Phase 3.

I.6.2.7. At the end of Phase 2 and the start of Phase 3, it was known that the In-Town Levee alternative was not likely to be an implementable plan. The results of Phase 2 indicated that the MN Diversion channel alternative would be the most cost effective alternative, but it was still unknown what channel capacity would provide the largest benefit/cost ratio. The local sponsors were also interested in the ND Diversion channel alternative. Therefore it was decided that for Phase 3, both the MN Diversion channel and ND Diversion channels be refined and evaluated in similar detail. From a geotechnical perspective, this meant reevaluating the stability of the MN Diversion channel and evaluating the stability of the ND Diversion channel. It also meant evaluating all the associated structures with the diversion channels.

I.6.2.8. During Phase 4, the MN Diversion alternative did not change, but changes were made to the ND Diversion alternative. The changes to the ND Diversion alternative involved raising the invert and widening the channel bottom. These changes required that the ND Diversion channel be reevaluated for seepage and slope stability along with reevaluating all the associated structures.

I.6.2.9. During both Phase 3 and Phase 4, there was an iterative process for the design of the diversion channel. Initially, the hydraulic analysis was completed based on an invert raise to determine the required cross sectional area of the channel. The seepage and stability analyses were then completed based on the preliminary hydraulic analysis results. The geotechnical analysis typically resulted in changes to the configuration of the channel to include benches, and in some instances the channel invert was raised. The geotechnical requirements were then incorporated into the hydraulic model and reanalyzed.

I.6.2.10. The seepage and slope stability analyses for the diversion channel were completed by the St. Paul District and are detailed below in the appendix. The geotechnical analyses for the other various components of the diversion channel were completed by Barr Engineering Company and are detailed in the RRD Report (Reference I.13.1).

I.6.3. Seepage

I.6.3.1. The steady-state-seepage modeling of the diversion channels was completed using Seep/W 2007 from GeoSlope International Ltd. The reason the seepage analysis of the diversion channel was completed was to couple the seepage results with the slope stability analyses. The coupled results would provide more realistic pore pressures that could be used in the effective stress stability analyses instead of a piezometric line.

I.6.3.2. Initially in Phase 2, the seepage models represented the "full cross-section" of the channel. However, since the sections were essentially symmetric about the channel centerline, "half-space" models were used to reduce the required work and computation time. The results obtained using the "half-space" models were found to be the same as those models using the full section. During Phase 3, there were three sections along the MN Diversion channel in which the stratigraphy changed dramatically perpendicular to the centerline of the channel. In these instances, the "full cross-section" was used and the stratigraphy was varied as needed throughout the section. The "full cross sections were also used for the ND Diversion channel as the channel bottom width was initially only 100 feet wide and this could have constrained the failure surface.

I.6.3.3. Seep/W has the capability of modeling both saturated and unsaturated materials and this feature was used for the seepage analysis. Materials that were expected to be above the piezometer line (groundwater table) were modeling using the "Saturated / Unsaturated" material model type. This requires that a hydraulic conductivity function be keyed in. For the FMMFS, Seep/W's built-in tools were used to estimate the volumetric water content functions which in turn were used to estimate the hydraulic conductivity functions. The input for the materials expected to stay saturated, or below the groundwater table, was less complicated. It only required the key in of the saturated permeability (K_{sat} in Seep/W), volumetric water content, and coefficient of volumetric compressibility, M_v.

I.6.3.4. The required parameters needed for the seepage analysis are summarized below in Table I - 8. The parameters were based on tests results in as much as practical. For materials in which testing was not available, the parameters were estimated. In the case of the saturated permeability of the formations, this was based on the typical ranges of permeability for different types of soils that are published in many geotechnical engineering books. Many of the foundation materials are massive and very likely homogenous so the ratio of the vertical (k_y) to horizontal (k_x) permeability was set to unity for simplification. The permeability of the Till formation was varied until a phreatic surface that seemed reasonable was obtained. It was found that increasing the permeability of the Till formation by 2 orders of magnitude above the Brenna and Argusville formations and using a k_y/k_x ratio of 0.25 resulted in a reasonable looking phreatic surface.

Table I - 8: Summary of Selected Permeability Parameters

Material	Material Model Type (1)	Sample Material (2)	Vertical Permeability		Horizontal Permeability		Volumetric Water Content (3)	M_v (4)	Residual Water Content (5)
			k_y (cm/sec)	k_y (ft/day)	k_y/k_x ratio	k_x (ft/day)	(ft3/ft3)	(1/psf)	(ft3/ft3)
Alluvium	Sat / Unsaturated	Silty Clay	1.0E-06	2.8E-03	1	2.8E-03	0.5	9.0E-06	0.050
Sherack	Sat / Unsaturated	Silty Clay	1.0E-06	2.8E-03	1	2.8E-03	0.5	9.0E-06	0.050
PL Sherack	Sat / Unsaturated	Silty Clay	1.0E-04	2.8E-01	1	2.8E-01	0.5	9.0E-06	0.050
West Fargo	Sat / Unsaturated	Silt	1.0E-04	2.8E-01	1	2.8E-01	0.4	3.0E-06	0.040
Harwood	Sat / Unsaturated	Silt	1.0E-05	2.8E-02	1	2.8E-02	0.5	9.0E-06	0.050
OX Brenna	Sat / Unsaturated	Silty Clay	5.0E-07	1.4E-03	1	1.4E-03	0.55	1.0E-05	0.055
Brenna	Saturated Only	N/A	1.0E-07	2.8E-04	1	2.8E-04	0.63	3.0E-05	0.063
Argusville	Saturated Only	N/A	1.0E-07	2.8E-04	1	2.8E-04	0.6	3.0E-05	0.060
Silts	Saturated Only	N/A	1.0E-06	2.8E-03	1	2.8E-03	0.4	3.0E-06	0.040
Silty Sands	Saturated Only	N/A	1.0E-04	2.8E-01	1	2.8E-01	0.4	3.0E-06	0.040
Till	Saturated Only	N/A	5.0E-06	1.4E-02	0.25	5.7E-02	0.45	3.0E-05	0.045
Sand	Sat / Unsaturated	Fine Sand	1.0E-02	2.8E+01	1	2.8E+01	0.4	3.0E-05	0.040

Notes:
(1) Indicates how the material was model in Seep/W. If material above expected groundwater table, Sat/Unsaturaded. If below the groundwater table, Saturated Only.
(2) Indicates what sample material type was used when estimating the volumetric water content function in Seep/W.
(3) Volumetric Water Content Based on Porosity taken from testing except for PL Sherack, Sand, and Till which are estimated values.
(4) M_v for Alluvium, Sherack, OX Brenna, and Brenna based on consolidation data. All other materials estimated.
(5) The residual water conent was estimated to be 10% of the saturated water content.

I.6.3.5. The boundary conditions used in the "half-space" and "full cross section" seepage models consisted of total head, potential seepage, and no flow boundaries. The total head boundary conditions were used to represent the ground water elevation, which was assumed to be 5 feet below the ground surface. The total head boundary conditions were placed along the vertical side(s) of the model, opposite the centerline of the channel. Potential seepage boundary conditions were placed within the diversion channel to allow Seep/W to calculate the location where water would exit the slope. No-flow boundary conditions were placed on the bottom of the model and at the centerline of the diversion channel for the "half-space" models.

I.6.3.6. A sensitivity analysis was performed during Phase 2 in order to determine the appropriate distance between the total head boundary condition and the channel. The sensitivity analyses varied the distance of the total head boundary conditions from 1000 feet to infinity from the centerline of the channel. It was found that increasing the distance the total head boundary conditions were from the centerline of the channel decreased the exit gradients along the channel bottom, the quantity of flow into the channel, and also the phreatic surface near the channel. From these results and discussion amongst the St. Paul District geotechnical engineers, it was thought that that the diversion channel excavation would influence the groundwater table near the excavation. It was decided that a distance of 2,000 feet (approximately ½ mile) from the centerline of the diversion channel would be used in the modeling as it seemed to be a reasonable distance that the diversion channel would influence the groundwater table. If the diversion influences the groundwater table at distances greater than 2,000 feet, then the exit gradients, quantity of flow, and phreatic surface would be less that those computed using the 2,000 foot distance.

I.6.3.7. During Phase 4, a sensitivity analysis was conducted to see what affect the k_y:k_x ratio would have on the piezometric line and the slope stability results. The three sections for each diversion alternative with the lowest FSs were checked. The sensitivity analysis compared the results for k_y:k_x ratios of 1/5 and 1/10 against the original analysis using a k-ratio of unity. The results indicated that the slope stability FSs were slightly reduced, generally by less than 1%, when a k-ratio of 1/5 was used instead of a k-ratio of unity. Even with the reduction, the minimum calculated FS remained at or above the required FS. When a k-ratio of 1/10 was used, the FSs increased above those found using a k_y:k_x ratio of 1.

I.6.4. Slope Stability

I.6.4.1. Conditions

I.6.4.1.1. The slope stability analyses for the diversion channel were completed using Slope/W 2007. The use of Slope/W allowed the pore pressures determined by Seep/W to be used in the stability analyses rather than a piezometric line. Following the guidance indicated in EM 1110-2-1902, Slope Stability (Reference I.13.4), the diversion channel excavated slopes were analyzed for the end-of-construction case and the long-term case. The end-of-construction case is the condition in which the excavated slope is undrained and total stress shear strength parameters are used in the slope stability analysis without any pore pressures. The long-term case is the drained condition of the excavated slope after the pore pressures have reached equilibrium. In this case, steady-state-seepage analyses were used in estimating the long-term pore water pressures. The long-term condition uses effective stress shear strength parameters.

I.6.4.1.2. For Phase 2, the global (failure surface encompassing the entire excavated slope) stability of the MN Diversion channel slope was evaluated based on the long-term (drained) condition and the undrained (short-term or end-of-construction) condition.

I.6.4.1.3. During Phase 3, the stability of the diversion channels were further evaluated. As in Phase 2, the long-term (drained) and undrained (short-term) conditions evaluating the global stability of the excavated channel were deemed necessary. In addition, localized, drained failure of the slope was also determined to evaluate the shallow sloughing failures that could lead to maintenance issues. The localized failures were evaluated on the lower portion of the slope and also the upper portion of the slope. This same methodology was used when reanalyzing the ND Diversion alternative during Phase 4.

I.6.4.1.4. The undrained and long-term conditions provide a range of expected performance of the excavated slope and it was reasoned that if the stability analyses resulted in meeting the required minimum factors of safety for both conditions, the channel configuration and selected slopes were adequately stable. Completing these two analyses was deemed adequate for the design of the diversion channel excavation but consideration was also given to other analyses that could be completed.

I.6.4.1.5. The classic rapid-draw-down analysis in which the water that has been on a slope for a considerable duration, allowing steady-state-seepage condition to be establish, and then suddenly drop was deemed not applicable for the excavated slope. The increase/decrease of the flood water on the slope was short enough that the pore pressures at and near the face of the excavated slope would not be dramatically changed.

I.6.4.1.6. In addition, a staged excavation type analysis was contemplated. This type analysis would require the use of a finite-element/finite-difference procedure to model the changes in stress and pore pressures throughout the excavation process. The procedure would use effective stress shear strength parameters, as the pore pressures are calculated. The excavation would cause the stresses and pore pressures adjacent to the slope to decrease due to the unloading of the materials. The most drastic changes in pore pressure would occur at the face of the slope, and decrease with distance from the slope. In the vicinity of the spoil pile, there would be an increase in stress and pore pressures. This type of analysis was considered to be more complex than required for the feasibility study and not completed for the feasibility study for a number of reasons, listed below. It is recommended that this type of analysis be completed during the planning phase as it may reveal additional information.

(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 due to the spoil pile.

(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 considerably flat, being 1V on 7H.

I.6.4.2. Target Factors of Safety

I.6.4.2.1. The Corps Engineering Manual (EM) 1110-2-1902, Slope Stability (Reference I.13.4) identifies the minimum required factors of safety (FS) for dams while EM 1110-2-1913, Design and Construction of Levees (Reference I.13.5) identifies the minimum FS for levees. Neither of these manuals specifically identifies the minimum required FS for excavated slopes associated with a diversion channel. EM 1110-2-1902 recommends that for slopes other than dams the minimum FS be selected based on uncertainty of the shear strength parameters and the consequences of failure.

I.6.4.2.2. The St. Paul District assessed what target factors of safety should be used for evaluation of the stability of the diversion channels. The St. Paul District’s experience in the Red River Valley indicates that the long-term, drained condition typically controls the design of a project due to the low drained shear strength of the Brenna formation. Once a failure has occurred, the drained shear strength is further reduced, which creates a situation that is often times difficult and expensive to repair. Therefore, the St. Paul District selected the target FSs of 1.4 and 1.3 for the long-term and undrained conditions, respectively, to coincide with the required minimum FS for levee stability. The reasoning for selecting these target FSs was to reduce the potential that the diversion channel slopes would fail and result in the implementation of a difficult and expensive fix.

I.6.4.2.3. The St. Paul District also considered the possibility of localized failures. These failures are associated with smaller potential failure surfaces which do not encompass the entire diversion channel slope. The potential failure along these surfaces could lead to difficulty in maintenance and mowing, but would not affect the overall stability of the slope. Therefore a target FS of 1.2 was selected when analyzing the localized, drained (long-term) failures.

I.6.4.3. Critical Slip Surface Search Procedure

I.6.4.3.1. The St. Paul District's experience in riverbank slope stability in the Red River Basin indicates that non-circular type failures are the most critical, resulting in lower FS than circular failure surfaces. This is due to the fact that the failure surfaces extend over a considerable length (in effect, have long neutral blocks) through a weak soil layer that is at depth. The configuration of the diversion channel alternatives is approaching that of the riverbanks and a similar situation occurs in which the failure surfaces have long neutral blocks through a weak soil layer. In these situations, a circular failure surface does not adequately represent the most critical failure mechanism. In order to capture these critical non-circular type failures the "optimize" feature in Slope/W was enabled, as was the case when analyzing the levee setback distances for the In-Town Levee alternative. All stability analyses were completed using the Spencer's Method.

I.6.4.3.2. Slope/W has a number of ways in which to search for the critical failure surface. The "Entry and Exit" slip surface option was the search mechanism used. The "Wedge" slip surface option was also used for the global, long-term condition to verify that the "Entry and Exit" search found the critical failure surface.

I.6.4.3.3. For all searches and conditions, the range of entry points of the failure surface search were based on set criteria. In the case of the global, long-term condition, the "entry" of the failure surface search extents were set 50 feet from the top of slope to ¼ of the excavation depth below the existing ground surface. The "exit" extents were from 5 feet above the bottom of the channel to a point near the centerline of the diversion. The global, undrained condition was placed from the top of the slope and extending 250 feet away from the slope. The exit extents were the same as the global, long-term condition.

I.6.4.3.4. During Phase 2, there was no effort made in maintaining consistency of the extents for the localized drain slope condition but for Phase 3 and Phase 4 the effort was made. The localized lower drained slope condition extents for the "entry" portion were positioned on top of the bench to 5 feet above the bottom of the diversion channel. The "exit" extents were those used for the global, long-term condition. The "entry" extents for the localized upper drained slope conditions extended 50 feet from the top of slope to 5 feet above the bench. The "exit" extents started 5 feet above the bench and extended along the bench. These "entry" and "exit" extents are illustrated in the figures below.

I.6.4.3.5. During Phase 3 and Phase 4, the critical circular failure surface location was checked. If the failure surface started or exited at the limits of the extents, the "Entry and Exit" search was refined by moving the extents such that the critical failure surface started and exited within the limits and not at the edges.

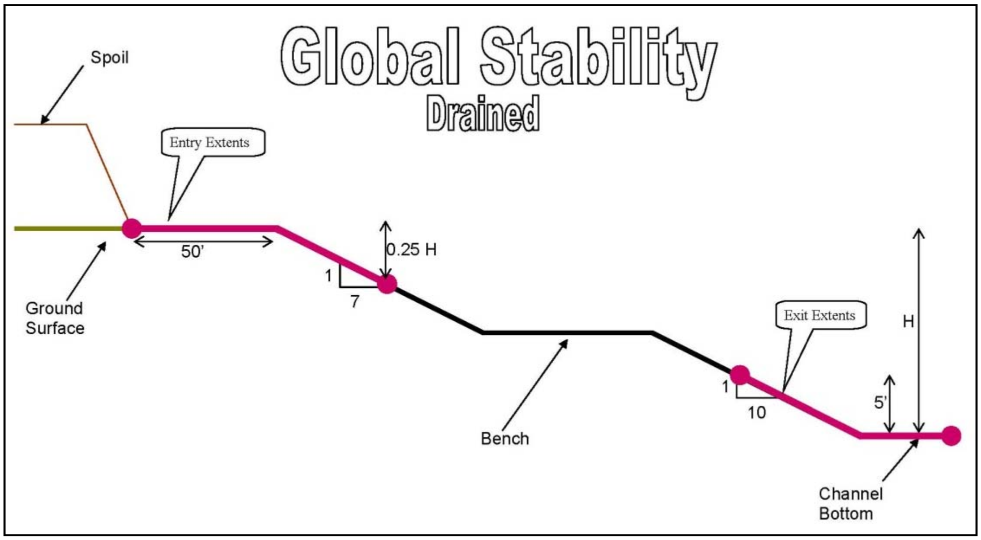

Figure I - 3: Global, Long-Term Condition Search Extents

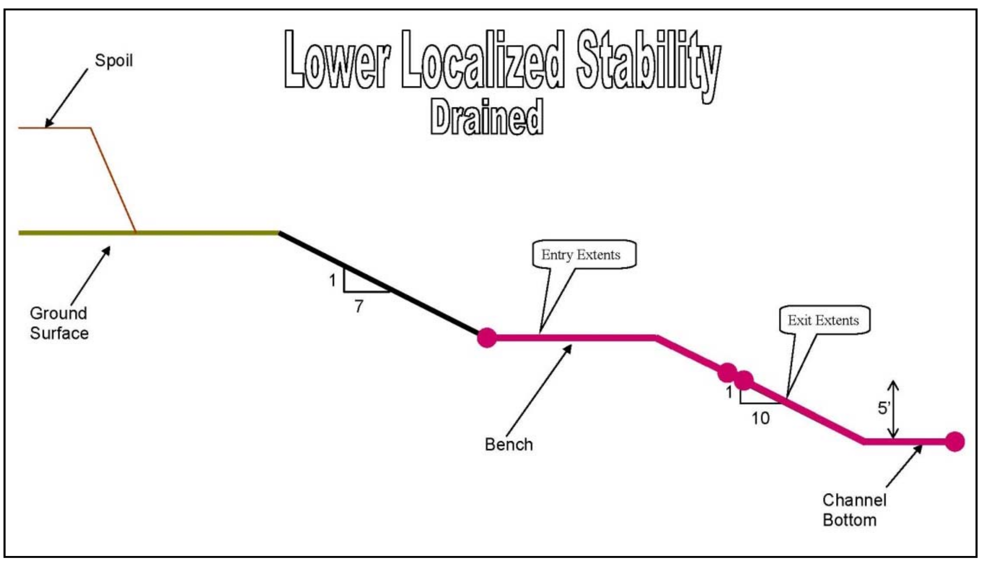

Figure I - 4: Localized Lower Slope, Long-Term Condition Search Extents

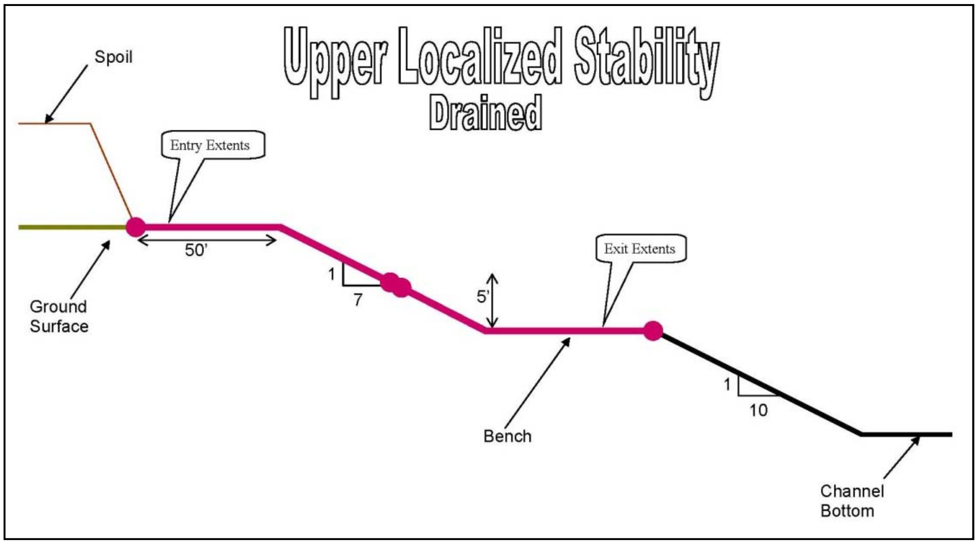

Figure I - 5: Localized Upper Slope, Long-Term Condition Search Extents

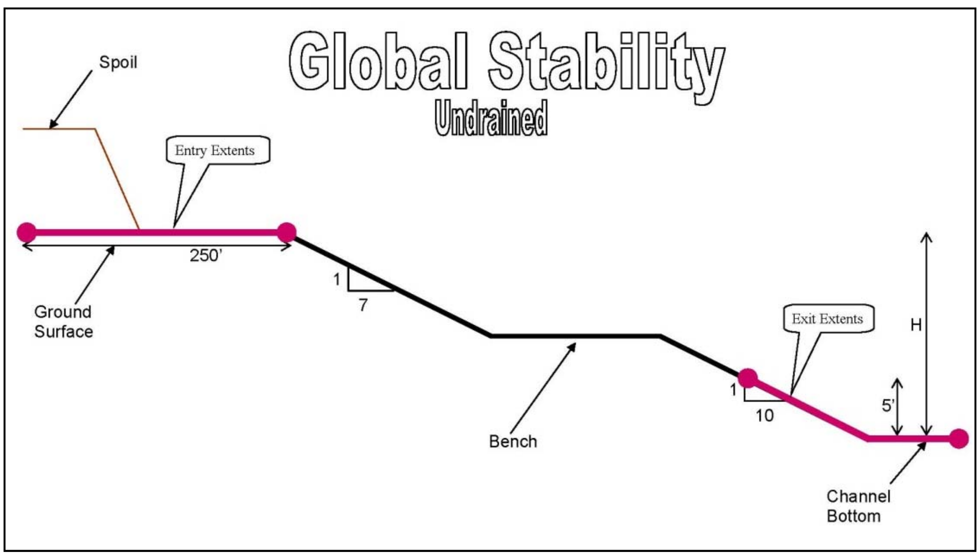

Figure I - 6: Global, Undrained Condition Search Extents

I.6.4.4. Low Flow Channel

I.6.4.4.1. The design of the diversion channels include a low flow channel located at the centerline of the diversion. The low flow channel is 3 feet deep with a bottom width of 10 feet. The side slopes are 1V:4H. During the initial part of Phase 3 it was recommended that the low flow channel be lined with riprap to minimize erosion of the low flow and channel bottom. The Phase 3 seepage and stability models included the riprap lined low flow channel.

I.6.4.4.2. Alternatives to using a riprap lined low flow channel were investigated during Phase 3. A recommended alternative included placement of riprap along the lower portion of the slope and constructing grade control structures every 5,000 feet. The revisions to the ND Diversion alternative during Phase 4 to the seepage and stability models removed the riprap from the low flow. In addition, no riprap was included along the lower portion of the slope.

I.6.4.5. Spoil Piles

I.6.4.5.1. The excavation for the diversion channel alternatives result in a large quantity of material that needs to be disposed of. The most economic way to excavate the channel is to place the spoil adjacent to the diversion channel. The spoil pile height was selected to be 15 feet based on consideration of the following issues:

- Real Estate: The higher the spoil pile height, the less real estate that is required in order to spoil excavated materials.
- Stability of Excavated Slope: The higher the spoil pile height, the increased potential to induce higher destabilizing forces on the excavation slope.
- Settlement of Spoil Pile: There will not be much compactive effort nor any moisture control when placing the spoil material, therefore the higher the spoil pile, the greater the potential for settlement of the spoil and also differential settlement.
- Consolidation of Foundation Materials: The higher the spoil pile, the more the foundation is likely to consolidate and settle.
- Bearing of the Spoil Pile: With lack of compactive effort and moisture control of the spoil pile, along with the natural consistency of the excavated materials, the strength of the spoil pile in reference to supporting farm equipment on top, could potential decrease with increasing spoil pile height.

I.6.4.5.2. For Phase 2, the assumption was made that the spoil would be placed immediately adjacent to the excavated slope at a 1V:7H slope, which was an extension to the excavated slope grade. The height of the spoil pile was limited to 15 feet. In the seepage and stability analyses, the spoil pile was modeled as a material with permeability and shear strength assigned to it.

I.6.4.5.3. During Phase 3, the way the spoil pile was modeled was reevaluated. It was decided that the spoil pile would be modeled as a surcharge with a unit weight of 125 pcf and a maximum height of 15 feet. The slope of the spoil was maintained at 1V:7H, but it was set back 50 feet from the top of the excavated channel. The spoil piles were evaluated the same way during Phase 4.

I.6.4.6. Bridge Crossing

I.6.4.6.1. At the beginning of Phase 2, it was recommended to use a channel slope of 1V:5H at the location of bridge crossings in order to minimize the length of the bridges. Based on the results of the seepage and stability analysis completed during Phase 3, it was determined that the steeper slopes would be unstable and would require some form of reinforcement and/or stabilization techniques. Due to this, it was recommended that the slope configuration at the location of the bridges be the same as the configuration of the diversion channel, resulting in an increase to bridge lengths.

I.6.5. Phase 2 Results for the MN Diversion

I.6.5.1. The initial analyses on the four MN Diversion channel sections were completed for various channel bottom widths and depths. In the case of Section 4, it was found that the minimum FS was met for any excavated channel depths. For Section 1, only a shallow excavated channel met the required FS for long-term slope stability. It was found for Section 2 and 3 that the minimum FS for long-term slope stability could not be met for any of the excavated channel depths. These findings led to the recommendation that the maximum channel excavation depth allowed would be approximately 30 feet and that benching of the slope or other means to increase stability may be required in certain locations. A summary of these results are presented below in Table I - 9.

Table I - 9: Summary of Initial MN Diversion Stability Results

						Factor of Safety Global Slope Stability	
Cross-Section	Channel Bottom Width [ft]	Ground Surface Elevatioin [ft]	Water Table Depth [ft]	Channel Elevation [ft]	Channel Depth [ft]	Long-Term (Drained)	Short-Term (Undrained)
1	50	886	881.5	860	26	1.103	N/D
	225		881.5	868	18	1.293	N/D
	500		881.5	876	10	1.706	1.787
2	50	911	906.1	866	45	0.710	N/D
	225		906.1	874	37	0.833	N/D
	500		906.1	882	29	1.277	1.674
3	50	913	906.9	870	43	1.121	N/D
	225		906.9	878	35	1.116	N/D
	500		906.9	886	27	1.246	1.351
4	50	910	906.6	875	35	2.086	N/D
	225		906.6	884	26	2.542	N/D
	500		906.6	892	18	3.080	3.278

I.6.5.2. The initial geotechnical analyses were revised using the revised design parameters. In addition, the revised geotechnical analyses were only completed for a 500-foot wide bottom and the associated excavation profile. The results of the revised analyses were similar to the initial analyses in the fact that Sections 1 and 4 met the minimum required FS for slope stability and Sections 2 and 3 did not. A summary of the revised results is presented below in Table I - 10.

Table I - 10: Summary of Revised MN Diversion Stability Results

						Factor of Safety							
						Long-Term (Drained)						Short-Term (Undrained)	
Cross-Section	Channel Bottom Width [ft]	Ground Surface Elevatioin [ft]	Water Table Depth [ft]	Channel Elevation [ft]	Channel Depth [ft]	Global - Circular Opt	Global - Circular	Global Wedge Opt	Global Wedge	Localized Circular Opt	Localized Circular	Global Circular Opt	Global Circular
1	500	886	881.5	876	10	1.722	1.746	N/D	N/D	1.887	N/D	1.935	1.967
2	500	911	906.1	882	29	0.886	0.928	N/D	N/D	0.386	0.408	1.607	1.693
3	500	913	906.9	886	27	1.201	1.206	1.204	1.321	1.083	1.098	1.325	1.350
4	500	910	906.6	892	18	2.689	2.705	N/D	N/D	2.484	2.536	2.874	3.142

I.6.5.3. The revised geotechnical analyses required that modifications be made to the diversion channel slopes for Sections 2 and 3. The modifications to Section 2 included adding a 25-foot wide riprap berm on the slope, flattening the slope in the Brenna formation to 1V on 10H, and including a 118-foot wide bench in the channel slope. In the case of Section 3, the slope in the Brenna formation was flattened to 1V on 12H. These modifications to the channel slope increased the FS to the required value. A summary of the results of modified channel are presented below in Table I - 11.

Table I - 11: Summary of Stability Results for the Modified MN Diversion Sections

						Factor of Safety							
						Long-Term (Drained)						Short-Term (Undrained)	
Cross-Section	Channel Bottom Width [ft]	Ground Surface Elevatioin [ft]	Water Table Depth [ft]	Channel Elevation [ft]	Channel Depth [ft]	Global - Circular Opt	Global - Circular	Global Wedge Opt	Global Wedge	Localized Circular Opt	Localized Circular	Global Circular Opt	Global Circular
2	500	911	906.1	882	29	1.390	1.577	N/D	N/D	1.041	1.138	2.189	2.662
3	500	913	906.9	886	27	1.370	1.463	1.434	1.581	1.452	1.503	1.467	1.511

I.6.5.4. The modifications to Section 2 and 3 increased the required amount of excavation and real estate. In addition, riprap is required for Section 2 only. These modifications were not required along the entire Minnesota Diversion channel alternative, only in the areas associated with the two sections. A review of the soil explorations indicated that approximate 16,000 LF of diversion channel had stratigraphy similar to Section 2 while the stratigraphy along approximately 73,500 LF of diversion channel was similar to Section 3. As such, the various modifications are required along approximately 89,500 LF, or 67% of the total diversion channel length. Calculations were performed to determine what the overall affect is on the estimated excavation and real estate needs. Using a weighted average for the modifications, it was determined that overall, excavation would need to be increased by approximately 5% to account for the flatter slopes and bench and real estate would also increase by approximately 5%. A summary of the increased quantities is presented below in Table I - 12.

Table I - 12: Summary of Increased Quantities Due to Channel Modifications

Weighted Average for Increased Quantities				
	Length	Increased Excavation	Increased Real Estate	Required Riprap (CY)
No Mod	44,245	0	0	0
Mod 2	16,000	25.4%	24.8%	266,667
Mod 3	73,500	2.9%	4.0%	0
Total Length	133,745			
Weighted Average Increase		**4.6%**	**5.1%**	266,667

I.6.5.5. All analysis results for the Minnesota Diversion alternative are presented in Attachment I-9.

I.6.6. Phase 3 and 4 Design Sections

I.6.6.1. The geology in the Fargo-Moorhead metro area is consistent over a fairly large extent. With this in mind, the design sections were selected where there were noticeable changes in stratigraphy or depth of the excavated channel. In all, eleven sections were selected along the MN Diversion channel alignment and two along the MN Extension channel. On the ND side, ten sections were selected. The locations of the sections along with the reach they represent are summarized below in Table I - 13 and Table I - 14. The locations are also graphically represented on the maps contained within Attachment I-2.

Table I - 13: MN Diversion Section Locations and Extents

		Reach		Distance		
Section	Location	Start	End	(feet)	(miles)	Percent
MN 1	STA 20+00	0+00	70+00	7,000	1.3	5%
MN 2	STA 175+00	70+00	220+00	15,000	2.8	11%
MN 2B	STA 350+00	220+00	420+00	20,000	3.8	15%
MN 3	STA 510+00	420+00	570+00	15,000	2.8	11%
MN 4A	STA 582+00	570+00	640+00	7,000	1.3	5%
MN 4B	STA 688+00	640+00	730+00	9,000	1.7	7%
MN 5B	STA 796+00	730+00	850+00	12,000	2.3	9%
MN 6	STA 930+00	850+00	1055+00	20,500	3.9	16%
MN 7	STA 1100+00	1055+00	1155+00	10,000	1.9	8%
MN 7B	STA 1185+00	1155+00	1235+00	8,000	1.5	6%
MN 8	STA 1270+00	1235+00	1309+80	7,400	1.4	6%
				130,900	25.9	
MN 9A	EXT CH STA 80+00	0+00	105+00	10,500	2.0	64%
MN 9B	EXT CH STA 130+00	105+00	164+84	5,900	1.1	36%
				16,400	3.1	

Table I - 14: ND Diversion Section Locations and Extents

Section	Location	Reach Start	Reach End	Distance (feet)	Distance (miles)	Percent
ND 1	STA 120+00	0+00	390+00	39,000	7.4	20%
ND 2	STA 545+00	390+00	660+00	27,000	5.1	14%
ND 3	STA 940+00	660+00	1000+00	34,000	6.4	18%
ND 4	STA 1080+00	1000+00	1150+00	15,000	2.8	8%
ND 5	STA 1225+00	1150+00	1300+00	15,000	2.8	8%
ND 5B	STA 1445+00	1300+00	1490+00	19,000	3.6	10%
ND 6B	STA 1550+00	1490+00	1550+00	6,000	1.1	3%
ND 6B U/S	STA 1550+00	1550+00	1670+00	12,000	2.3	6%
ND 6C	STA 1720+00	1670+00	1770+00	10,000	1.9	5%
ND 7	STA 1810+00	1770+00	1922+00	15,200	2.9	8%
				192,200	36.4	

I.6.6.2. When developing the seepage and stability models for these sections, the ground surface was generally set at the highest elevation (rounded up to the nearest foot) within plus or minus 3,000 feet of the section location. For the channel invert, the lowest elevation (rounded down to the nearest foot) within 3,000 feet was selected. The stratigraphy in the seepage and stability models was generally depicted as the most conservative representation encountered between the limits of the section. A summary of the ground surface, diversion channel invert, and stratigraphy are shown in Attachment I-10.

I.6.6.3. There were many changes made in the way the stability of the diversion channel excavated slopes was evaluated after Phase 2. First, additional soil information was gathered along the MN Diversion channel alignment that required refinement to the stratigraphy. Secondly, the number of cross sections analyzed increased from four to eleven along the MN Diversion channel and two for the MN Extension channel. Thirdly, soil borings were obtained along the ND Diversion alignment and stratigraphy was developed for ten cross sections. Previously in Phase 2, no analyses were completed for the ND Diversion alternative. Fourthly, the shear strength tests results were reevaluated and curvilinear shear strength parameters were developed for use in the stability analyses for the diversion channels. These changes resulted in a major change in the configuration of the diversion channel slopes.

I.6.6.4. The initial seepage and stability analyses completed at the beginning of Phase 3 were done for the ND Diversion channel. The results of the analyses indicated that the 1V:7H excavated slope did not meet the target slope stability FS. In order to meet the target FS, the invert of ND Diversion channel was raised 3 feet from the elevation analyzed in Phase 2. In addition, a bench was added. The bench height and width were set at 7 and 70 feet, respectively, and were kept constant along the length of the diversion channel to simplify the hydraulic modeling. The flatter bench slope, 1V:10H was required to increase drained stability of the lower slope. The bottom width of the ND Diversion channel was 100 feet for the majority of the alignment for both Phase 2 and Phase 3.

I.6.6.5. During Phase 4, the ND Diversion channel was redesigned to meet revised hydraulic requirements. The changes included raising the invert again, generally between 2 and 8 feet. In addition, the riprap was removed from the low flow channel. The decrease in depth resulted in slopes being more stable, reducing the benching requirements and allowing the bottom portion of the channel slope to be steepened to a slope of 1V:7H.

I.6.6.6. Upon completion of the initial analyses for the ND Diversion, the MN Diversion was reanalyzed. The previous results in Phase 2 had indicated that the 1V:7H slope did not meet the target FS for the majority of the MN alignment. Taking what was learned from the ND Diversion channel analyses, a bench was also added to the MN Diversion channel. The benching requirement was a 7-foot high, 70-foot wide bench with the slope flattened to 1V:10H below the bench, and kept constant along the entire length of the diversion channel.

I.6.6.7. The soil exploration was being completed at the same time as the seepage and stability analyses were being done for the MN Diversion alternative. The soil data was incorporated into the models as it was received. The additional exploration completed around the Dilworth area indicated that there was a sand layer starting at a depth between 60 to 90 feet below the ground surface. The sand layer encountered along the MN Diversion alignment was assumed to be part of the Buffalo aquifer formation. The elevation of this sand layer with respect to the invert of the diversion channel had a significant effect on the stability and the uplift on the impervious blanket. Due to these concerns, the portion of the MN Diversion alignment just north of Dilworth was shifted to the west in addition to raising the invert 4 feet from the inlet structure through this area. The same benching requirements as that of the ND Diversion channel in Phase 3 (7-foot high, 70-foot wide bench with the slope flattened to 1V:10H below the bench) were used for the MN Diversion alignment.

I.6.6.8. During the Phase 3 and 4 analyses, it was found that certain diversion channel sections did not meet the target FS for the global undrained condition with the spoil pile at its maximum height of 15 feet. At these sections, the spoil pile height was reduced for a set length near the diversion channel and then stepped up to the maximum height. For the MN Diversion, the spoil pile stepped length was 100 feet while for the ND Diversion, it was 50 feet. The reduction of the spoil pile height resulted in an increase to the global undrained FSs. The modified configuration of the spoil pile is illustrated in Figure I - 7 below.

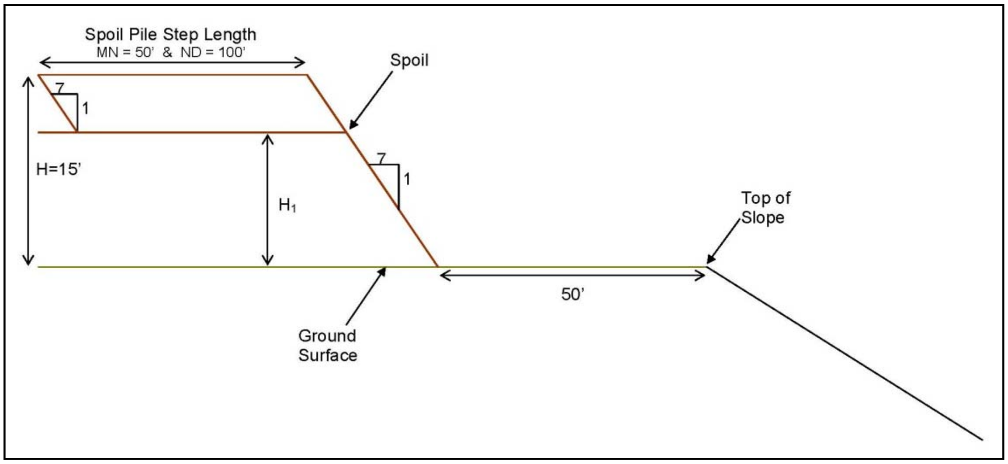

Figure I - 7: Spoil Pile Configuration With Step

I.6.6.9. The changes made to the configuration of the diversion channel slopes, the invert elevations of the channels, and spoil pile height and step lengths were coordinated with the required disciplines (i.e. hydraulics, civil layout, cost estimating). The final configuration of the sections for the MN Diversion channel and ND Diversion channel are summarized below in Table I - 15 and Table I - 16, respectively.

Table I - 15: MN Diversion Channel Configuration – Phase 3

	Channel Configuration				Bench Configuration				
Section	Ground Surface (ft)	Bottom Elev. (ft)	Depth (ft)	Bottom Width (ft)	Bench Width (ft)	Bench Height (ft)	Bench Slope (1V:_H)	Channel Slope (1V:_H	Spoil Pile Height, H_1 (ft)
MN Section 1	895	875	20	225	70	7	10	7	11
MN Section 2	894	876	18	225	70	7	10	7	15
MN Section 2B	896	878	18	225	70	7	10	7	15
MN Section 3	906	880	26	225	70	7	10	7	11
MN Section 4A	910	884	26	400	70	7	10	10	13
MN Section 4B	912	885	27	400	70	7	10	10	15
MN Section 5B	912	888	24	400	70	7	10	7	15
MN Section 6	914	890	24	400	70	7	10	7	15
MN Section 7	914	894	20	400	70	7	10	7	15
MN Section 7B	910	895	15	400	70	7	10	7	15
MN Section 8	912	897	15	400	70	7	10	7	15
MN Section 9A	911	897	14	215				7	15
MN Section 9B	914	900	14	215				7	15

Table I - 16: ND Diversion Channel Configuration – Phase 4

Section	Channel Configuration: Ground Surface (ft)	Channel Configuration: Bottom Elev. (ft)	Channel Configuration: Depth (ft)	Channel Configuration: Bottom Width (ft)	Bench Configuration: Bench Width (ft)	Bench Configuration: Bench Height (ft)	Bench Configuration: Bench Slope (1V:_H)	Bench Configuration: Channel Slope (1V:_H	Spoil Pile Height, H_1 (ft)
ND Section 1	882	865	17	250				7	8
ND Section 2	890	868	22	250	15	8	7	7	11
ND Section 3	900	877	23	250	25	8	7	7	15
ND Section 4	900	879	21	250	20	8	7	7	15
ND Section 5	903	881	22	250	40	8	7	7	12
ND Section 5B	913	885	28	250	40	8	7	7	
ND Section 6B	920	903	17	250				7	
ND Section 6C	913	905	8	250				7	
ND Section 7	912	901	11	250				7	

I.6.7. Phase 3 and 4 Results for the Diversion Channel Alternatives

I.6.7.1. The slope stability results for the MN and ND Diversion channel alternatives are presented below in Table I - 17 and Table I - 18, respectively. The tables indicate the minimum FS determined for the five slope stability conditions analyzed. Table I - 17, for the MN Diversion channel has an additional column which indicates the side of the channel that the FS corresponds to, as the stratigraphy for MN Sections 4A, 4B, and 5B could not be assumed uniform laterally across the section. The side of the channel is identified by looking downstream.

Table I - 17: MN Diversion Channel Results – Phase 3

Section	Stability Analysis: Min FS					Side of Channel
	Effective Stress				Total Stress	
	(A) Global Entry/Exit	(B) Wedge (Global)	(C) Lower Localized	(D) Upper Localized	(2) Undrained Global	
MN Section 1	1.701	1.828	2.604	2.230	1.331	Left Side
MN Section 2	1.911	1.899	3.647	3.245	1.368	Left Side
MN Section 2B	2.041	1.834	2.642	2.680	1.476	Left Side
MN Section 3	1.657	1.634	2.029	1.703	1.442	Left Side
MN Section 4A	1.474	1.478	1.652	1.502	1.306	Left Side
	1.475	ND	ND	ND	1.306	Right
MN Section 4B	1.457	1.456	1.571	1.447	1.357	Left Side
	1.447	1.419	1.484	1.420	1.395	Right
MN Section 5B	1.498	1.483	1.722	1.488	1.436	Left Side
	1.568	ND	ND	ND	1.442	Right
MN Section 6	1.611	1.592	1.952	1.476	1.393	Left Side
MN Section 7	1.839	1.839	2.048	1.819	1.533	Left Side
MN Section 7B	2.191	2.195	2.127	2.115	1.683	Left Side
MN Section 8	2.080	2.080	1.875	2.189	1.724	Left Side
MN Section 9A	2.026	2.029	2.156	2.025	1.872	Left Side
MN Section 9B	1.616	1.620	1.996	1.615	1.600	Left Side

Table I - 18: ND Diversion Channel Results – Phase 4

	Stability Analysis: Min FS				
	Effective Stress				Total Stress
Section	(A) Global Entry/Exit	(B) Wedge (Global)	(C) Lower Localized	(D) Upper Localized	(2) Undrained Global
ND Section 1	1.554	1.558	1.702	2.206	1.345
ND Section 2	1.431	1.437	1.508	1.957	1.315
ND Section 3	1.421	1.411	1.710	1.791	1.332
ND Section 4	1.429	1.425	1.611	1.880	1.364
ND Section 5	1.411	1.412	1.418	1.640	1.306
ND Section 5B	1.416	1.415	1.208	1.409	1.507
ND Section 6B	1.512	1.513	1.636	2.146	1.563
ND Section 6C	2.353	2.350	2.565	4.330	2.092
ND Section 7	2.201	2.173	2.274	3.116	2.000

I.6.7.2. The Phase 3 results for the MN Diversion channel and the Phase 4 results for the ND Diversion channel indicate that the channels meet the target FSs for all conditions. The results from the seepage and stability for the MN Diversion channel are located in Attachment I-11 and the results for ND Diversion channel are in Attachment I-12.

I.6.7.3. A check of the stability analysis was completed during Phase 3 using Slide 5.0 from Rocscience. Slide 5.0 is a limit equilibrium slope stability program which is also capable of coupling the results of the steady state seepage calculations into the stability analysis. The sections checked were MN Diversion Section 6 and ND Diversion Section 2, as these sections had the lowest long-term global stability FS for the sections with assumed symmetric about the channel centerline. The results from Slide 5.0 indicated similar factors of safety as that which were determined by GeoStudio. For MN Diversion Section 6, Slide 5.0 calculated a slightly different failure surface with a FS of 1.564, slightly lower than the Slope/W FS of 1.611. For ND Diversion Section 2, Slide 5.0 calculated a FS of 1.433 while Slope/W FS was 1.411. In this instance, the failure surfaces between the two programs were similar. These results are included at the end of the respective diversion analysis result attachments.

I.6.8. Uplift Analysis and Results

I.6.8.1. In addition to evaluating the diversion channel excavated slopes for stability, uplift on the bottom of the channel was also evaluated in areas where impervious foundation materials were overlaying a pervious substratum. One area in which this condition existed was along the MN Diversion channel alignment near and around Dilworth, MN. The extent of this area runs from approximately STA 440+00 to STA 880+00.

I.6.8.2. The calculation of the uplift factor of safety can be expressed as the critical exit gradient divided by the average gradient through the impervious foundation. This expression is shown in the below equation. The target FS for uplift of 1.5 was selected. The FS value of 1.5 is similar to the FS suggested in Engineering Manuals that deal with seepage [i.e. EM 1110-2-1914, Relief Wells (Reference I.13.6); EM 1110-2-1913, Levee Design (Reference I.13.5; EM 1110-2-1901, Seepage for Dams Reference I.13.3)].

$$FS = \frac{i_c}{i_o} = \frac{\left(\frac{\gamma_{sat} - \gamma_w}{\gamma_w}\right)}{\left(\frac{\Delta h}{Z_T}\right)} = \frac{(\gamma_{sat} - \gamma_w) \times Z_T}{\gamma_w \times \Delta h}$$

Where:
i_c = the critical exit gradient
I_o = the average gradient through the impervious foundation
Δh = the difference in head between the pervious substratum and the head at the top of the impervious foundation
Z_T = the transformed thickness of the impervious foundation
γ_{sat} = saturated soil unit weight
γ_w = unit weight of water

I.6.8.3. The piezometric head in the previous substratum used to calculate the uplift FSs was assumed to be 7.5 feet below the ground surface. This value was decided upon after review of the observed piezometric levels from the surrounding wells monitored by the Minnesota Department of Natural Resources (MN DNR) and the vibrating wire piezometers installed by the St. Paul District for the feasibility study. The observed piezometric levels in the MN DNR observation wells range from 7.5 feet below ground surface (BGS) to 17 feet BGS. There is a chart in Attachment I-13 that shows these observed levels along with a map indicating the well locations. The USACE's vibrating wire piezometers have been observing piezometric levels in the range of 10 to 14 feet BGS. The value of 7.5 feet BGS represents the highest elevation that has been observed in the area.

I.6.8.4. The stratigraphy used to calculate the uplift FSs was taken directly from the boring logs. The borings indicated that the impervious foundation materials were composed of the Brenna and Argusville formations. There were many instances in which silt (semi-impervious) and silty sand (semi-pervious) formations were encountered beneath the impervious material. The pervious substratum, composed of clean sands (SP) was found beneath the impervious, semi-impervious, and semi-pervious formations.

I.6.8.5. The uplift FSs were calculated at all the borings locations throughout this area. These borings were not always located at or near the centerline of the diversion channel. The general trend was that the uplift FS decreased as the boring locations moved east of the alignment. This was due to the pervious substratum thickness increasing toward the east and rising upward in the stratigraphy. The uplift FSs calculated along the centerline are summarized in Table I - 19. The results indicate that the target FS of 1.5 is obtained for all locations along the centerline with the exception of the location at 12th Ave S. At this location, the two borings were taken on either side of the alignment centerline because access to the centerline was an issue. The results indicate boring 10-102M, located west of the centerline had a FS of 0.9 while boring 10-103M, located to the east, was 1.6. There were no clean sands (SP) encountered in boring 10-102M, only a silty sand (SP-SM) at a depth of 54 feet BGS and the uplift FS reported was calculated based on this elevation. This low FS is considered to represent the worst case and that the actual uplift condition along the MN Diversion channel in this location would be acceptable. Further soil exploration is needed in this area to verify this assumption.

Table I - 19: Uplift Factor of Safety Along MN Diversion Channel Centerline

Area	STA	Offset		Boring	FS (gradient)
70th Ave NW	~440+00	400	Right	10-87M	**5.28**
57th Ave N	~510+00	400	Left	10-82M	**3.59**
43rd Ave N	~585+00	900	Right	10-93M	**2.07**
28th Ave N	~636+00	400	Left	10-104M	**1.74**
15th Ave N	~688+00	50	Right	10-83M	**1.87**
HWY 10	~741+00	150	Right	10-103M	**1.46**
12th Ave S	~795+00	805	Left	10-102M	0.86
	~795+00	2800	Right	10-101M	1.61
I-94	~850+00	4300	Right	09-43C	**2.63**
40th Ave S	~-910+00	3100	Right	09-15M	**2.74**

I.6.9. Geotechnical Considerations for Structures

I.6.9.1. As previously stated, Barr Engineering Company completed the geotechnical analyses and design for the various structural components of the diversion channel alternatives. These analyses are detailed in Appendix F of the RRD Report (Reference I.13.1) and can be found in the main report's Attachment 5. The various geotechnical analyses completed by Barr Engineering Company for the various components along with the location within Appendix F further details can be found is detailed below in Table I - 20.

Table I - 20: Location of Barr's Geotechnical Analyses

Geotechnical Analysis	Location
General Geotechncial Engineering Discussion	RRD, Appendix F, pg F-47
General Pile Design Discussion	RRD, Appendix F, Section F4.1.1, pg F-56
Erosion Control Methods for Low Flow Channel	RRD, Appendix F, Exhibit K
Review of Geotechnical Data	RRD, Appendix F, Exhibit L
Seepage Analysis for the Hydraulic Structures	RRD, Appendix F, Exhibit M
Slope Stability Analysis for the Storage Area 1 Levees	RRD, Appendix F, Exhibit N
Slope Stability Analysis for the radial walls associated with the Sheyenne and Maple River hydraulic structures	RRD, Appendix F, Exhibit N
Slope Stability Analysis for the radial walls associated with the Sheyenne and Maple River hydraulic structures	RRD, Appendix F, Exhibit N
Pile Capacity	RRD, Appendix F, Exhibit O

I.6.10. Conclusion

I.6.10.1. Overall, the geotechnical design of the MN and ND Diversion channel is considered to meet the intent of the feasibility study, which is to provide a preliminary design and cost estimate for an implementable project. The results of the geotechnical analyses presented above support this conclusion. It is also recognized that additional work will be required during the Plans and Specification phase. Some of the tasks and issues to be further evaluated are summarized below in Section I.12.0.

I.7.0. GROUND WATER CONSIDERATIONS

I.7.0.1. For the proposed Minnesota alignment alternatives the Buffalo Aquifer was identified as a planning constraint early in the feasibility study. Water usage from the aquifer has declined in recent years but is still tapped for individual, irrigation, and municipal water wells. The Buffalo Aquifer may be characterized as a north-south trending, complex, heterogeneous outwash deposit composed of primarily of sand and gravel placed during the last glacial epoch. Studies have shown that along its east-west boundaries the Buffalo aquifer becomes increasingly fine-grained and can include silt and clay beds. Located five to seven miles east of Moorhead, the deposit is interpreted to have been formed in a tunnel valley by glacial meltwater exiting the southern end, or snout, of a glacier. The exiting meltwater was under pressure and occurred in multiple events which are indicated by the vertical and horizontal meandering of the deposit. In Clay County the Buffalo Aquifer is 1 to 2 miles wide, and up to 250-feet thick. The top of the aquifer is at, or very near, ground surface adjacent to the Buffalo River but is buried in glacial lake clays along diversion alignments proposed to date.

I.7.0.2. The Buffalo River, located approximately 5-miles east of Moorhead, runs parallel to and along the east side of the aquifer and contributes significant recharge; especially in the northern reach of the aquifer near the City of Moorhead's north well field. Regional aquifer flow in the clayey lake plain soils adjacent is generally westward or toward the Red River of the North; variations due to local hydrology, such as over-pumping, drought conditions, and adjacent wetlands can alter local groundwater flow directions.

I.7.0.3. In 1994 the City of Moorhead opened a new water treatment plant and began taking more water from the Red River of the North. Water levels in the aquifer have risen approximately 15 feet in the succeeding 10 years. Over the last 30 years, many studies have been conducted on the Buffalo Aquifer and additional groundwater management initiatives and studies are ongoing.

I.7.0.4. For the proposed North Dakota alignment alternatives the West Fargo Aquifer is the primary water source of concern. It is possible to divide the West Fargo Aquifer into several separate sub-units but, for the purposes of this report it shall be treated as one. Water from the aquifer is tapped for individual, irrigation, and municipal water wells. The West Fargo Aquifer is a buried glacio-fluvial deposit placed during the last glacial epoch that extends generally in a north-south direction for about 30 miles in Cass County. The modern day Sheyenne River traverses the same general trend of the West Fargo Aquifer from about 6 miles south of Horace, ND to about 2 miles south of Argusville, ND. The aquifer ranges in width from about 2 ½ to 8-miles and underlies an area of approximately 110 miles. Typically the aquifer is overlain by deposits of glacial till and glacio-lacustrine lake clay at depths of approximately 70 to 170 feet below ground surface. The aquifer is composed of material ranging in size from fine sand to boulders but is primarily fine to medium sand. In places these coarse grained deposits may be interbedded with silt or clay, especially near the top of the aquifer. The deposit is interpreted to have been formed in a tunnel valley by glacial meltwater exiting the southern end, or snout, of a glacier, in the same manner as the Buffalo aquifer.

I.7.0.5. Recharge to the West Fargo Aquifer probably occurs primarily through lateral movement of water through the till and associated deposits and by downward percolation of shallow groundwater through the glacio-lacustrine deposits. Due to the relatively tight nature of the surrounding soils it is likely that the recharge rate of the aquifer is not able to keep pace with the withdrawal rate and this is reflected in declining water levels. Regional aquifer flow appears to be influenced by areas of heavy pumping but generally the piezometric surface slopes from east to west. The average depth of the water level in the West Fargo Aquifer is not defined but it is known that the decline is such that unconfined (non-artesian) conditions now exist.

I.7.0.6. The city of West Fargo draws its municipal water supply entirely from 8 production wells located in the West Fargo Aquifer. Until alternate water sources are located it is reasonable to assume that water levels will continue to decline in the aquifer.

I.7.0.7. Other, unnamed aquifers occur at various depths within the tills and glacio-lacustrine clays adjacent to the proposed diversion alignments. These buried aquifers may generally be characterized as elongate, discontinuous, lenses composed primarily of sand and gravel. Accurately locating and delineating these aquifers is difficult due to their scattered nature and relatively small aerial extent. On-going studies by the Corps of Engineers and others will aid in better defining these types of aquifers.

I.7.1. Shallow Groundwater

I.7.1.1. All of the diversion channel alternatives would have a similar effect on shallow groundwater. The shallow groundwater table is defined as the locally observed groundwater table near the ground surface; typically located within the first 15 feet below the ground surface. The groundwater table fluctuates seasonally, depending on the soil type, precipitation and climatic conditions experienced throughout the year or years. Periodic fluctuation of the groundwater table is assumed to occur even without the construction of a diversion channel. Groundwater is not considered a significant source for water in the area due to the relatively low permeability of soils and the low volume of water expected to flow through these soils.

I.7.1.2. Under the conditions reasonably anticipated, the flow of the shallow groundwater should be “downhill” or toward the area of lower hydraulic potential. After the excavation of a diversion channel is completed, the “downhill” or lowest potential area should be the bottom of the diversion channel. This may lower the groundwater table near the diversion channel but, at most, only to the depth of the excavated diversion channel. The lateral extent of the lowered groundwater table would likely be confined to areas immediately adjacent to the diversion channel including the spoil banks. Areas outside the extent of this would likely see very little to no change. The natural groundwater flow quantities through tight clayey soils would reasonably be expected to be relatively small.

I.7.1.3. A lowered shallow groundwater table could potentially reduce the capacity of shallow local wells that are recharged by the groundwater table. The risk to the shallow groundwater table as related to the proposed diversion is low because the anticipated area effects would be concentrated adjacent to the diversion channel. The lowering of the shallow groundwater table may cause consolidation of the surrounding soils and settlement of structures within the area affected. Only structures immediately adjacent to the proposed diversion channel would have the potential to settle. Since the area affected is not expected to extend beyond the channel and spoil piles it is unlikely that any structures remaining after construction would be impacted. If local shallow wells experience reduction in capacity, the depth of the well could be increased or an additional well be installed to mitigate for the reduced capacity. Wells and structures that are within the proposed footprint of the diversion would be removed or abandoned, while those immediately adjacent would be identified and monitored to quantify any impacts.

I.8.0. PHASE I ENVIRONMENTAL SITE ASSESSMENT

I.8.0.1. A Phase I Environmental Site Assessment (ESA) was completed for both the MN and ND Diversion channel alternatives in December 2010. It conformed to ASTM Standard Practice E1527-00. The ESA recommended a limited Phase II Environmental Site Assessment depending upon the ultimate selected diversion alternative.

I.9.0. SOURCES OF CONSTRUCTION MATERIALS

I.9.1. Borrow Sources

I.9.1.1. The In-Town Levee alternative is the only alternative in which borrow materials would need to be identified for the construction of levees. This material is readily available. The local sponsor is responsible for identifying sites to be used as borrow sources. Any locations proposed will need to be evaluated from a geotechnical and cultural/archeological perspective prior to any use on the proposed project. Geotechnical parameters to be defined prior to approval include the thickness of topsoil, presence or absence of saline soils, thickness and suitability of alluvial/fluvial soils, water bearing seams and water table conditions, natural moisture content, determination of plasticity indices, gradation, and Proctor density. Soil cracking within the top 5 feet (freeboard zone) of earthen levees is a common occurrence throughout the Red River Valley. This problem may be alleviated by the application of a specific type of clayey soil to cap the levee. If determined necessary, these soils shall be identified and designated as “select” prior to completion of plans and specifications.

I.9.1.2. The borrow materials for the diversion channel alternatives will come from the excavation itself. There will be a surplus of excavated material that will require it to be spoiled adjacent to the diversion channel.

I.9.2. Concrete Aggregate, Riprap, and Bedding

I.9.2.1. Sources for fine and coarse concrete aggregate, bedding, and riprap should be available locally. Acceptable quality commercial aggregates in the Fargo/Moorhead vicinity are obtained from the beach ridges of glacial Lake Agassiz east of the Red River. Additional material may be available from field stone piles in farm fields. Most of the material consists of rounded, wave-washed boulders, cobbles, and sand. If large quantities of riprap size material are required, producers will need adequate lead time in order to stockpile material. Outside sources of quarried, angular, stone should also be available approximately 200 miles east of the proposed project in western and central Minnesota. Additional investigations will be necessary prior to plans and specifications in order to accurately quantify the amount of stone product available within a reasonable radius of the area.

I.10.0. INSTRUMENTATION

I.10.0.1. Instrumentation was installed during Phase 2 and Phase 3 that consisted of nested sets of vibratory wire piezometers and slope inclinometers. The purpose of the vibratory wire piezometers was to determine the groundwater table and piezometric levels in the different foundation materials. The purpose of the slope inclinometers was to determine the elevation of the slip surface in an active slide area along the Red River.

I.10.0.2. A total of seven nested sets of vibratory wire piezometers were installed. The piezometers were located in areas of interest, such as near the Buffalo Aquifer and locations of major structures along the diversion alignments. In addition, one piezometer was installed in the Buffalo aquifer. Piezometers P1 through P6 were installed in August 2009, P7 though P15 were installed in June 2010, and P16 through P23 were installed in July 2010. A summary of the piezometers installed and their locations is included in Table I - 21 below. These locations are illustrated by the figures in Attachment I-14.

Table I - 21: Piezometer Locations and Associated Boring

Piezmeters	Location	Boring	Ground Surface Elevation	Depth	Elevation
P1	Gooseberry Park, Moorhead	Moorhead 09-21M/P	886.2	16.3	869.9
P2				48.3	837.9
P3				75.3	810.9
P4	28th Ave N & 60th St N	Moorhead 09-14M/P	913.1	20.0	893.1
P5				50.0	863.1
P6				75.0	838.1
P7	MN Diversion Red River Control Structure	Moorhead 10-81M/P	907.8	30.0	877.8
P8				60.0	847.8
P9				90.0	817.8
P10	93rd Ave N & 60th St N	Moorhead 10-88M/P	911.3	35.0	876.3
P11				70.0	841.3
P12				105.0	806.3
P13	Wild Rice River, Brodshaug Farmyard	Fargo 10-77M/P	905.7	30.0	875.7
P14				50.0	855.7
P15				100.0	805.7
P16	ND Diversion Red River Control Structure	Fargo 10-79M/P	905.3	80.0	825.3
P17				20.0	885.3
P18				40.0	865.3
P19	MN Diversion, HWY 10	Moorhead 10-106M/P	913.3	19.4	893.9
P20				64.4	848.9
P21				81.4	831.9
P22				101.4	811.9
P23	Buffalo Aquifer, 28th Ave N	Moorhead 10-105M/P	916.6	43.0	873.6

I.10.0.3. The observed piezometric levels are illustrated by the figures in Attachment I-14. A discussion on the observed readings for each instrumentation cluster is provided below.

I.10.0.4. 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 and are remaining fairly constant. P1 (shallowest) levels continued to fall during the summer of 2011 but showed an increase in the fall to a level similar to P3 (deepest). These trends would indicate that there is a higher pressure at depth.

I.10.0.5. 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 levels of all piezometers have increased since their installation. P6 (deepest) has shown the greatest increase, approximately 5.5 feet, while P-5 (middle) and P4 (shallowest) has shown approximate 4 foot and 3 foot increases, respectfully. Still P6 (deepest), which is located in a sand formation, is reading the lowest piezometric levels, approximately 7 feet below ground surface (BGS).

I.10.0.6. Piezometers P7, P8, and P9 were installed at the proposed location of the Red River Control Structure for the MN Diversion channel alternative. These instruments are reading piezometric levels between 30 and 34 feet BGS. The readings also indicate that P7 (shallowest) is indicating the highest piezometric level while P8 (middle) is showing the lowest levels. In April 2011, the piezometer levels P8 and P9 increased 4 to 5 feet in and then decreased back to previous levels, which is in conjunction with the Spring 2011 flood.

I.10.0.7. 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. Piezometers P10 and P11 are showing a slight increase in piezometric levels since their installation, at approximately 2.5 feet. P10 (shallowest) is indicating piezometric levels approximately 8 feet BGS. P11 (middle) and P12 (deepest and installed in a sand formation) are indicating piezometric levels approximately 10.5 to 14 feet BGS, lower than P10 (shallowest), respectively. In April 2011, the piezometer levels for increased slightly and then decreased, which is in conjunction with Spring 2011 flood.

I.10.0.8. 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) indicated a decreasing piezometric level between July 2010 through February 2011, but then started increasing at a small rate. In May 2012, P13 indicated a piezometric level approximately 30 feet BGS. P14 (middle) is indicating a piezometric level decrease of about 18 feet, between July 2010 till May 2012. The May 2012 level for P14 is 52 feet BGS. The piezometric level for P15 (deepest) was fairly constant at a level between 78 and 80 feet BGS between July 2010 and October 2010. At the beginning of October 2010, there was a rapid increase in piezometric levels other one month time period, increasing approximately 5 feet to a level 75 feet BGS. Since November 2010, P15 has shown a small but steady increasing trend and as of May 2010 indicated a level 74 feet BGS.

I.10.0.9. Piezometers P16, P17, and P18 were installed near the proposed location of the Red River Control Structure for the ND Diversion channel alternative. These piezometers have shown a slight decrease in piezometric levels initially after installation but started increase in levels after about 2 months. P17 (shallowest) is indicating piezometric levels approximately 11 feet below ground surface. P18 (middle) and P16 (deepest) are indicating piezometric levels approximately 0.5 foot and 1 feet lower than P17, respectively.

I.10.0.10. 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 were not available for this cluster between July and November 2011. In December 2011, a new data collector was connected to these instruments and started logging. P19 (shallowest) is indicating piezometric levels 3 to 4 feet BGS. Data for piezometers P21 (lower middle) P22 (deepest) has only been collected for a short period of time, since April 2011. Over a two month period, these piezometers have increased 1 to 2 foot to a May 2012 level which is approximately 5 feet BGS. Between November 2011 and May 2010, P20 (upper middle) has shown approximately a 1-foot increase, with the piezometric level being 11 feet BGS.

I.10.0.11. 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. Like P19 through P22, this data collector was also inundated with water. Due to its location, it has been difficult to relocate the data collector. It is anticipated that the data collector will be relocated in the Summer of 2011.

I.11.0. REVIEWS

I.11.0.1. There have been a number of reviews completed during the feasibility study. The reviews included internal district peer reviews, agency technical reviews, and also an independent external peer review. Comments and questions have come up during these reviews that have prompted changes to the analyses and report. It was considered important to include these comments and questions and the associated responses in the report. It is a way to illustrate the changes made throughout the feasibility study and how comments and questions were considered. The different reviews can be found in Attachment I-15.

I.12.0. ADDITIONAL WORK

I.12.0.1. The geotechnical analysis and geology interpretation for the feasibility study was completed in enough detail to evaluate the different alternatives and estimate their costs. It is acknowledged that additional work will be required for the Plans and Specifications stage. A summary of the additional work that is anticipated is listed below.

(1) Soil Exploration: Additional soil exploration will be required to refine the stratigraphy along the selected project. This is especially true for the MN Diversion channel alignment in the area near Dilworth.

(2) Undisturbed Testing: Additional undisturbed testing should be completed to verify the drained and undrained strengths of the critical formations (i.e. Oxidized Brenna, Brenna, and Argusville). In addition, sampling and testing of the till formation is needed.

(3) Test Excavation: A test excavation could be completed that is instrumented to try and capture the changes in pore pressure and any movements that occur. Observation of the test excavation could help in refining the method(s) used to excavate the diversion channel.

(4) Diversion Channel Analysis: Additional analyses to evaluate the stability of the diversion channel are required. These analyses should account for any changes in stratigraphy or shear strength parameters determined because of additional exploration and testing. In addition, the configuration of the diversion channel and spoil piles should be refined to minimize volume and cost of excavation. Analyses that incorporate the construction staging should be performed to supplement the results of the traditional analyses completed to date.

(5) Diversion Channel Slopes at Bridges: Investigate techniques that would improve stability at bridge locations so steeper slopes could be used and determine the cost of these alternatives.

(6) Pile Load Tests: A pile load test was developed in Spring 2011 and will be conducted between July 2011 and November 2011. The results of the pile load test will help in the understanding of pile behavior under load and pile capacity.

(7) Limited Phase II Environmental Site Assessment depending upon the final selected alternative.

(8) Final Borrow site evaluation and concrete aggregate testing.

I.13.0. REFERENCES

I.13.0. Houston Engineering, Inc. 2008 (April 30). “Project No. 5031-2, City of Fargo Flood Insurance Study, North Side Flood Control Evaluation, Fargo, ND and Moorhead, MN,”, Houston Engineering, Inc., Fargo, ND.

I.13.1. Moore Engineering, Inc; Houston Engineering, Inc; Barr Engineering Company;, 2011 (FEB 2011). “Red River Diversion, Fargo-Moorhead Metro Flood Risk Management Project, Feasibility Study, Phase 4,” Moore Engineering, Inc; Houston Engineering, Inc; Barr Engineering Company; and HDR, Inc., West Fargo, ND.

I.13.2. North Dakota Geological Survey. 2010 (March). “Geologic Investigation No. 94, Earthquakes in North Dakota,” North Dakota Geological Survey, Bismarck, ND.

I.13.3. U.S. Army Corps of Engineers. EM 1110-2-1901. 1993. “Seepage Analysis and Control for Dams,” U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

I.13.4. U.S. Army Corps of Engineers. EM 1110-2-1902. 2003. “Slope Stability,” U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

I.13.5. U.S. Army Corps of Engineers. EM 1110-2-1913, 2000. “Design and Construction of Levees,” U.S. Army Engineering Waterways Experiment Station, Vicksburg, MS.

I.13.6. U.S. Army Corps of Engineers. EM 1110-2-1914, 1992. “Design, Construction, and Maintenance of Relief Wells,” U.S. Army Engineering Waterways Experiment Station, Vicksburg, MS.

I.13.7. U.S. Army Corps of Engineers. ER 1110-2-1806. 1995. “Earthquake Design and Evaluation for Civil Works Projects,” U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

I.13.8. U.S. Army Corps of Engineers. 2007. “Feasibility Study and Environmental Assessment, Fargo, North Dakota, Appendix D, Geotechnical Design and Geology,” St. Paul District, Corps of Engineers, St. Paul, MN.

I.13.9. U.S. Army Corps of Engineers. 1998. “General Reevaluation Report and Environmental Impact Statement, East Grand Forks, Minnesota and Grand Forks, North Dakota, Appendix B, Geotechnical Design and Geology,” St. Paul District, Corps of Engineers, St. Paul, MN.