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General Report: Geotechnical Engineering and Geology

Fargo Moorhead Metropolitan Area
Flood Risk Management Project

North Dakota Diversion Alignment

Engineering and Design Phase

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General Report: Geotechnical Engineering and Geology

1 INTRODUCTION

1.1 PROJECT DESCRIPTION

The Fargo-Moorhead Metropolitan Flood Risk Management (FMM Metro) project is required to reduce the risk of flood for the Fargo-Moorhead Metropolitan (FM Metro) area. In order to accomplish this, a diversion channel will be constructed that will divert flood from the Red River of the North and other tributaries, around the FM Metro area, through a diversion channel located in North Dakota. This was the plan recommended in the feasibility study known as the locally preferred plan (LPP). In addition to the diversion channel, the project requires control structures on the Red River of the North and Wild Rice rivers, hydraulic structures on the Sheyenne and Maple Rivers, an inlet to the diversion, outlet to the Red River, drop structures, fish passages, tie-back embankments, and upstream storage and staging areas.

1.2 PURPOSE

The purpose of the "General Report: Geotechnical Engineering and Geology" is to document geotechnical engineering and geology information that is general to the project. The general report will cover the following items:

- Regional Geology
- Subsurface Investigation
- Laboratory Testing
- Design Parameters
- Analysis Methodology

The geotechnical design and geology for specific reaches and/or features will be presented in supplemental reports associated with the reach/feature.

2 REGIONAL GEOLOGY

2.1 Physiography

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.

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 Wild Rice River of ND, Sheyenne River, Maple River, Lower Rush River, Rush River, and the Buffalo River.

2.2 Topography

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.

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.

2.3 Geology

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 throughout the course of the feasibility study. Additional exploration has been completed along the project alignment in order to more closely define the site specific subsurface conditions.

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-Moorhead. 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:

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

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

2.3.1 Bedrock

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.

2.3.2 Undifferentiated Glacial Sediment

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.

2.3.3 Argusville Formation (Unit A)

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.

2.3.4 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.

2.3.5 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 in areas of rivers. 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.

2.3.6 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. Within the Fargo-Moorhead area the formation is found as two members, The West Fargo member and the Harwood member. 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.

2.3.7 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.

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.

2.3.8 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.

2.4 Structure

Jointing within the glacio-lacustrine deposits at the site has been observed in the soil borings 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.

2.5 Site Hydrogeology

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.

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. 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. Along the river, the water surface profile from the secondary bank riverward varies also, with the flattest profile occurring during the fall/winter months. 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.

In an effort to better understand the groundwater regime, nested vibrating wire piezometers (instruments installed at multiple elevations in one location) were installed at a number of locations in the Fargo-Moorhead area. The piezometers are typically nested together at one location and located in lower, middle and upper elevations and/or sandy layers encountered.

2.6 Seismic Risk and Earthquake History

The FM 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 8), the FM 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 FM metro location indicated is included in Attachment 1.

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 3). Northwest of the FM 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 FM metro 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 1 is a map indicated the “Earthquakes in North Dakota”, and was obtained from the North Dakota Geological Survey, *Geologic Investigation No. 94* (Reference 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 FM metro. 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.

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.

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 1.

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 1. For both scenarios, the probability of a magnitude 5 earthquake or larger is between 0.0 and 0.01.

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 1 along with a map that indicates the locations and magnitudes.

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 FM metro area. Based on this low risk, the performance of project features will not be assessed using earthquake loading cases.

3 SUBSURFACE INVESTIGATION

There has been much subsurface investigation completed by the Corps within the FM metro area. This includes the Red River corridor, the Minnesota Diversion alignment, and the North Dakota. The subsurface investigation was complete during different phases of the feasibility project and PED, using different methods that included machine borings, off-set undisturbed borings, cone penetration testing (CPT) soundings with U2 pore pressure measurements, seismic CPT soundings with U2 pore pressure measurements, and nested vibrating wire piezometers. The historic subsurface investigation completed for the different alternatives during feasibility is summarized below.

3.1 Subsurface Investigation of Alternatives

The number and type of exploratory holes completed for the different alternatives are included below in Table 1. 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.

Table 1: Summary of Subsurface Investigation of Alternatives

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
MN Diversion Channel	43	4	33	5	0
Total	**86**	**11**	**68**	**6**	**2**

The locations of the exploratory holes are shown on the figures presented in Attachment 2. The exploratory holes are numbered in sequential order as they were obtained. Each side of the river has its own sequential order. 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.

The boring log staffs for the alternatives can be found in *Attachment I-4* of the "FMMFS Appendix D: Geotechnical Design and Geology" (Reference 9).

3.2 North Dakota Subsurface Investigation

There has been subsurface investigation for the North Dakota Diversion channel at different times has throughout the course of the project and will continue. The subsurface investigation completed as of 13 May 2013 is summarized below in Table 2 and the locations can be seen on figures in Attachment 3.

Table 2: Summary of Subsurface Investigation for ND Diversion

Phase	Machine Borings	Undisturbed Borings	Undisturbed Borings, Erosion Sample	Auger Boring	CPT	Piezometer	Vane Shear	Pressuremeter
Feasibility Phase 3	24	5	0	0	0	2	0	0
Feasibility Phase 4	17	5	0	0	36	0	0	0
PED, Phase 1	55	13	2	2	0	9	2	5
PED, Phase 1 Topsoil	0	0	0	21	0	0	0	0
West Fargo Erosion	0	0	9	0	0	0	0	0
Wild Rice River Mitigation	3	0	0	0	0	0	0	0
Total	**99**	**23**	**11**	**23**	**36**	**11**	**2**	**5**

3.3 Borings

The machine borings were conducted using various drilling rigs: all-terrain vehicle (ATV) rig, track mounted rig, and truck mounted rig.

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” ID x2 ½” OD 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. No corrections were completed for the blow counts. Disturbed samples were also collected and tested for moisture content, Atterberg limits, and in some cases, grain size distribution.

The undisturbed soil borings are 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.

3.4 Cone Penetration Tests

The CPT method of soil exploration was implemented during the first and fourth exploration programs of the feasibility study. CPT soundings were first conducted in the Spring/Summer 2009 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 between November 2010 through January 2011. During this time, 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.

There were a number of CPT soundings were conducted off-set from machine boring 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.

The seismic cone was implemented in order to obtain the shear wave velocity of the soils in an attempt to be help distinguish between the Brenna and Argusville formations. It was found during the feasibility study that typically the shear wave velocity was higher in the Argusville formation than the Brenna formation. Using this method, the Brenna/Argusville contact can only be estimated and could be off as much as ten feet.

3.4.1 Determination of N_{kt}

Undrained shear strengths of clays can be estimated from CPT sounding results. A common method to estimate undrained shear strength is to relate it to the tip resistance using a correlation factor. There are other expressions using either excess pore pressure or sleeve resistance that can also be used. The correlation values used in the expressions will vary depending on location. It is therefore necessary to determine the appropriate correlation value for each project. This is done by comparing the estimated undrained shear strength to laboratory results.

For the FMM project, thirteen undisturbed borings were taken adjacent to CPT sounding locations, eight of them having unconsolidated, undrained (UU) triaxial tests run an undisturbed samples. The number of samples tested per boring varied with two borings having only one UU test while the others had 3 to 6 samples tested. The peak undrained shear strength values were plotted against the estimated the undrained shear strength. The correlation factor was adjusted until there is a good agreement.

For the FMM project, the undrained shear strength correlation using tip resistance was selected. The expression is:

$$S_u = \frac{q_t - \sigma_{vo}}{N_{kt}}$$

Where,

S_u = undrained shear strength

q_t = tip resistance

σ_{vo} = total vertical overburden stress

N_{kt} = bearing factor

The N_{kt} factor was adjusted until the CPT undrained shear strength was in good agreement with the peak undrained shear strength results from UU tests. It was found that there was a wide range of N_{kt} values, from 14 to 20. A summary of the comparison is below in Table 3. It was found that using a N_{kt} values of either 18 or 19, the CPT undrained shear strength best fit the UU test results. Based on this evaluation, a N_{kt} value of 18 can generally be used for the FMM project. It is recommended that if CPTs are used in design, that the designer review the CPTs and available laboratory tests results to determine if it is necessary to adjust the N_{kt} value for designing a specific feature. It is recommended that the N_{kt} value be reevaluated as additional correlation data becomes available.

Table 3: Summary of Comparison of CPT Undrained Shear Strength to UU Tests

CPT	Number of Tests	N_{kt}	Undisturbed Correlation	Other Undisturbed Testing	Notes
Fargo 09-23C	**5**	**18**	Fargo 09-23MU		Nkt of 18 good match to the UU data
Fargo 09-25C			Fargo 09-25MU		
Fargo 09-26C	1	14	Fargo 09-26MU		Nkt of 14 matchs to one point of UU series as UU series has wide spread
Fargo 09-27C	1	20	Fargo 09-27MU		Nkt of 20 trend matches the one UU in Brenna;
Fargo 10-110C	**6**	**18**	Fargo 11-110MU	10-80MU	Nkt of 18 splits the UU results; Upper material, Nkt = 22; Lower material, Nkt = 16
Fargo 11-118C	**5**	**19**	Fargo 11-118MU	09-59MU & 10-78MU	Nkt of 19 fairly good match to most of the UU results
Fargo 11-119C	**3**	**16**	Fargo 11-119MU	10-79MU & 11-107MU (MN)	Nkt of 16 good match for upper material; Nkt of 13 good match for the lowest Argusville UU
Moor 09-11C			Moor 09-11MU		
Moor 09-14C			Moor 09-14MU		
Moor 09-25C	**5**	**19**	Moor 09-25MU		Nkt of 19 good match to the UU data
Moor 09-34C			Moor 09-34MU		
Moor 09-53C			Moor 09-53MU		
Moor 11-107C	**4**	**14**	Moor 11-107MU	11-119MU & 10-79MU	Nkt of 14 matchs most of UU data; Nkt of 12 matches the top Brenna UU test

Plots of the undrained shear strength and the undisturbed test results can be found in Attachment 10.

4 GROUND WATER CONSIDERATIONS

Soil borings have been conducted to delineate the stratigraphy, and for conducting laboratory testing of the soils necessary to define the physical parameters of the subsurface geology. For the feasibility study vibrating wire piezometers with automated data-loggers were installed straddling the proposed alignments east of Dilworth, MN (FCP) and west of Fargo, ND (ND35K and LPP). Many piezometers from the feasibility phase are still functional and continue to provide valuable data. Since the conclusion of the feasibility study, additional piezometers have been installed along the selected alignment (LPP). Piezometers are used to record subsurface groundwater levels, and this information is used to better understand the groundwater regime in the vicinity of the selected diversion alignment. The piezometers are located in lower, middle and upper elevations and/or sandy layers encountered to further understand the ground water regime. Nested piezometers (instruments installed at multiple elevations in one location) with data-loggers may also be placed at proposed structure locations as the design phase progresses.

Along the selected alignment, the West Fargo Aquifer is the primary water source of interest. 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 square miles. Typically the aquifer is overlain by deposits of glacial till and glacio-lacustrine 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.

Recharge to the West Fargo Aquifer occurs primarily through lateral movement of water through the till and associated deposits and with minor downward percolation of shallow groundwater through the glacio-lacustrine deposits. Due to the low permeability 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 down west to east.

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.

Throughout the Fargo-Moorhead area there are shallow pervious features that may act as small aquifers. A shallow channel-like formation has been identified crossing the proposed alignment approximately 750 ft north of the Maple River. Based on current information the formation is about 500 ft wide and consists of clays and silts in the upper 15 ft and poorly graded sands below that to a depth of approximately 40 ft. The formation is underlain by lacustrine clays.

Other, unnamed aquifers may occur at various depths within the tills and glacio-lacustrine clays adjacent to the proposed diversion alignment. 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.

4.1 Shallow Groundwater

The shallow groundwater table is defined as the locally observed groundwater table near the ground surface; typically located within the first 20 feet below the ground surface. Piezometers installed along the proposed alignment in August 2012 indicate varying degrees of downward flow towards the till and outwash deposits. Piezometers far from the Red River show significant decrease in the piezometric level with depth, while a set of piezometers near the proposed outlet indicates nearly hydrostatic conditions.

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 occurs 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.

Under the conditions reasonably anticipated, the flow of the shallow groundwater should be “downhill” or toward the area of lower hydraulic potential. Existing examples of this type of feature are the Red River of the North and its local tributaries, the Maple River and the Sheyenne River. After the excavation of a diversion channel is completed, the diversion will act as another area of low hydraulic potential. This may lower the groundwater table near the diversion channel, but not below 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 excavated material berms (EMBs). 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.

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. One possible exception to this generalization is where the alignment crosses a shallow channel-like sand formation north of the Maple River. Here, water levels within the sandy formation could be reduced some greater distance away from the diversion due to the relatively pervious nature of the sandy soils.

The lowering of the shallow groundwater table may cause consolidation of the surrounding soils and settlement of structures within the affected area. 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 EMBs 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.

4.2 Aquifers

For the purposes of this report, aquifers in the study area are defined as pervious, water-bearing geological formations that are located at depth and covered by relatively impermeable formations. Aquifers may provide a major source of water and are assumed to have some amount of artesian pressures. The major aquifer in the study area is the West Fargo Aquifer in North Dakota. The West Fargo Aquifer is relatively limited in aerial extent and is deep compared to the proposed channel. The aquifer will be separated from the channel by significant lacustrine deposits that will tend to act as a buffer between the channel and the aquifer. For these reasons measurable impacts are considered unlikely.

The first potential effect that construction of a diversion channel could have is the lowering of the artesian pressures in the aquifer. With the construction of the diversion channel, the seepage path length from the aquifer to the ground surface could be reduced approximately by the depth of the channel excavation. In the case of water flowing upwards out of the aquifer, this reduced seepage path length and creation of a lower potential area may increase the upward flow. If the quantity of flow out of an aquifer is greater than the quantity of flow recharging the system, the artesian pressures can be reduced. The total volume of water available at a given time in the aquifer may be negatively impacted as well. The result of the lowered artesian pressures would be that more pumping would be necessary for private and municipal use. However, as stated previously, piezometer measurements along the diversion alignment currently indicate downward groundwater flow from from the lacustrine soils to the glacial till and drift. In this case the impact of the channel on any aquifer at depth would be negligible or positive since the lacustrine soils appear to act as a recharge zone.

The second potential effect of a diversion channel is that contamination of the aquifer could occur. The diversion channel excavation would reduce the length that contaminants would have to travel from the ground surface to the aquifer. However an aquifer under artesian pressure will have a positive pressure "outward" toward the diversion channel making it very difficult for potential contaminants to "migrate" against this pressure (away from the diversion) and towards the main portion of the aquifer. In the unlikely event of contamination the use of the aquifer as a source of water for domestic use could be restricted.

Due to the relatively impervious nature of the subsurface materials likely to be encountered along the proposed alignment, the flow of water from the aquifer due to artesian pressures and the migration of contaminates into the aquifer are minimized. The diversion channel would present no greater risk of contamination of the West Fargo Aquifer than any other existing natural or manmade channel in the Fargo area.

There are two mitigation/adaptive management measures that could be taken to reduce the risk of long term changes to the aquifer as a result of the proposed diversion. The first adaptive management measure would be to monitor the aquifer and the areas surrounding the diversion channel to see if the artesian pressures are being lowered after excavation of the diversion channel and what direction the water is flowing. Piezometers that have been installed along the alignment as part of the project design effort will remain in place in order to measure any impact of the channel on shallow groundwater behavior. If an impact to the aquifer was detected the second mitigation measure would be to place a more impervious buffer between the aquifer and the channel excavation to minimize the flow of water into the diversion channel or contaminants into the aquifer. If a pervious material was encountered during channel excavation, over-excavation of this material could be required and impervious fill placed to provide this buffer.

Finally, there are some smaller scale sand and gravel beds that could be considered localized aquifers (groundwater instead of buried aquifers) if the beds are extensive enough to provide potable water for a residence or farmstead. The existence of these smaller localized aquifers needs to be matched with existing water wells to prevent or compensate for the loss of individual water wells along the alignment.

5 INSTRUMENTATION

Instrumentation has been installed throughout the project area in order to better understand the groundwater regime within the project limits. Instrumentation was installed during Phase 2 and Phase 3 of the feasibility study and during the initial stages for Preconstruction, Engineering and Design (PED). This instrumentation consisted of nested sets of vibratory wire piezometers, where multiple piezometers are installed within the same boring but at different elevations. The piezometers are typically nested together and located in lower, middle and upper elevations and/or sandy layers encountered The purpose of the nested vibratory wire piezometers is to determine the groundwater table and piezometric levels in the different foundation materials.

During the feasibility study, 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. During the initial stages of PED, nine nested sets of vibratory wire piezometers were installed along the ND Diversion alignment, in areas of interest. Data is no longer being collected at instrumentation locations along the Buffalo Aquifer. Data is still being collected at the instrumentation locations in Minnesota that are located adjacent to the Red River.

A summary of the piezometers installed and their locations is included in Table 4 and Table 5 below. These locations are illustrated by the figures in Attachment 4. The observed piezometric levels are illustrated by the figures in Attachment 4.

Table 4: Summary of Instrumentation Locations and Data for the MN Diversion

Instrumentation for MN Diversion							
Piezmeters	**Location**	**Boring**	**Project Phase**	**Install Date (Discontinued)**	**Ground Surface Elevation**	**Depth**	**Elevation**
P1	Gooseberry Park, Moorhead	Moorhead 09-21M/P	Feasibilty Study	8/28/2009	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	Feasibilty Study	8/28/2009 Discontinued **12/18/2012**	913.1	20	893.1
P5						50	863.1
P6						75	838.1
P7	MN Diversion Red River Control Structure	Moorhead 10-81M/P	Feasibilty Study	5/21/2010	907.77	30	877.77
P8						60	847.77
P9						90	817.77
P10	93rd Ave N & 60th St N	Moorhead 10-88M/P	Feasibilty Study	6/12/2010 Discontinued **12/18/2012**	911.266	35	876.266
P11						70	841.266
P12						105	806.266
P19	MN Diversion, HWY 10	Moorhead 10-106M/P	Feasibilty Study	7/17/2010 Discontinued **12/18/2012**	913.29	19.39	893.9
P20						64.39	848.9
P21						81.39	831.9
P22						101.4	811.9
P23	Buffalo Aquifer, 28th Ave N	Moorhead 10-105M/P	Feasibilty Study	7/19/2010 Discontinued **11/19/2010**	916.61	43	873.61

Table 5: Summary of Instrumentation Locations and Data for the ND Diversion

Instrumentation for ND Diversion							
Piezmeters	**Location**	**Boring**	**Project Phase**	**Install Date**	**Ground Surface Elevation**	**Depth**	**Elevation**
P13	Wild Rice River, Brodshaug Farmyard	Fargo 10-77M/P	Feasibilty Study	6/24/2012	905.68	30	875.68
P14						50	855.68
P15						100	805.68
P16	ND Diversion Red River Control Structure	Fargo 10-79M/P	Feasibilty Study	6/28/2010	905.252	80	825.252
P17						20	885.252
P18						40	865.252
P24	Maple River (north) - 26th St NW & Cass Co. HWY 20 (2012 Loc #6)	Fargo 12-190M/P	PED	8/22/2012	899.2	12.6	886.6
P25						27.6	871.6
P26						54.6	844.6
P27						75.6	823.6
P28	Sheyenne River (south) - 169th Ave SE & 48th St SE (2012 Loc #8)	Fargo 12-191M/P	PED	8/22/2012	918.5	20.5	898
P29						46.5	872
P30						81.5	837
P31						121	797.5
P32	Drain 14 & Cass Co. HWY 10 (2012 Loc #5)	Fargo 12-192M/P	PED	8/22/2013	900.5	19.7	880.8
P33						29.7	870.8
P34						51.7	848.8
P35						76.7	823.8
P36	V. Johnson Property - Reach 1 (2012 Loc #1)	Fargo 12-193M/P	PED	8/24/2012	880.6	33.2	847.4
P37						60.2	820.4
P38						80.2	800.4
P39						91.7	788.9
P40	173rd Ave SE - Reach 1 (2012 Loc #2B)	Fargo 13-202M/P	PED	2/16/2013	880.5	31.6	848.9
P41						61.6	818.9
P42						81.6	798.9
P43						101.6	778.9
P44	172nd Ave SE & 25th St SE - Reach 1 (2012 Loc #3D)	Fargo 13-203M/P	PED	2/22/2013	885.1	29.4	855.7
P45						59.4	825.7
P46						79.4	805.7
P47						99.4	785.7
P48	28th St SE - Reach 4 (2012 Loc #4B)	Fargo 13-204M/P	PED	2/25/2013	889.7	29.7	860
P49						59.7	830
P50						79.7	810
P51	Sheyenne River - 81st St S & 47th St SE (2012 Loc #7)	Fargo 13-205M/P	PED	2/27/2013	917.6	29.6	888
P52						59.6	858
P53						79.6	838
P54						119.6	798
P55	Horace to West Fargo Diversion - 26th St W & 40th Ave W (2012 Loc #9B)	Fargo 13-206M/P	PED	2/28/2013	905	30.7	874.3
P56						50.7	854.3
P57						65.7	839.3
P58						85.7	819.3

6 DESIGN PARAMETERS

6.1 Jar Sample Testing

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.

6.2 Undisturbed Testing

There was much undisturbed testing completed at locations throughout the entire FM metro area during the feasibility study. Additional undisturbed testing is expected to be gathered along the North Dakota diversion alignment during PED. Due to the fact that the geology of the FM metro area has been influenced by the legacy of glacial Lake Agassiz, there is not much variation found due to spatial concerns. For each formation, the spatial variability is within reason. Therefore it is prudent that all test data be used when developing design parameters.

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 6 shows the undisturbed testing request for the different phases of the project while Table 7 shows testing completed on each soil unit, both tables updated as of 13 May 2013 . The results of the laboratory test completed during feasibility can be found in "Attachment I-04" of *Appendix I, Geotechnical Design and Geology* of the *Fargo-Moorhead Metropolitan Area Flood Risk Management Final Feasibility Report and Environmental Impact Statement.* The laboratory tests completed during PED are presented in Attachment 5.

Table 6: Summary of Undisturbed Testing By Phase

	Feasibility Study			PED	
Test	Phase 2	Phase 3	Phase 4	Phase 1	Total
DS	13	4	0	7	24
Residual	12	0	0	0	12
UU, Undisturbed	12	12	24	27	75
R-bar, Undisturbed	28	12	25	18	83
Consolidation	10	5	19	8	42
Constant Rate of Strain Consolidation	0	7	2	8	17
Atterberg Limits	30	15	26	36	107
Spec. Gravity	5	0	26	22	53
Hydro & Sieve	10	13	26	24	73
Sample Extrusion	38	15	26	30	109
Proctor	0	0	0	10	10
R-bar, Remolded	0	0	0	2	2
UU, Remolded	0	0	0	2	2
Fully Softened	0	0	0	8	8

Table 7: 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	Proctor	R-bar, Disturbed	UU, disturbed	Fully Softened
Alluvium	0	2	7	7	3	2	10	1	3	4	0	0	0
Sherack	0	3	6	11	2	2	12	5	4	2	0	0	2
PL Sherack	0	0	0	0	0	0	0	0	0	0	0	0	0
Poplar River	0	0	0	1	0	0	1	0	0	0	0	0	0
PR - Harwood	0	0	2	2	1	0	2	0	2	0	0	0	0
PR - WF	1	0	2	2	1	0	2	0	2	0	0	0	0
OX Brenna	0	1	8	10	6	0	13	5	20	0	0	0	4
Brenna	11	3	21	25	13	6	30	14	9	4	2	2	0
B/A Transition	1	1	6	7	3	0	7	5	6	0	0	0	0
Argusville	8	2	17	13	11	4	21	15	18	0	0	0	2
Till	0	0	2	4	1	0	5	4	5	0	0	0	0
Not Distinguished	3	0	4	1	1	3	4	4	4	0	0	0	0
TOTAL	**24**	**12**	**75**	**83**	**42**	**17**	**107**	**53**	**73**	**10**	**2**	**2**	**8**

6.3 Selection of Design Shear Strength Parameters

The effective shear strength parameters used for the FMMFS are based on the ultimate (post-peak) strength failure criteria that equated to a strain of 15%. There are a number of reasons for this. First, ultimate strengths have been used for previous St. Paul District (MVP) projects within the Red River Valley. In addition, experience within the Red River Valley indicates that clays within this region are fissured and the weakest of these clays exhibit brittle stress-strain behavior. This can lead to progressive failure of the riverbanks and cut slopes, which is commonly seen. As a result of the brittle stress-strain behavior and progressive failure mechanism, the peak shear strength cannot be mobilized along the potential shear surfaces simultaneously. Also, experience indicates that large amount of strain (more than 10%) may occur in natural or cut slopes during the life time of the project. The effective stress shear strength test data indicates that if the materials exhibit brittle stress-strain response, the peak strength occurs typically between 3 and 8 percent strain. For those materials that do not exhibit a brittle stress-strain response, the maximum stress typically remains constant beyond 10%

strain For these reasons, the effective stress shear strength parameters are based on the ultimate (post-peak) strength failure. Both R-bar and DS test results were used in the determination of the effective stress shear strength parameters.

In the case of the total stress analyses, ultimate undrained shear strength parameters are used. There are a few reasons why the use of ultimate, undrained shear strength parameters are used: 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) 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. The selection of the undrained shear strength (c_u) was based on the results of the Q tests.

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.

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. In the case of the excavated material berm and semi-compacted excavated material, the unit weights were based on compacted unit weights obtained during the construction of the Horace to West Fargo and West Fargo Diversions. For the excavated material berm, the average value plus one standard deviation was selected. For the semi-compacted material, the value associated with a cumulative probability/frequency of 94% was selected.

The selected design parameters can be found in Table 8 and the data points for the curvilinear shear strength parameters in Table 9. In addition, the Mohr-Coulomb and curvilinear shear strength plots and undrained strength versus elevation plots are presented in Attachment 6.

Table 8: Summary of Selected Soil Parameters

Formation	Unit Weight (1) γ_{sat} (pcf)	Shear Strength Parameters: Effective Stress (2) c' (psf)	Effective Stress (2) ϕ'	Total Stress, c (psf) Ultimate (3)	Total Stress, c (psf) Peak (4)	Residual $\phi'_{residual}$
Alluvium	**120**	0	**31**	assume values of Sherack		20
Sherack	**115**	bi-linear curve, see note (5)		**900**	1400	13.0
Poplar River - West Fargo	**123**	0	**34**	**1900**	1900	25
Poplar River - Harwood	**116**	0	**26**	**1200**	1450	assume values of West Fargo
Poplar River, All	**119**	assume values of Harwood		assume values of Harwood		assume values of West Fargo
Oxidized Brenna	**108**	see curvilinear envelope (6)		**900**	1000	5.5
Brenna	**106**	see curvilinear envelope (6)		**575**	650	9.0
Argusville	**110**	see curvilinear envelope (6)		see note (7)	825	10.5
Unit "A" Till (8)	**123**	**225**	**22**	**1900**	2200	N/A
Semi-Compacted Excavated Material	**123 (9)**	**50 (10)**	**14 (10)**	**600 (10)**	N/A	N/A
Excavated Material Berm	**121 (9)**	**50 (10)**	**14 (10)**	**600 (10)**	N/A	N/A
Levee Fill						
Sand (11)	**125**	0	**32**	N/A		N/A
Riprap (11)	**125**	0	**30**	N/A		N/A

Notes:

(1) The unit weights are taken as the average value of all the laboratory test results received during the Fargo-Moorhead feasibility study.

(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% axial strain.

(3) 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.

(4) 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 are not used in the end-of-construction stability analysis, but are presented such that strain-softening behavior can be inferred.

(5) The ultimate bi-linear effective stress curve for the Sherack is $\phi' = 28^0$, c' = 0 psf and than at 2000

(6) 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 envelope is one standard deviation less than the most likely value envelope.

(7) The Argusville formation ultimate undrained shear strength was assumed to be linearly increasing with depth. Initial cohesion was assumed to be 575 psf, with an increase of 10 psf/FT

(8)The Unit "A" Till parameters are based on limited samples as undisturbed samples are difficult to obtain. These parameters are only valid for slope stability analysis; for foundation design, parameters should be determined using local knowledge and/or in situ testing results.

(9) Values based on compacted unit weights obtained during the construction of the Horace to West Fargo and West Fargo Diversion channels.

(10) Values for Semi-Compacted Excavated Material and Excavated Material are similar to the in-situ strength of thr Brenna formation as the excavted materal will be placed randomly without very much compactive effort.

(11) Assumed values based judgment.

Table 9: 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
				8000	1740

6.3.1 Fully Softened Shear Strength Testing

Fully softened shear strength (FSS) testing was conducted on seven undisturbed samples having various liquid limits and for various formations. The purpose was to compare the fully softened shear strength results to the ultimate shear strength values from R-bar and DS tests. It was found that the results of the FSS testing were comparable to the ultimate shear strength results from R-bar and DS tests. In some instances the FSS results indicated higher shear strengths than the intact samples. From this it was believed that the drained shear strength envelopes developed from R-bar and DS tests are adequate for use in the FMM project. Details of the FSS testing can be found in Attachment 9.

6.4 Selection of Design Permeability Parameters

During the feasibility study, permeabilities for the different formations were back-calculated from consolidation test results using the "log of time" method presented in EM 1110-2-1901. The results of the back-calculations indicated that the permeabilities of all the formations were similar. Using engineering judgment and typical range of fines in the formations, vertical permeabilities were selected. The Brenna and Argusville formations contain very high percentages of fines, therefore a k_y value of $1.0\text{x}10^{-7}$ cm/s was assumed. The Alluvium and Sherack materials have a lower proportion of fines than the Brenna and Argusville, so a k_y value that was one order of magnitude greater than the former was assumed. The Unit "A" till formation is typically coarser than the Lake Agassiz sediments, therefore a permeability value half an order of magnitude greater than the Alluvium/Sherack formations was assumed. The vertical to horizontal permeability ratio (k_y/k_x) of all soils except the till was set to unity as a simplifying assumption. This was based on the fact that for the most part the soils are lake deposits and do not have lenses of coarser materials. For the till, a ratio of $k_y/k_x = 0.25$ was assumed.

During Phase 4 of the Feasibility Study, 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.

At the beginning of the PED phase, a seepage calibration was performed in order to support the assumptions made during feasibility. The seepage calibration was conducted at locations where vibrating wire piezometers were installed adjacent to the Red River of the North as part of the feasibility study. A discussion of the seepage calibration modeling and results are presented in Attachment 7. From this seepage calibration the required seepage parameters were determined. The required parameters needed for the seepage analysis are summarized below in Table 10.

Table 10: Summary of Selected Permeability Parameters

Material	Material Model Type (1)	Sample Material (2)	Vertical Permeability		Method of Selection (3)	Horizontal Permeability			Volumetric Water Content (4)	M_v (5)	Residual Water Content (6)
			k_y (cm/sec)	k_y (ft/day)		k_y/k_x ratio	k_x (ft/day)	k_x (cm/sec)	(ft3/ft3)	(1/psf)	(ft3/ft3)
Alluvium	Sat / Unsaturated	Silty Clay	**1.0E-06**	2.8E-03	**Calibration**	**0.25**	0.0113	4.00E-06	0.5	9.0E-06	0.050
Sherack	Sat / Unsaturated	Silty Clay	**1.0E-06**	2.8E-03	**Calibration**	**0.25**	0.0113	4.00E-06	0.5	9.0E-06	0.050
PL Sherack	Sat / Unsaturated	Silty Clay	1.0E-04	2.8E-01	Judgement	1	0.28	1.00E-04	0.5	9.0E-06	0.050
West Fargo	Sat / Unsaturated	Silt	1.0E-04	2.8E-01	Judgement	1	0.28	1.00E-04	0.4	3.0E-06	0.040
Harwood	Sat / Unsaturated	Silt	1.0E-05	2.8E-02	Judgement	1	0.028	1.00E-05	0.5	9.0E-06	0.050
OX Brenna	Sat / Unsaturated	Silty Clay	**5.0E-07**	1.4E-03	**Calibration**	**1**	0.0014	5.00E-07	0.55	1.0E-05	0.055
Brenna	Saturated Only	N/A	**1.0E-07**	2.8E-04	**Calibration**	**1**	0.00028	1.00E-07	0.63	3.0E-05	0.063
Argusville	Saturated Only	N/A	**1.0E-07**	2.8E-04	**Calibration**	**1**	0.00028	1.00E-07	0.6	3.0E-05	0.060
Unit "A" Till	Saturated Only	N/A	**5.0E-06**	1.4E-02	**Calibration**	**0.25**	0.057	2.00E-05	0.45	3.0E-05	0.045
Silts	Saturated Only	N/A	1.0E-06	2.8E-03	Judgement	1	0.0028	1.00E-06	0.4	3.0E-06	0.040
Silty Sands	Saturated Only	N/A	1.0E-04	2.8E-01	Judgement	1	0.28	1.00E-04	0.4	3.0E-06	0.040
Sand	Sat / Unsaturated	Fine Sand	1.0E-02	2.8E+01	Judgement	1	28	1.00E-02	0.4	3.0E-05	0.040

Notes:
(1) Indicates how the material was model in Seep/W. If material is exoected to be above 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) The selection of the vertical permeability was based on engineering judgement and back-calculated permeability values from consolidation tests. Seepage calibration was completed to validate and refine certain formations.
(4) Volumetric Water Content Based on Porosity taken from testing except for PL Sherack, Sand, and Till which are estimated values.
(5) M_v for Alluvium, Sherack, OX Brenna, and Brenna based on consolidation data. All other materials estimated.
(6) The residual water conent was estimated to be 10% of the saturated water content.

7 ANALYSIS METHODS

There are many features that need to be designed for the project. There will be many different entities designing these features. To maintain consistency throughout the project, similar features must be designed using similar methodology. The methodology for analyzing the different features have been or will be developed. The following sections summarize the methodology to follow when analyzing the different features of the project.

7.1 Diversion Channel

The analysis of the diversion channel will follow the guidance outlined in the Memorandum for Record (MFR) "*Fargo-Moorhead Metro Flood Risk Management Project – MFR-002,Diversion Channel and Low-Flow Design*."

7.2 Levees along Diversion Channel

The analysis of the levee portion of the excavated material berms will follow the guidance outlined in the MFR "*Fargo-Moorhead Metro Flood Risk Management Project – MFR-001,Levees and Excavated Material Berms along the Diversion Channel*."

8 REBOUND OF THE CHANNEL DUE TO SOIL EXPANSION

8.1 Introduction

Rebound of the diversion channel is anticipated as a result of the large-scale excavation above highly compressible soils. Unloading due to the excavation will lead to negative pore water pressures beneath the diversion that will dissipate over time as local pore water pressures equilibrate with the larger groundwater regime. The soil structure expands as the pores draw in water, ultimately causing the ground surface within the diversion to rise.

8.2 1-D Ultimate Rebound Calculations

The total amount of rebound can be estimated using 1-D consolidation theory assuming that expansion occurs along the recompression index. While initial "immediate" unloading can be thought of as a change in total stress, soil expansion is governed by the ultimate change in effective stress compared to the initial condition. This is an important distinction to make as groundwater levels have an impact on the change in effective stress. Lower initial groundwater levels will lead to a greater change in effective stress, and therefore greater amount of rebound. For the purposes of estimating rebound groundwater was assumed to be 10 ft below ground surface, as described in Attachment 7.

Analysis of the diversion excavation was performed at a number of locations between the Maple River aqueduct structure and the diversion outlet. The invert of the main channel (as opposed to the low-flow channel) was used as the reference elevation, as the 1-D assumption is less valid for the deeper and narrower low-flow channel. Results indicate that approximately 14 to 20 inches of total rebound can be expected in the channel depending on stratigraphic differences. These calculations are reported in the individual reach DDRs.

8.3 Time Rate of Rebound

In addition to the 1-D ultimate rebound calculations, time-rate of expansion was evaluated for a variety of conditions in order to estimate how much rebound will occur during different periods of the project life. Of primary interest were the amount of rebound expected during construction and the amount of rebound following construction during the first 25 years. Based on the experience of the Horace to West

Fargo and West Fargo Diversions it is anticipated that sediment cleanout may be required within 25 years of operation, during which overexcavation could be performed to account for rebound. It should be noted that hydrologic uncertainty means that cleanout could be required much sooner or much later than 25 years.

Simplified calculations were performed using Terzaghi's theory of one-dimensional consolidation. The time rate of expansion calculations used laboratory test data for the coefficient of consolidation c_v, assuming that the coefficient of expansion would be comparable in magnitude. Figures ( ) and ( ) show coefficient of consolidation test data for the Brenna and Argusville Formations, respectively. The results are summarized in Table 11. The table utilizes data points from the anticipated in-situ stress ranges based on depth below ground surface before and after excavation. As is typical for most soils, laboratory results for c_v showed significant spread, with a definite trend of increasing c_v with decreasing stress. Also, values of c_v were higher for the Argusville Formation and lower for the Brenna formation. For the simplified 1D time rate consolidation computations, a single composite value of c_v was assumed to represent both the Brenna and Argusville formations. For the purposes of determining the composite value the Brenna formation was assumed to be twice as thick as the Argusville formation.

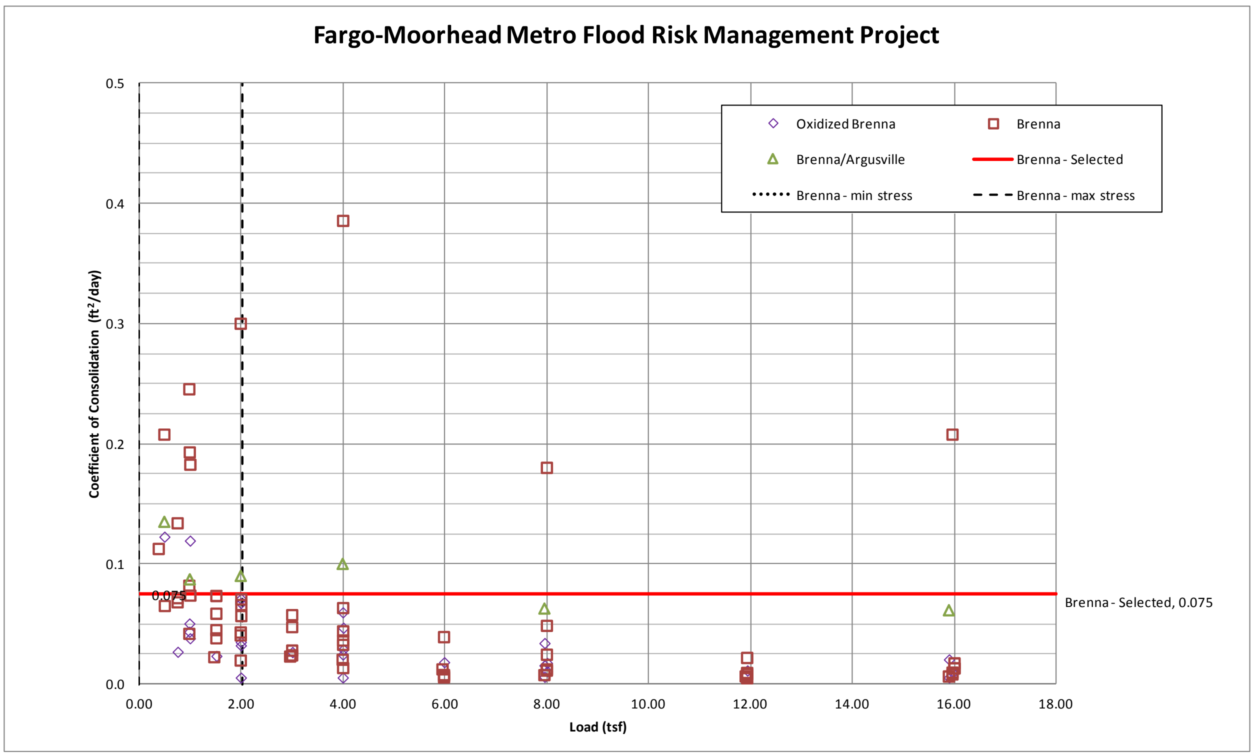

Figure 1: C_v test results for the Brenna Formation, showing range of in-situ stresses and selected value

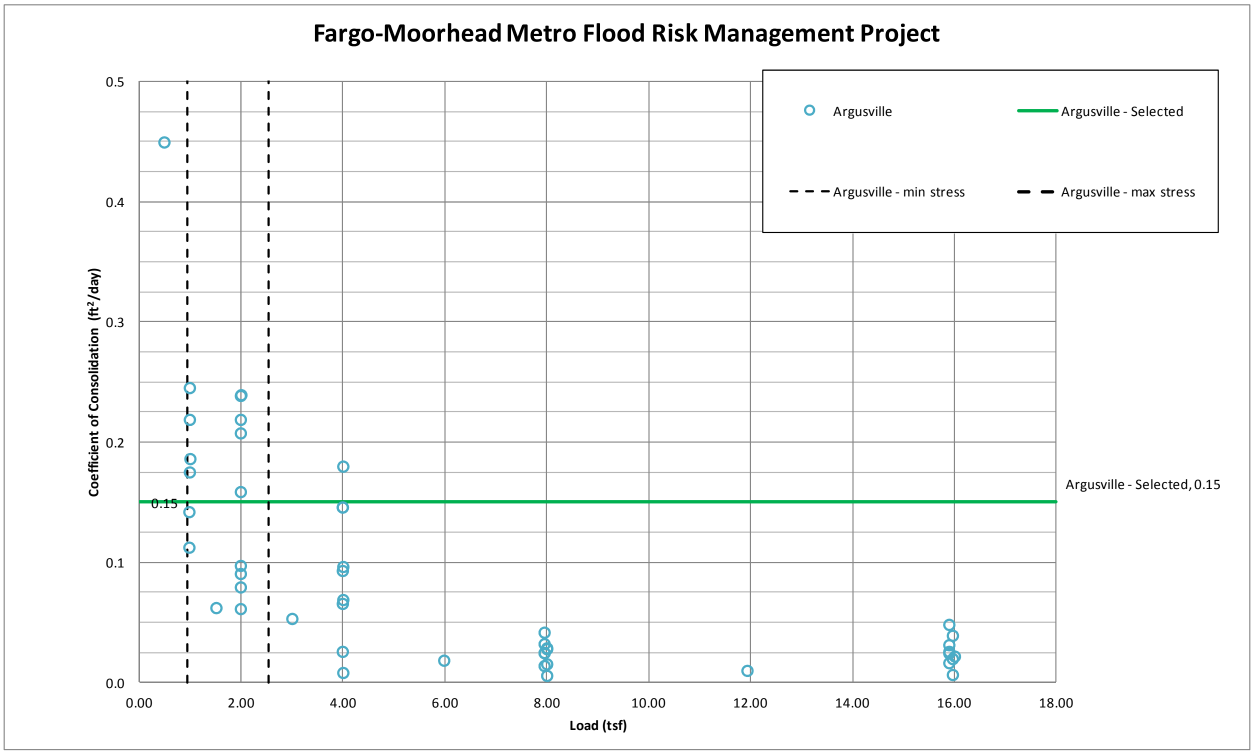

Figure 2: C_v test results for the Argusville Formation, showing range of in-situ stresses and selected value

Table 11: Summary of c_v test data

Formation		Brenna	Argusville
Depth Range (ft)		0 to 75	25 to 95
Stress Range (tsf)		0 to 2.02	0.94 to 2.54
c_v (ft^2/day)	Mean	0.087	0.158
	Median	0.068	0.167
	Minimum	0.005	0.061
	Maximum	0.300	0.245
	Selected	0.075	0.150
	Composite	0.100	

It was assumed that c_v was isotropic. Both the lower boundary with the glacial till and the upper free boundary were treated as free-draining boundaries. It is unclear how the upper boundary affects drainage in the context of rebound – while portions of the low-flow channel will be exposed to water the vast majority of the time, the diversion channel bottom will be dry except during flooding. The analysis assumes that drainage behavior at this boundary is no different from a fully saturated boundary. In reality there will be a partially saturated zone where water may flow much slower than in the fully-saturated zone, but where subaerial exposure and desiccation cracking could result in a greater portion of pore air.

The rate of rebound depends on the thickness of the compressible soils beneath the channel bottom, which varies along the alignment. Near the outlet the thickness is as great as 73 ft, whereas in areas of Reach 5 the thickness beneath the channel is closer to 40 ft.

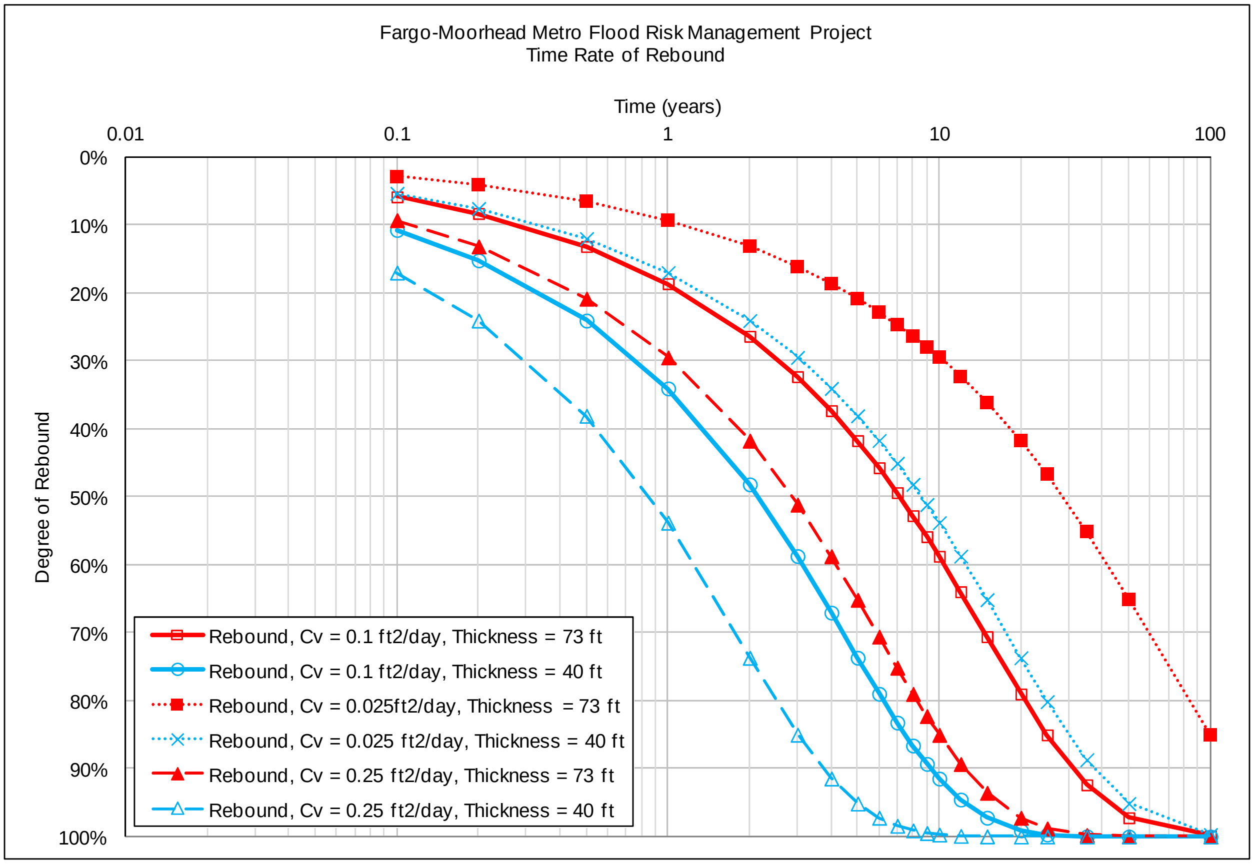

Figure 3: Results of Terzaghi time-rate of rebound analysis

Results from these simplified analyses indicate that there is significant uncertainty regarding time-rate of rebound (Figure 3). Assuming a constant coefficient of consolidation, between 13% and 24% of rebound could be anticipated during construction, and an additional 72% to 76% between end-of-construction and 25 years.

Rebound was also studied in Sigma/W independently from the above analyses, and is reported separately (Attachment 8). The approach to the analysis was not exactly consistent with the simplified analyses but provided some qualitative comparisons. Results indicate that the amount and rate of rebound are highly sensitive to soil hydraulic properties, groundwater table, and the thickness of the rebounding layer. Using the modified Cam-clay constitutive model, Sigma/W indicated lower values for ultimate rebound with a greater proportion of rebound occurring during construction (around 30%).

Rebound has been discussed with the engineers responsible for the Red River Floodway expansion in Winnipeg, Manitoba. There, preliminary calculations suggested that rebound would be negligible, falling within construction tolerances. Final grading checks were typically performed several weeks following final grading, and no significant rebound was measured. It was noted that the lacustrine sediments in Winnipeg are generally not as thick as in Fargo, with 40 ft maximum thickness beneath the channel. In Fargo, lacustrine sediments beneath the channel could be 40-75 ft thick, so performance of the Red River Floodway could be taken to represent a lower bound for rebound. Another factor that could result in possible differences in time-rate of rebound is artesian pressures beneath the clay, which are not present in the Fargo-Moorhead area.

Generally speaking it is believed that rebound has the potential to be a minor O&M concern compared to the more significant issue of sedimentation. While ultimate rebound can be predicted, it is less clear from analyses what proportion of the ultimate rebound will occur during construction and how much will be left to occur during the project life. Time-rate predictions of rebound during construction vary between 13% and upwards of 30% of ultimate, without accounting for variability of soil properties and drainage distances. Observations in geotechnical engineering practice indicate that the rate of consolidation is typically greater than what is predicted with oedometer tests. This is due to the influence of the soil macro-fabric, which cannot be captured in small undisturbed samples. Therefore, it is possible that enough rebound will occur during construction that the remaining amount will be insignificant to the project's functionality. Preliminary hydraulic analysis indicates that assuming 75% of the predicted 1-D rebound happens in the period between end-of-construction and 25 years, it is unlikely that rebound will have a significant impact on the hydraulic performance of the diversion. Field measurements will be obtained during and after construction of the first reaches in order to verify these preliminary conclusions. However, without additional field verification of rebound behavior there is insufficient information to justify channel overexcavation in order to account for rebound during construction.

9 REFERENCES

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10 ATTACHMENTS

Attachment 1: Seismic Information

Attachment 2: Soil Exploration Maps for Feasibility Alternatives

Attachment 3: Soil Exploration Maps for North Dakota Diversion

Attachment 4: Instrumentation

Attachment 5: PED Undisturbed Testing Results

Attachment 6: Selected Parameters

Attachment 7: Seepage Calibration Report

Attachment 8: Numerical Modeling Report

Attachment 9: Fully Softened Shear Strength Testing Report

Attachment 10: CPT Undrained Shear Strength and N_{kt} Values