Appendix H
Geotechnical Analysis: Credit to Existing Levees

Fargo-Moorhead Metropolitan Area Flood Risk Management

Final Feasibility Report
and Environmental Impact Statement

July 2011

US Army Corps
of Engineers ®

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Fargo – Moorhead Metro Feasibility Study

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Appendix H

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Appendix H:

Geotechnical Analysis: Credit to Existing Levees

PURPOSE

1. Both the communities of Fargo, ND and Moorhead, MN have constructed levees that are protecting areas of their cities from the Red River of the North. These levees have been design and constructed by the communities and are not a part of the Corps' federal or non-federal levee programs. An evaluation is necessary to determine the reliability of the existing levees as required by Policy Guidance Letter No. 26, *Benefit Determination Involving Existing Levees*, dated 23 December 1991 (Reference 1).

2. The purpose of this is report is to summarize the geotechnical analysis that was completed in associated with the "credit to existing levees" study. A reliability analysis was completed to determine the probability of failure of the existing levees as a function of the floodwater elevation. The USACE ETL 1110-2-556, *Risked-Based Analysis on Geotechnical Engineering for Support of Planning Studies*, dated 28 May 1999 (Reference 2), was used a guidance. This guidance expired on 30 June 2004 and was rescinded on 4 MAR 2009. The "credit to existing levees" analysis was completed prior to the ETL being rescinded. There is no other guidance that has replaced this ETL and due to this fact it is still the most appropriate document to use as guidance.

EXISTING LEVEES

3. The levees constructed by the communities of Fargo, ND and Moorhead, MN vary in height, top width, and side slopes and are scattered throughout the metro area. The locations of the existing levees can be seen in Attachment H-1.

4. These levees were inspected during a site visit on 8 October 2008 and many again were observed during a site visit between 19 and 20 November 2008. In general, the sod cover / turf was well established on the levee. In some instances, trees and shrubs were growing at and/or beyond the toe of the levees. In a couple instances, trees and shrubs were growing on the levee. Structures, such as homes and apartment/condo buildings were adjacent to the levees. The purpose of the inspection was to assess the current conditions of the levee and not to evaluate if the levees conformed to current USACE's guidelines. Photos of the levees can be seen in Attachment H-2.

5. The project documents for the existing levees indicate that the levees were generally constructed and/or modified between 1997 and 2003; the North Broadway Addition levee reach was constructed as early as 1990 while the Southwood Drive Levee reach was constructed as recently as 2007. It is unknown exactly where the fill material was obtained for the construction of the existing levees, but clay material is readily available in the area and it is highly likely that the levees are constructed of this clay material. The exact construction method of the levees is also unknown, but the general practice is to place fill materials in lifts using a dozer and compact it either with the dozer or using a sheepsfoot or padfoot roller. It is not likely that an inspection

trench was excavated beneath the centerline of the existing levees, which is standard practice for permanent levees that are designed and constructed by the Corps' St. Paul District.

6. Most of these levees do not afford a continuous line of protection. These discontinuities range from being low spots in the existing levee, openings in the levee to maintain day-to-day traffic, or the fact that the levee does not tie into high ground. These discontinuities require that emergency measures (i.e. emergency levees, sandbags, or HESCO Bastion Concertainers) be constructed / placed to provide a continuous line of protection to the areas behind the existing levees. A summary of the elevation in which the discontinuities occur can be found in Table 1. All elevations are presented in North American Vertical Datum 1988 (NAVD 88). Some of the project as-built drawings use different vertical datums, but were converted to NAVD 88 for reporting purposes.

Table 1: Summary of Discontinuity Elevations

City	Project ID	Location	Top of Levee	Highest Flood Level Protection	Notes
Fargo	5078-2	10th St. N	898.5	896	Based on Broadway 1st Addition As-Builts that indicate top of levee at 896 FT.
Fargo	4903	Cardinal Muench Seminary	898	897.3	Based on levee as-builts that indicate top of levee at 897.3 FT
Fargo	4579-3	10th Ave S (Dike West)	905	904	Based on levee as-builts that indicate top of levee at 904 FT
Fargo	4908	Lindenwood Drive	905	901.5	Based on contours that indicate discontinuity (driveway) between 901 to 902 and ties into ground between 901 to 902 FT.
Fargo	4980	Southwood Drive	905	903.5	Based on contours that indicate discontinuity and lowest top of levee between 903 to 904 FT.
Fargo	5093	Drain 27 / Rose Coulee	909	906.3	Based on as-builts of area projects that indicate lowest top of levee at 906.3 FT.
Moorhead	97-13-15	Woodlawn Park	904	904	Based on levee as-builts that indicate top of levee at 904 FT.
Moorhead	97-13-14	Horn Park	905	899.5	Based on contours and as-builts that indicate a required closure between 899 and 900 FT.

7. Because of the varying configuration of the levees and locations, eight individual cross sections were analyzed for the "credit to existing levees". These cross sections represent the highest portion along the existing levee reaches. Six of the eight levee reaches are located in Fargo and the other two in Moorhead. Detailed information concerning the levee reaches is summarized below. The locations are summarized in Table 2 and the characteristics are summarized in Table 3. In addition, for each levee reach there is an associated attachment that indicates the specific location of the reach, contours, and drawing/plan sets for the reach.

Table 2: Summary of Location of Existing Levee Reaches

City	Project ID	Location	River Mile	HEC-RAS Section
Fargo	5078-2	10th St. N	440 to 440.5	320 to 321
Fargo	4903	Cardinal Muench Seminary	443.3 to 444.6	330 to 333
Fargo	4579-3	10th Ave S (Dike West)	452.8 to 453.1	388 to 390
Fargo	4908	Lindenwood Drive	453.4 to 453.8	391 to 392
Fargo	4980	Southwood Drive	457.3 to 457.6	407
Fargo	5093	Drain 27 / Rose Coulee	460.4	415 / 7994
Moorhead	97-13-15	Woodlawn Park	452.2	386 to 387
Moorhead	97-13-14	Horn Park	455.5 to 455.7	402 to 403

Table 3: Summary of the Characteristics of the Existing Levee Reaches

City	Project ID	Location	Top of Levee	Elevation at Dryside Toe	Max Height (feet)	Top Width (feet)	Riverside Slope (1V: XH)	Landside Slope (1V: XH)
Fargo	5078-2	10th St. N	898.5	893.5	5	10	3	3
Fargo	4903	Cardinal Muench Seminary	898	893	5	5	6	6
Fargo	4579-3	10th Ave S (Dike West)	905	890	15	10	3	3
Fargo	4908	Lindenwood Drive	905	897	8	8	4	4
Fargo	4980	Southwood Drive	905	901	4	4	3	3
Fargo	5093	Drain 27 / Rose Coulee	909	904	5	10	5	5
Moorhead	97-13-15	Woodlawn Park	904	894	10	10	3	3
Moorhead	97-13-14	Horn Park	905	888	17	10	3	3

8. The Fargo-Moorhead has experienced ten flood events that have crested over 34.5 feet since 1882, with four events occurring since 1997. The largest flood of record occurred on March 28, 2009. A summary of the historical flood crests is provided in Table 4.

Table 4: Historical Flood Crest Elevations for the Fargo-Moorhead Metro Area

Flood Crest (ft)	Year
(1) 40.82	March 28, 2009
(2) 40.1	April 7, 1897
(3) 39.57	April 17, 1997
(4) 37.8	April 11, 1882
(5) 37.34	April 15, 1969
(6) 37.13	April 5, 2006
(7) 36.69	April 14, 2001
(8) 35.39	April 9, 1989
(9) 34.93	April 19, 1979
(10) 34.65	April 16, 1952
* Gage located at approximately River Mile 453. ** Flood Elevation: Crest + 861.4 + 0.94 ft	

9. The river stage is measured at the United States Geological Survey (USGS) gage station 05054000, which is located at approximate river mile 453, near the Fargo water treatment plant and adjacent to the 10th Ave S levee reach. The flood crest or gage reading can be converted to a flood elevation by adding 866.32 feet to the gage reading. This provides a flood elevation at the gage station in NAVD 1988 vertical datum.

10. Because of the natural slope of the river and the various low head dams and bridges on the river, the river / flood elevation is not constant throughout the project area. Therefore a flood profile is required to determine the flood elevation at the existing levee locations for the various flood events. The flood elevation of the last four flood events was estimated using the flood profile that was developed during “Phase 2” of the Feasibility Study. The last four flood events were selected as these events coincide with the time that these levees would have been in place. From the estimated flood profile, the amount of flood water above the toe of the existing levees was determined and is summarized in Table 5. In addition, the table presents the percentage the flood water above the toe compared to the height of the levees.

Table 5: Summary of the Historical Crests

City	Project ID	Location	Amount Historical Crest is Above Levee Toe: April 17, 1997 39.57		April 14, 2001 36.69		April 5, 2006 37.13		March 28, 2009 40.82	
Fargo	5078-2	10th St. N	-0.1	-2%	-3.0	-59%	-2.5	-51%	1.2	23%
Fargo	4903	Cardinal Muench Seminary	1.9	37%	-1.0	-20%	-0.6	-12%	3.1	62%
Fargo	4579-3	10th Ave S (Dike West)	12.3	82%	9.4	63%	9.9	66%	13.6	90%
Fargo	4908	Lindenwood Drive	5.9	73%	3.0	38%	3.4	43%	7.1	89%
Fargo	4980	Southwood Drive	3.5	89%	0.7	17%	1.1	28%	4.8	120%
Fargo	5093	Drain 27 / Rose Coulee	2.7	54%	-0.2	-4%	0.3	5%	4.0	79%
Moorhead	97-13-15	Woodlawn Park	7.8	78%	4.9	49%	5.3	53%	9.0	90%
Moorhead	97-13-14	Horn Park	15.8	93%	12.9	76%	13.4	79%	17.1	100%

11. The existing levees have performed satisfactorily during the four major recent flood events as there have not been any reported signs of distress observed during high water. This is generally consistent with the District's experience in the Red River Valley. The District typically constructs permanent projects using homogeneous clay levees, with a 10-foot top width and 1V on 3H side slopes. These projects typically perform well during floods. During flood fights, emergency clay levees are constructed with side slopes varying between 1V on 2H to 3H side slopes and 8 to 12 foot top widths. These emergency levees generally holdup during floods, but there are instances in which additional work is required to reinforce them. By comparison, the existing levees of Fargo and Moorhead, which are similar in nature to emergency levees and permanent projects, should have similar performance and provide some reliability during similar flood events.

10th Street North

12. The 10th Street North Levee reach refers to the three levee reaches located at the north end of Fargo, near the Fargo Waste Water Treatment Plant. More specifically, these levees protect the following subdivision: Broadway 1st Addition, River Addition, Royal Oaks 2nd Addition, and the Red River Addition. The levees within this reach were constructed and/or modified 3 different times, the first in 1990, again in 1998, and finally in 2000.

13. These levees are located directly adjacent to homes. The levee cross section matches the Corps' typical section, with a 10-foot top width and 1V:3H side slopes. The levees are at most 5 feet high, or top elevation approximately 898.5 FT. The highest level of protection afforded by these levees is 896 FT, which is the top elevation of the Broadway 1st Addition, as indicated in the as-built drawings. Trees and shrubs were observed to be growing on the levee and also beyond the toe. A portion of the levee has a fence constructed on the top. These encroachments would not meet current Corps requirements.

14. Of the four recent flood events, only the 2009 flood event was high enough that water reached the levee slope. The amount of water was a little more than 1 foot, or slightly more than 20% of the levee height.

Cardinal Muench Seminary

15. The Cardinal Muench Seminary levee reach protects the area of the Cardinal Muench Seminary in the city of Fargo. The majority of this levee is less than two feet in height with the exception of the north end, which approaches 5 feet in height. The highest level of protection afforded by these levees is 897.3 FT, which is the top elevation of the levee as indicated in the

as-built drawings. The roadway that circles the seminary is the closest feature to the levee and is located beyond the dryside toe of the levee. This levee reach existed prior to 1999. In 1999, a portion of the levee was relocated closer to the river to its current location.

16. The typical section of this levee is a 5-foot top width, which is less than the Corps' standard top width of 10 feet. The levee side slopes are 1V:6H, which is flatter than the Corps' standard slope of 1V:3H.

17. Out of the four recent events, water reached this levee only twice (1997 and 2009). The height of water on the levee was approximately 2 and 3 feet for the 1997 and 2009 floods, respectively. This equates to approximate 40% and 60% of the levee height.

10th Avenue South (Dike West)

18. The 10th Avenue South levee reach refers to the levee that was constructed by the City of Fargo to extend the Corps' project "Dike West" southward to provide additional protection to the area near the Fargo water treatment plant. Houses and residential apartment/condo buildings are located adjacent to the levee.

19. The typical section of this levee features a 10-foot top width with 1V:3H side slopes, which matches the Corps' typical section. The highest level of protection afforded by these levees is 904 FT, which is the top elevation of the levee as indicated in the as-built drawings. In two instances, retaining walls were constructed into the dryside slope of the levee to accommodate the residential buildings. The highest retaining wall is approximately five feet in height and the topography slopes down and away from the walls. The lowest elevation behind the levee/wall is approximately 890, which is 15 feet below top of levee. Retaining walls are not typically incorporated into a levee cross section unless properly designed. It is unknown as to how the retaining walls and levee were designed.

20. Flood waters were high enough during the last four flood events to be on the levee. The flood waters ranged from approximately 9.4 feet to 13.6 feet above the toe, or approximately 60% to 90% of the levee height.

Lindenwood Drive

21. The Lindenwood Drive levee reach is located in the area north of the Lindenwood Park, off of Lindenwood Drive South, in Fargo. The levee reach alignment was shifted further away from the river in 1999. The current levee consists of an 8-foot top width and 1V:4H side slopes, which is similar to the Corps' typical section. At most, the levee is eight feet in height, corresponding with a top elevation of approximately 905 FT. The highest level of protection afforded by these levees is 901.5 FT, which is the elevation of the driveway that is cut through the levee. Also, the levee does not tie into high ground, but instead slopes down to an elevation between 901 and 902 FT. At one location, a retaining wall was constructed into the dryside slope of the levee in order to minimize the encroachment of an adjacent house.

22. Since the levee was modified in 1999, the last three flood events were high enough that water was on the levee. The water heights ranged from 3 to 7 feet, or approximately 40% to 90% of the levee height.

Southwood Drive

23. The Southwood Drive levee reach is associated with the area at the east end of Southwood Drive, just south of the Fargo Country Club. This levee was constructed in 2007 to a

top elevation of 905 FT. It is a small levee with a 4-foot top width and is at most approximately 4 feet high. The highest level of protection afforded by these levees is 903.5 FT, which is based on the lowest top elevation of the levee and the ground surface elevation the levee ties into. The side slopes are 1V:3H. There is approximately 130 linear feet of retaining wall constructed within the wetside slope. There are many residential homes adjacent to this levee.

24. During the 2009 flood, this levee reach was raised as the crest was predicted to overtop it. This emergency raise of this levee allowed the flood waters to reach an approximate height of 4.8 above the toe, which is 0.8 feet above the top of the permanent levee.

Drain 27 / Rose Coulee

25. The Drain 27 / Rose Coulee levee reach is associated with the drainage ditch "Drain 27" and the Rose Coulee in Fargo and is a few miles long. A portion of the drain and coulee runs through the Rose Creek Golf Course in Fargo and drains into the Red River. So when the Red River is high water, water can back up into the ditch and coulee. Therefore there are levees constructed along the ditch to confine the backwaters within the ditch, preventing homes from being flooded. In addition to providing protection from the Red River, the drain and coulee direct overland flow from the west to the Red River, prevent flooding. Drain 27 and Rose Coulee were constructed prior to the 1997 flood, and were modified afterwards in 1997, 2002, and 2003.

26. The entire drain and coulee work together as a system, the reliability of which is only as good as the most critical section. Due to the relative similarity of the area, only one section was analyzed for this reach and this was at the highest levee location along the ditch. The levee has a 10-foot top width, 1V:5H side slopes, and on the dryside, is at most 5 feet high. This section is more robust than the Corps' typical section of a 10-foot top width and 1V:3H side slopes. The top elevation of the most of the levees is 909 FT, but in some places, it is only 906.3 FT.

27. Since the modifications to the Drain 27 / Rose Coulee reach in 1997, two of the three most recent flood events have been high enough that water rose above the dryside toe. Flood waters were 0.6 feet above the toe in 2006and 4 feet in 2009. During the 2009 flood, the levees through this reach were raised due to a predicted flood crest greater than the top of levee.

Woodlawn Park

28. The Woodlawn Park levee reach is located on the southeast corner of Woodlawn Park in Moorhead. The levee was in-place during the 1997 flood, but was raised to an elevation of 904 FT and extended after the event. The existing levee is constructed with a 10-foot top width and 1V:3H side slopes, which is the Corps' typical section. At most, the levee is 10 feet tall. This levee is located a considerable distance away from the Red River so the water at this location is relatively slow moving. In addition, a lift station is constructed within the levee prism.

29. Water has been on this levee during all four recent flood events. The height of water on the levee has ranged from approximately 5 to 9 feet above the toe, or 50% to 90% of the levee height, respectfully.

Horn Park

30. The Horn Park levee reach is located in the area of Horn Park in Moorhead, just south of Interstate 94, along River Shore Drive. The levee was in-place during the 1997 flood but was reconstructed with a slight shift riverward following the flood. The top elevation of the levee is

905 FT, but the ground surface on the northeast end of the alignment is only 899.5 FT. The typical section consists of a 10-foot top width and 1V:3H side slopes. The majority of the levee is six feet in height but there is a portion in which the levee approaches a height of 17 feet.

31. Since the modifications in 1997, flood waters have been on the levee during the last three most recent events. The height of water above the toe ranged from approximately 13 to 17 feet, or 75% to 100% of the levee height, respectively.

STRATIGRAPHY

32. For the "Credit to Existing Levees" analysis, soil exploration was not completed. Due to the lake deposition environment of the area, the geology is similar throughout Fargo-Moorhead area and the stratigraphy was generalized. The generalization was based on the findings of the previous Corps Section 205 project, Fargo-Ridgewood. It was acceptable to assume this generalized stratigraphy during the "credit to existing levees" analysis as the main failure mode, slope stability, was dependent on only the configuration of the levee and the material it was immediately founded on. The range of the formations' top elevation and the selected top elevation for the "Credit to Existing Levees" analysis is summarized below in Table 6.

Table 6: Summary of Generalized Stratigraphy for Fargo-Moorhead Area

Soil Formation	Range of Top Elevation	Selected Top Elevation
Plastic Laminated Sherack / Sherack	890-891	Ground Surface
Brenna	856 - 858	857
Argusville	803 - 831	815

GEOLOGY

33. The geology of the Fargo-Moorhead area consists of 5 major formations. A summary of the formations is included below and was taken directly from the USACE "Appendix D, Geotechnical Design and Geology" for the Fargo-Ridgewood project, date 2007 (Reference 3).

34. 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. A brief history of the Pleistocene Epoch and related stratigraphy is presented, therefore, 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).

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

36. Bedrock. Bedrock lies at an estimated depth greater than 300 feet beneath the glacial sediments in the region. The bedrock is likely composed of Pre-Cambrian Era, crystalline, igneous and/or metamorphic rock. The bedrock lies well below the influence of the proposed project.

37. Undifferentiated Glacial Sediment. Up to 200 feet of till overlies 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.

38. Argusville Formation. The lowest foundation unit of interest is the 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 drift, this unit has only scattered sand and gravel. The sand and gravel in the deposit was likely 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, brown to dark gray, glacio-lacustrine clay. A gritty texture is the most distinctive feature of the Argusville Formation. Along the proposed project alignment, the Argusville Formation has an approximate top elevation of 803-831 feet (+/- 5 feet), (NAVD 1988 adj.), and has an average thickness of 20-30 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.

39. 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. 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. The Brenna Formation is notoriously unstable as a foundation material throughout the Red River of the North Valley. At the project location, the top of the Brenna Formation has an approximate top elevation of 856-858 feet (+/- 5 feet), (NAVD 1988 adj.) and exhibits a gently undulating surface. In the project area, the unit has an average thickness of approximately 30-35 feet. The contact with the overlying Sherack or Poplar River Formations is an erosional unconformity. The upper 3 to 5 feet of the Brenna Formation is variably harder and more consolidated, probably due to desiccation during sub-

aerial exposure, than the bulk of the Brenna Formation. It is not thick enough, however, to substantially alter the basic weakness inherent within the formation.

40. <u>Poplar River Formation</u>. 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 characterized as laminated, soft to medium stiff, wet, silty, organic rich clay 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. Though known to be more than 100 feet thick in some areas of the formation, in the proposed project area, it averages 8 feet in thickness. The contact with the overlying Sherack Formation is conformable, usually interbedded, and gradational. Although not encountered in any project related soil borings to date, 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.

41. <u>Sherack Formation</u>. 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.

42. An interbedded zone of highly plastic, very wet, grey to grey-brown, silty clay and clayey silt with lower strength properties than most of the Sherack Formation was encountered in boring 01-5M. This unit was identified as the Lower Sherack Formation in the 1986 Design Memorandum for Flood Control (East Grand Forks) and is currently designated as the "plastic laminated" Sherack Formation by the St. Paul District. Typically this layer has a liquid limit greater than 80, with a plasticity index greater than 50. Most commonly, but not exclusively, this zone is located near the base of the Sherack Formation. It may also be found near the top or sandwiched throughout the unit. Often this weaker zone is not continuous from the upper to lower river banks along a given cross section. Generally, when found, the plastic, laminated zone is about 5 to 10 feet thick.

43. The Sherack Formation has been impacted more than any other unit in the project reach by erosion and flooding of the 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. On top of the secondary (upper) bank and beyond, the approximate average top of formation elevation within the project reach is 890 to 891 feet (+/- 5 feet), (NAVD 1988 adj.) with an average thickness of approximately 20 to 25 feet.

44. *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 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 to 15 feet in thickness along the proposed project alignment.*

TOPOGRAPHY

45. The program ArcMap from ESRI, Inc. was used to display the topography of the Fargo-Moorhead area. Contours were developed from a combination of previous data. Using ArcMap to display the contours, critical cross sections were selected. Profiles of the ground surface and existing levees were developed using a combination of both the displayed contours and as-built drawings obtained from the cities of Fargo and Moorhead.

PERFORMANCE MODES

46. The ETL 1110-2-556 (Reference 2) indicates that there are five major geotechnical performance modes to consider for the “credit to existing levees” analysis. These five performance modes are summarized below with an explanation of how they were evaluated.

Underseepage

47. The performance mode “underseepage” as used in ETL 1110-2-556 (Reference 2), is related to the phenomenon known as piping. Piping is the backward erosion of materials caused by seepage forces that are large enough to move particles. As this phenomenon continues, more and more soil particles are moved out of the soil mass and a “pipe”-like cavity is formed. If not caught in time, piping can lead to catastrophic failure of the structure or levee. It is assessed by comparing the maximum exit gradient to the critical gradient.

48. In the case of the Fargo-Moorhead area, the clay levees are constructed on foundation materials that are composed of clay type materials which are relatively impermeable. The District’s flood fighting experiences in the Red River Valley indicate that underseepage is not a likely concern for clay levees on clay foundations. In addition, judgment dictates that seepage should not be an expected concern for clay levees founded on clay foundations without a pervious subsurface stratum.

49. Based on the aforementioned reasons, judgment was used to estimate the probability of failure of the existing levees. The expected performance level of each levee reach with the flood water elevation at the top of the levee was selected. The selected performance level was based on the following:

- If the levee configuration was similar to the Corps’ typical section (10-foot top width, 1V:3H side slopes), and 10 feet in height or less, then performance was “GOOD”.

Levees greater than 10 feet in height were given An “ABOVE AVERAGE” performance level.

- If the levee configuration was less robust than the Corps’ typical section meaning narrower top width, and less than 10 feet high, the performance was “ABOVE AVERAGE”.
- If a retaining wall was constructed within the dryside slope, the performance level was “BELOW AVERAGE”, due to the reduction of the seepage path length.

50. Using the selected performance level, the probability of failure was obtained using Table A1 of ETL 1110-2-556, which is reproduced in Table 7 (Reference 2). The probability of failure at the elevation associated with the dryside toe was taken to be zero. For elevations in between the toe and top of levee, probability of failure was assumed to be linear. The selected expected performance levee for the underseepage failure mode and associated probability of failure is indicated in Table 8.

Table 7: Target Reliability Indices (reproduction of Table A1 of ETL 1110-2-556)

Expected Performance Level	Beta	Probability of Unsatisfactory Performance	Probability of Failure (1 in X times)
High	5	3.00E-07	3.33E+06
Good	4	3.00E-05	33333
Above Average	3	1.00E-03	1000
Below Average	2.5	6.00E-03	166.7
Poor	2	0.023	43.5
Unsatisfactory	1.5	0.07	14.3
Hazardous	1	0.16	6.3

Table 8: Selected Performance Levels for Underseepage Failure Mode

City	Project ID	Location	Selected Expected Performance Level for Underseepage	Probability of Failure, P(F)
Fargo	5078-2	10th St. N	GOOD	3.00E-05
Fargo	4903	Cardinal Muench Seminary	GOOD	3.00E-05
Fargo	4579-3	10th Ave S (Dike West)	BELOW AVERAGE	6.00E-03
Fargo	4908	Lindenwood Drive	BELOW AVERAGE	6.00E-03
Fargo	4980	Southwood Drive	ABOVE AVERAGE	1.00E-03
Fargo	5093	Drain 27 / Rose Coulee	ABOVE AVERAGE	1.00E-03
Moorhead	97-13-15	Woodlawn Park	GOOD	3.00E-05
Moorhead	97-13-14	Horn Park	ABOVE AVERAGE	1.00E-03

Slope Stability for short-term conditions

51. The performance mode "short-term stability" is to assess the stability of the levee under short-term, undrained conditions which is commonly referred to as end-of-construction situation. Due to the fact that the levees have been in place for a number of years, the end-of-construction condition is no longer applicable. The riverward slope is loaded rapidly during a flood event though. This loaded situation was evaluated at the highest levee section (Horn Park in Moorhead) to check to see if this was a critical condition. The analysis was run using undrained shear strength parameters and floodwater to the top of the levee. It was found that the short-term flood loading condition was less critical than the long-term condition. Therefore, the probability of failure due to slope instability was based on the long-term conditions and not the short-term conditions.

Slope stability for long-term conditions

52. The performance mode "long-term stability" is to assess the stability of the levee under long-term, drained conditions. Drained shear strength parameters are required for this analysis along with pore pressures. A summary of the slope stability procedure used is below.

Model

53. The slope stability models were set up in the program SLIDE 5.0 from Rocscience. The stratigraphy was generalized as previously stated. The ground surface profile and levee configuration was determined using both the contours developed and displayed in ArcMap and as-built drawings.

Soil Design Parameters

54. No soil exploration or testing was completed during the "credit to existing levee" study. Instead, design parameters were selected based on the test data that was obtained for previous projects in the area. These projects were the Horace and West Fargo Diversions and the Fargo-Ridgewood Section 205 project. This test data was used to estimate moist and saturated unit weights along with drained and undrained shear strengths.

55. The failure criteria used in the selection of the effective stress shear strength parameters was ultimate or post-peak failure. This is due to the fact that large strains may occur in the soils over time. In the case of the "levee fill" material, the effective shear strength of the material was estimated based on the Sherack effective friction angle and an effective cohesion intercept from triaxial tests on compacted levee fill material for the Grand Forks and East Grand Forks projects. The total stress shear strength parameters were selected based on peak values. The selected shear strength parameters are summarized in Table 9 and the Mohr-Coulomb shear strength plots can be seen in Attachment H-4. Table 10 summarizes the selected unit weights.

56. In addition to determining the mean values for the design parameters, the coefficient of variation or standard deviation is also required when completing a reliability analyses. The coefficients of variation are only required for those parameters that are expected to have a large influence on the slope stability results. For this reliability analyses those parameters are as follows and can be found in Table 9 and Table 10.

- Levee effective friction angle and cohesion intercept,
- Sherack effective friction angle, and
- Plastic Laminated (PL) Sherack effective friction angle.

Table 9: Summary of Shear Strength Parameter

Formation	Most Likely Effective Stress Shear Strength Parameters (1)						Most Likely Total Stress Shear Strength Parameters (2)			Fargo-Ridgewood Parameters (3)	
	ϕ'	σ	Coeff. Of Var.	c'	σ	Coeff. Of Var.	c	σ	Coeff. Of Var.	ϕ'	c
Levee Fill (4)	27	3	10%	250	75	30%	assume values of PL Sherack				
Plastic Laminated Sherack	21	3	14%	0	0		1125	175	16%	19	1150
Sherack	27	3	10%	0	0		assume values of PL Sherack			26	1150
Poplar River	30	2	6%	0	0		1800	350	19%	29	1700
Brenna	15	3	20%	0	0		650	100	15%	13	650
Argusville	17	3	17%	0	0		850	100	12%	16	850

Notes:
(1) The parameters for the formations not including Levee Fill 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. It is assumed that there is no cohesion intercept for the Mohr-Coulomb shear strength envelop except for the levee fill material.
(2) These parameters are based on triaxial shear tests with the failure criterion defined at peak deviator stress.
(3) These are the parameters that were selected and used in the analyses completed for the Section 205 Fargo-Ridgewood project.
(4) The effective friction angle parameter for the Levee Fill material is based on the Sherack formation shear strength parameter and the effective cohesion intercept is based on triaxial test results from the Grand Forks/East Grand Forks projects using compacted clay specimens.

Table 10: Summary of Unit Weights

Formation	γ_m (pcf)	γ_{sat} (pcf)
Levee Fill	assume values of Sherack	
Plastic Laminated Sherack	114	114
Sherack	118	121
Poplar River	121	121
Brenna	103	103
Argusville	105	105

Groundwater

57. The groundwater conditions and the phreatic surface through the embankment were modeled using the piezometric line feature for the slope stability analyses as this is a quick and easy way to approximate the pore pressures in the embankment and foundation materials. The phreatic surface through the levee was approximated as a straight line from the flood water surface elevation and dryside toe of the levee. This assumption is thought to be conservative in the fact that the assumed phreatic surface approximates the steady-state seepage condition if it were to develop. Due to the impervious nature of the materials and the short time duration of the flood events, the seepage conditions would not reach steady-state.

Foundation Conditions

58. Soil exploration was not completed prior to the "credit to existing levees" analysis; therefore generalized stratigraphy of the Fargo-Moorhead region was used. The generalized stratigraphy indicates that the Sherack formation is the top layer and that any levees constructed would be founded on this formation. Experience also indicates that the Sherack formation can be interbedded with a lower shear strength material identified as Plastic-Laminated (PL) Sherack. The PL Sherack occurs at random and could be found throughout the Fargo-Moorhead region. Therefore it was assumed that the levees could be constructed on top of the Sherack formation 50% of the time and the PL Sherack formation 50% of the time.

59. The contact elevation of the Sherack / PL Sherack and Brenna was taken to be 857. The contact elevation of the Brenna and Argusville was taken to be 815.

Procedure

60. The Taylor's Series procedure outlined in the ETL 1110-2-556 (Reference 2) was used to determine the reliability of the existing levees. The parameters that were varied for the analysis were the effective shear strength parameters for the levee, Sherack, and PL Sherack materials. Because there was uncertainty in whether Sherack or PL Sherack would be located beneath the existing levees, the Taylor's Series procedure was conducted for each scenario and the results were combined.

61. To complete the stability analyses the software program SLIDE 5.0 from Rocscience was implemented. The program has multiple features that were set standard for all analyses and are listed below.

- Factor of Safety was calculated using Spencer's Method,
- The failure surface was broken into 50 slices,
- Circular failure searches were used,
- Groundwater was models using a "water table line",
- The grid spacing for the center of the circular failure surface grid was 1 foot by 1 foot,
- Twenty-five radius increments were used,
- Number of trial slip surfaces = 17,576,
- Tension cracks filled with water were used.

Through-seepage

62. The performance mode "through-seepage" is related to the phenomenon known as internal erosion. Internal erosion is similar to piping but is the initiation of the particle movement through/along cracks or defects. Particle movement is initiated when the tractive shear stress exerted on the soil by flow of the water is greater than the resisting tractive shear stress of the soil (ETL 11102-556, Reference 2).

63. Through-seepage can also lead to surface erosion on the landside slope if there is a large enough quantity of flow through the embankment. This often occurs with sand embankments were water can easily flow through the embankment. The levees constructed in the Red River are homogeneous clay levees and water cannot easily flow through them. Therefore through-

seepage is not likely to be an issue as the quantity of flow and velocity through the levees is low and would not cause surface erosion.

64. For clay embankments that are constructed properly, there is a low probability that internal erosion will be a concern as indicated in the ETL. The ETL also goes on to indicate that there are no analytical models to assess internal erosion and judgment or historical data be used to estimate the probability of failure due to through-seepage.

65. Based on the aforementioned reasons, judgment was used to estimate the expected level of performance of the existing levees for the failure mode of through-seepage. The criteria used to select the performance level is as follows:

- If levee configuration was similar to the Corps' typical section, the expected level of performance was taken to be "ABOVE AVERAGE".
- If the levee configuration consisted of 1V:3H side slopes, less than 5 feet in height, but the levee had a narrower top width, performance was taken to be "ABOVE AVERAGE".
- If the levee had flatter side slopes than 1V:3H and was less than 5 feet in height, the level of performance was taken to be "GOOD".
- If a retaining wall constructed within the dryside slope of the levee, the performance level was taken to be "BELOW AVERAGE" due to the reduced seepage length.

66. Using the selected performance level, the probability of failure was obtained using Table A1 of ETL 1110-2-556, which is reproduced in Table 7. The probability of failure at the elevation associated with the dryside toe was taken to be zero. For elevations in between the toe and top of levee, probability of failure was assumed to be linear. The selected expected performance levee for the through-seepage failure mode and associated probability of failure is indicated in Table 11.

Table 11: Selected Performance Levels for Through-seepage Failure Mode

City	Project ID	Location	Selected Expected Performance Level for Through-Seepage	Probability of Failure, P(F)
Fargo	5078-2	10th St. N	ABOVE AVERAGE	1.00E-03
Fargo	4903	Cardinal Muench Seminary	GOOD	3.00E-05
Fargo	4579-3	10th Ave S (Dike West)	BELOW AVERAGE	6.00E-03
Fargo	4908	Lindenwood Drive	BELOW AVERAGE	6.00E-03
Fargo	4980	Southwood Drive	ABOVE AVERAGE	1.00E-03
Fargo	5093	Drain 27 / Rose Coulee	ABOVE AVERAGE	1.00E-03
Moorhead	97-13-15	Woodlawn Park	ABOVE AVERAGE	1.00E-03
Moorhead	97-13-14	Horn Park	ABOVE AVERAGE	1.00E-03

Surface Erosion

67. The purpose of the performance mode "surface erosion" is to assess the potential for the levee to be eroded by the current flowing parallel to the slope or by the wave action against the slope. The ETL 1110-2-556 (Reference 2) outlines a procedure to estimate the probability of

failure due to erosion due to flow parallel to the slope. This procedure was followed for the "credit to existing levee" analysis.

68. Hydraulic modeling of the river indicated that the expected river velocities adjacent to the levees would be between 1 and 2 feet per second for flows associated with flood events between the 100-year and 500-year. In general, the top of the levees are around the 100-year flood elevation, or slightly higher. Therefore a conservative river velocity of 2 feet per second was used for all water elevations. In addition, the following parameters were also assumed.

- Coefficient of variation for the velocity: approximately 11%
- Critical velocity where erosion begins: 5 feet per second (this is the maximum velocity allowed by the St. Paul District before riprap is to be used as erosion protection. Table 2-5 of EM 1110-2-1601 (Reference 4) indicates that the maximum permissible mean channel velocities for silt clay and clay earth is 3.5 and 6.0 feet per second, respectfully. The existing levee material is expected to be made up of more clay than silt clay material and the value of 5 feet per second falls in between the suggested range)
- Coefficient of variation of the critical velocity: 20%

69. Based on the assumed values, a probability of failure equal to 3.18×10^{-5} (1 chance in 31,444) was calculated. This value was used for all levees at all flood elevations.

70. With the narrow river basin and "short" fetch distances, any waves that are formed are not likely to erode a grass covered, clay slope. In addition, there currently are no models to predict erosion from wave action. Therefore, probability of failure was not calculated for this situation.

RESULTS

71. The performance of the existing Fargo and Moorhead levees and the lack of reported distress for the past flood events indicates that they have provided some reliable means against flooding. In general, the results of the "credit to existing levees" analysis tend to show this, as expected. However, only four recent flood events have occurred since the existing levees were constructed. Furthermore, the existing levees, for the most part, have not been tested with flood waters to the top of the crest. Therefore, the analyses performed show a wide range of reliabilities from high to low due to uncertainties associated with performance at maximum floodwater and variations in geometry and levee height.

72. The results of the "credit to existing levees" analysis indicate a wide range of probability of failure of the levees. With the flood water at the top of the levee, the probability of failure ranges from 0.001 to 0.38. These values are summarized below in Table 12. This relates to a frequency of failure on the range of 1 chance in 913 to 1 chance in 3.

Table 12: Summary of the Probability of Failure with Flood Water at Top of Levee

City	Project ID	Location	Probability of Failure, P(F)	Frequency of Failure	Reliability, R	Expected Performance Level
Fargo	5078-2	10th St. N	0.0011	913	0.9989	ABOVE AVERAGE
Fargo	4903	Cardinal Muench Seminary	0.0062	162	0.9938	BELOW AVERAGE
Fargo	4579-3	10th Ave S (Dike West)	0.2646	3.8	0.7354	HAZARDOUS
Fargo	4908	Lindenwood Drive	0.3825	2.6	0.6175	HAZARDOUS
Fargo	4980	Southwood Drive	0.0081	123	0.9919	BELOW AVERAGE
Fargo	5093	Drain 27 / Rose Coulee	0.0021	477	0.9979	AVERAGE
Moorhead	97-13-15	Woodlawn Park	0.0120	83	0.9880	POOR
Moorhead	97-13-14	Horn Park	0.0476	21	0.9524	HAZARDOUS

73. In general, the analysis indicates that for levees that have a similar cross section to the Corps' typical section and are 5 feet or less in height, the calculated "expected performance level" is "AVERAGE" to "ABOVE AVERAGE". If the levees slightly less robust than the Corp's typical section, the calculated performance level is "BELOW AVERAGE". As the levee heights increase, the performance level decreases to "POOR" to "HAZARDOUS". If retaining walls are incorporated into the dryside slope, the performance level is "HAZARDOUS".

74. The reliability analysis indicates that the performance of the levees, in most instances, is controlled by the stability of the levee. This is what is generally thought to be the case when clay levees are constructed on clay foundations. The stability of the levee is controlled by the strength of the levee material and also the foundation material. Without soil borings being taken at the location of the levees, it was assumed that there was a 50% chance that the foundation material would be Sherack and the other 50% of the time, the foundation would be PL Sherack. The strength of the PL Sherack is 75% of the Sherack. This has a major effect on the outcome of the stability analysis. This is the main reasoning that the overall probability of failure of the existing levees can be so high.

75. During the "credit to existing levees" study the probability of failure was determined for multiple flood elevations. In Table 12 above, only the calculated probability of failure with flood water at the top of the levees are indicated. Due to the "discontinuities" of many of these levees, flood water could flank these levees through adjacent lower spots. In addition, some portions of the levees themselves are lower. These "discontinuities" were incorporated into the relationship of the probability of failure to flood elevation. If the flood water elevation was higher than the "discontinuity", then the probability of failure of the levee reach was taken to be 1.

76. The relationship of the probability of failure to flood elevation can be seen in Figure 1. Figure 2 is a similar plot to Figure 1 but shows the relationship of the probability of failure to the height of water on the levee. These plots can also be found in Attachment H-4. The solid lines on the figures indicate the probability of failure when "discontinuities" are accounted for. The dashed lines on the figures indicate the probability of failure without the discontinuities. The performance levels are also indicated on the figures.

77. When the discontinuities are accounted for, the expected performance levels for the majority of the levees are between "ABOVE AVERAGE" and "BELOW AVERAGE". For the Horn Park levee in Moorhead, the expected performance level is in the "POOR" range before the flood waters reach the discontinuities. For the Lindenwood Drive and 10th Avenue S levees in

Fargo, which have retaining walls within the dryside slope, the expected performance level prior to the flood waters reaching the discontinuities, is “UNSATISFACTORY” and “HAZARDOUS”, respectively. This relationship is better shown in Figure 3, which only plots the probability of failures to 0.3.

78. Details of the slope stability and reliability calculations for the “credit to existing levees” analyses can be found in Attachment H-5.

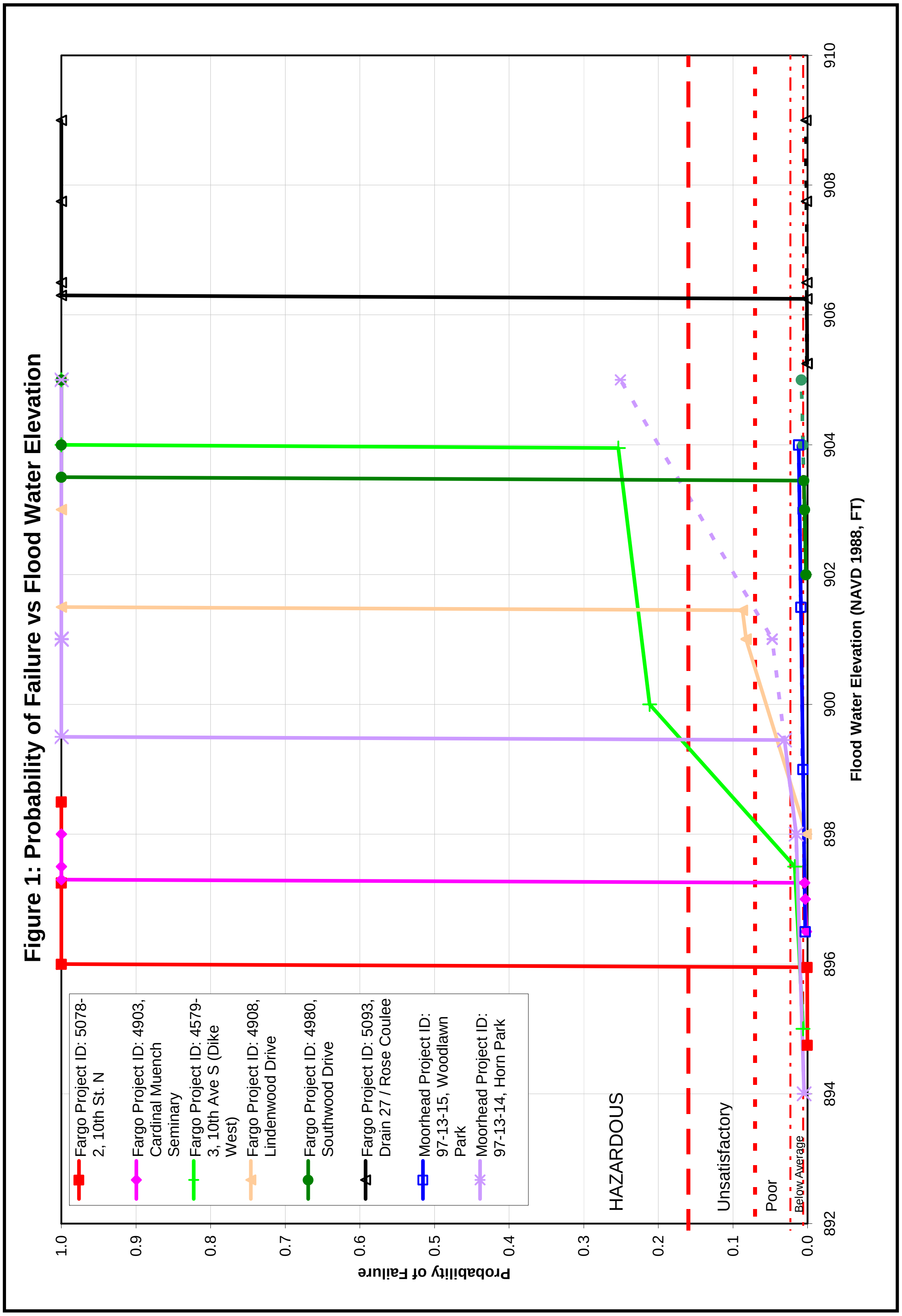


Figure 1: Probability of Failure vs Flood Water Elevation

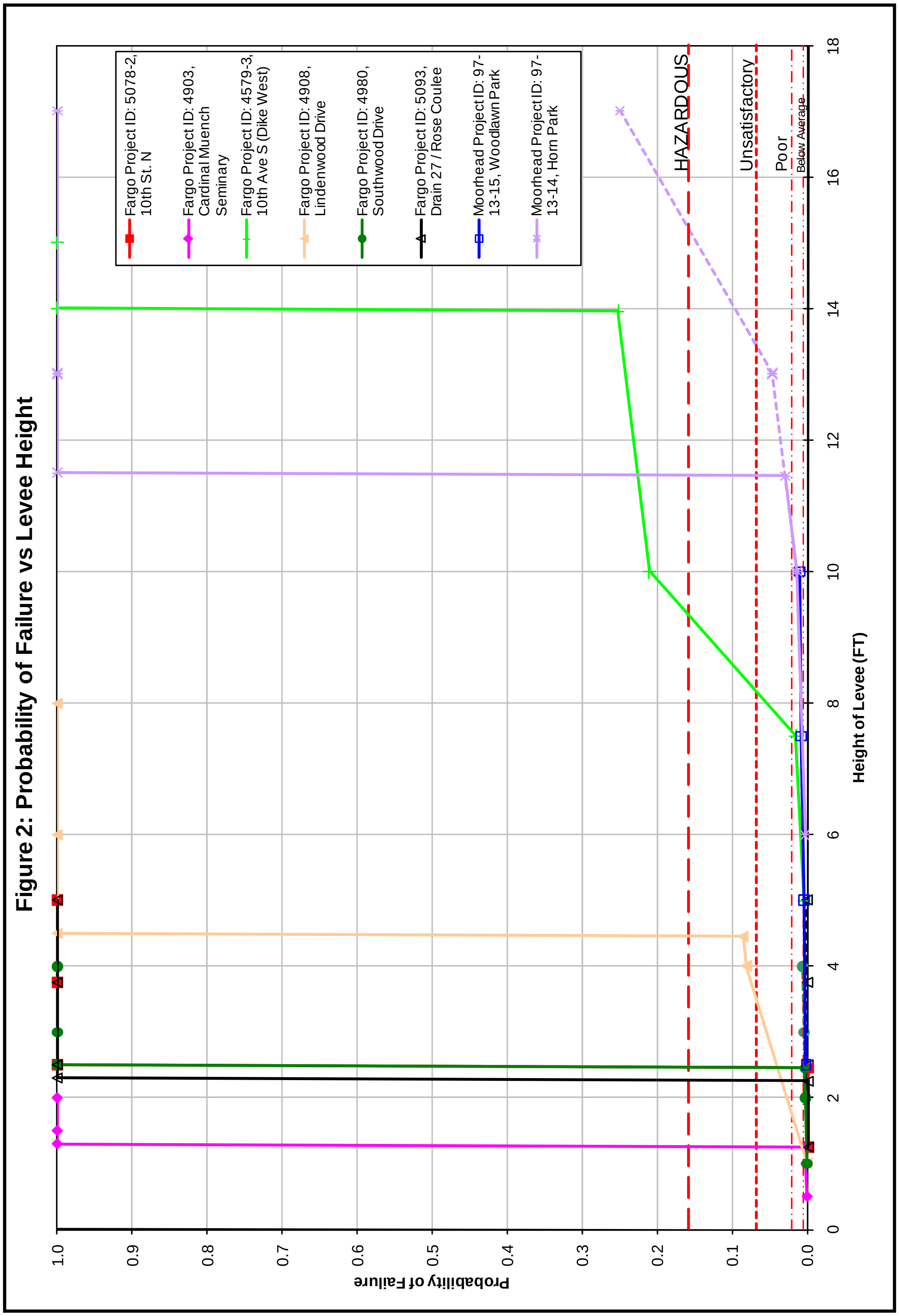


Figure 2: Probability of Failure vs Levee Height

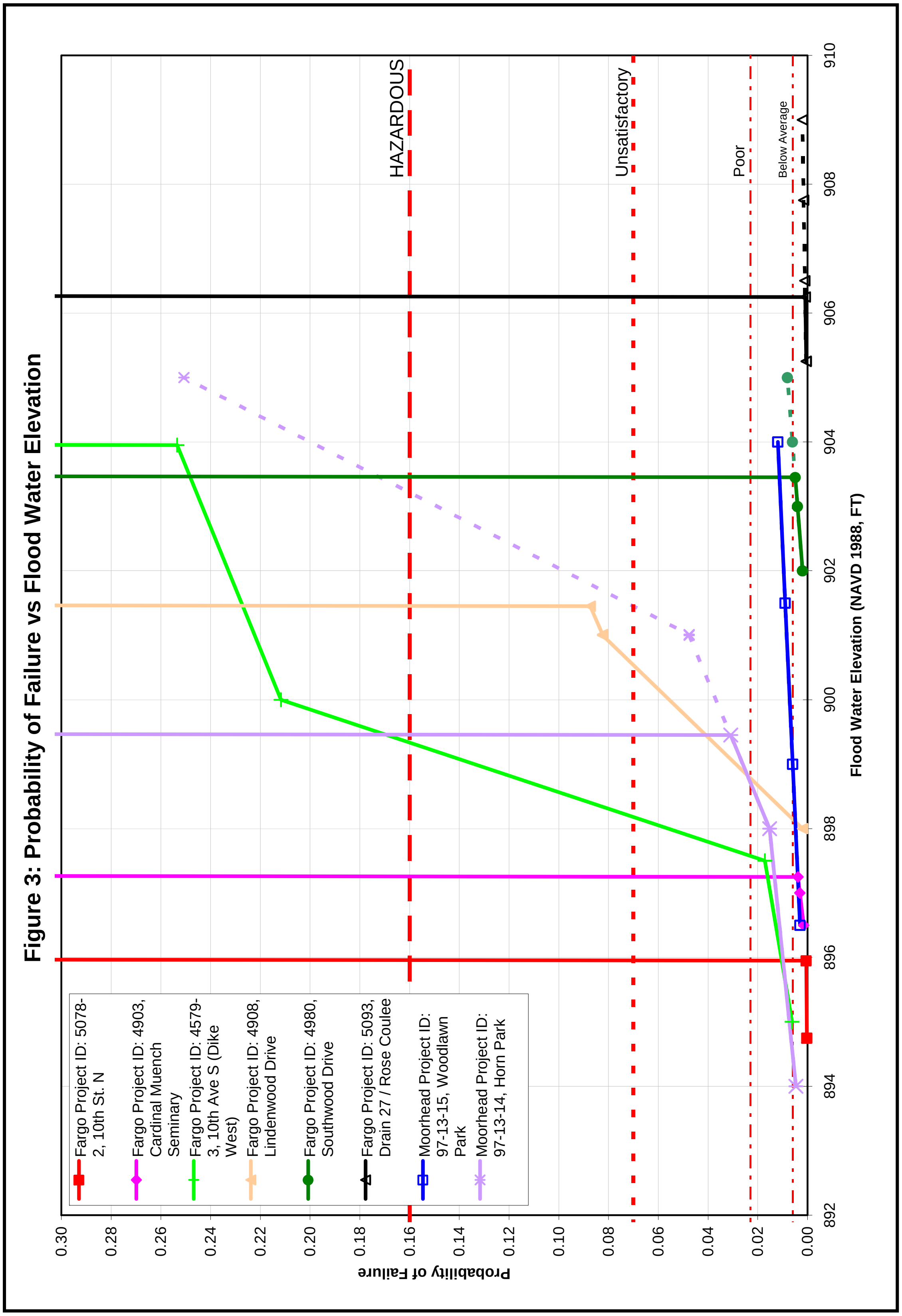


Figure 3: Probability of Failure vs Flood Water Elevation
Fargo Project ID: 5078-2, 10th St. N
Fargo Project ID: 4903, Cardinal Muench Seminary
Fargo Project ID: 4579-3, 10th Ave S (Dike West)
Fargo Project ID: 4908, Lindenwood Drive
Fargo Project ID: 4980, Southwood Drive
Fargo Project ID: 5093, Drain 27 / Rose Coulee
Moorhead Project ID: 97-13-15, Woodlawn Park
Moorhead Project ID: 97-13-14, Horn Park
HAZARDOUS
Unsatisfactory
Poor
Below Average
Probability of Failure
Flood Water Elevation (NAVD 1988, FT)
0.30
0.28
0.26
0.24
0.22
0.20
0.18
0.16
0.14
0.12
0.10
0.08
0.06
0.04
0.02
0.00
892
894
896
898
900
902
904
906
908
910

Short-Term (Undrained Conditions) Slope Stability

79. A check of the undrained (short-term) stability of the levee was completed to make certain that the long-term stability condition was most critical. An undrained stability analysis was completed on the Horn Park Levee, as this is the highest levee and has 1V:3H slopes, making it the most critical levee. Using average undrained shear strength parameters and a flood water elevation at top of the levee, a factor of safety against slide of 2.1 was computed. This is well above the factors of safety for the drained conditions of 1.3 and 1.0, using both Sherack and PL Sherack foundation materials, respectively. When a Taylor series slope stability analysis was completed using undrained shear strength parameters, the probability of failure was found to be negligible. Therefore this verifies that the assumption that the long-term (drain conditions) slope stability condition was most critical was correct.

Long-Term Slope Stability Check

80. A check of the long-term (drained) slope stability calculations was carried out to verify the results of SLIDE 5.0. To complete this check, the program Slope/W 2007 from GeoSlope, LTD was used. Two levee sections, Woodlawn Park and Horn Park, were modeled in Slope/W such that they were the same as the SLIDE 5.0 models. Taylor series slope stability runs were completed using Slope/W. For Woodlawn Park levee, only the Sherack foundation condition was modeled. The search mechanism used was a grid. The factors of safety computed by Slope/W were the same as SLIDE 5.0. For the Horn Park levee, both the Sherack and PL Sherack foundation were modeled. For both foundation conditions, the factors of safety computed by Slope/W were the same as those computed for SLIDE 5.0.

OTHER STRUCTURES

81. There are two locations in the Fargo-Moorhead area that should be noted because of their unique circumstances. These areas are summarized below.

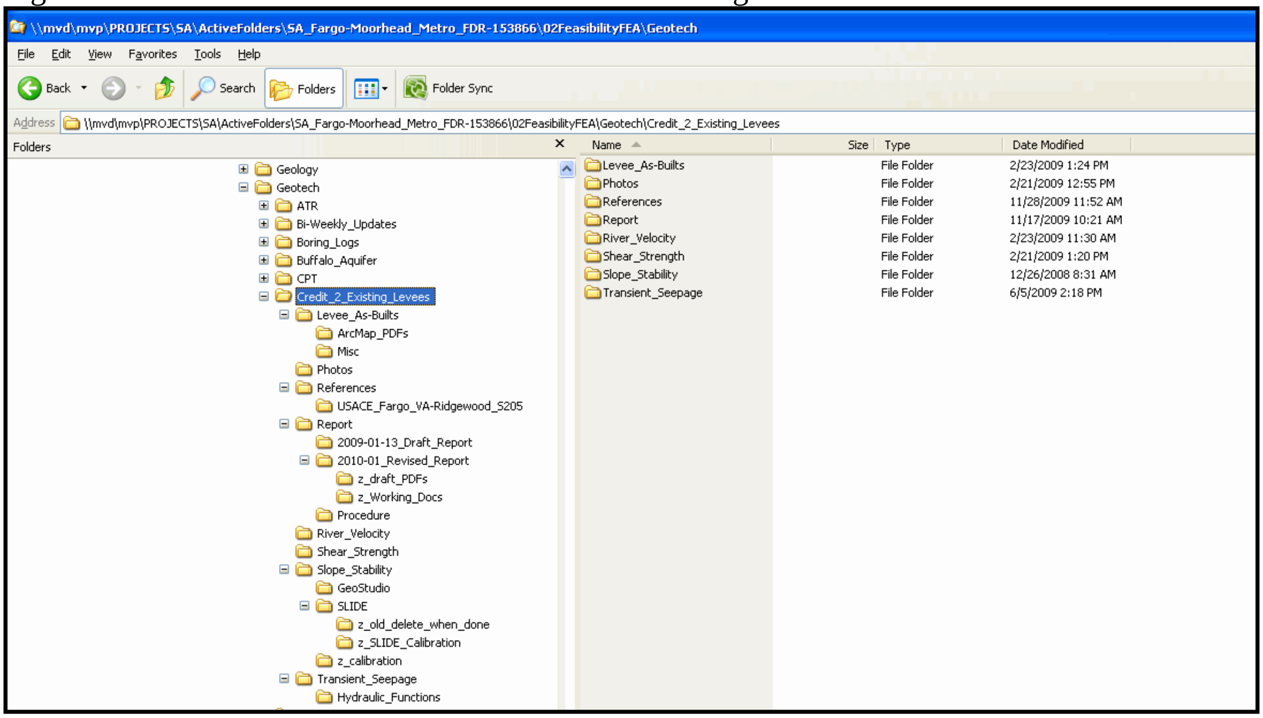
82. The first area that is unique is located at the Oak Grove Lutheran High School in Fargo. A floodwall to top elevation of 902.5 feet was constructed around the eastern portion of the high school to provide protection. This floodwall system ties into an existing levee on the north side that has an approximately elevation of 902 feet, or approximately 4 to 5 feet high. An emergency closure is required to be constructed on the south side when the flood water is expected to get above approximate elevation 897 feet. The Oak Grove levee is similar to the Fargo Southwood Drive levee. Therefore the reliability of the Oak Grove levee can be estimated based on the reliability of the Southwood Drive levee but the discontinuity if the Oak Grove levee must be accounted for.

83. The second levee is located in Moorhead along 1st Avenue, just north of the Moorhead Center Mall. The purpose of this levee is to prevent 1st Avenue from flooding during more frequent flood events. It does not protect any structures. At a flood stage of 31.0 feet, the City of Moorhead floods the underpass and traffic is rerouted. Construction of an emergency levee is also required on the west side of the underpass to protect and provide access to the apartment complex and Heritage Hjemkomst Center.

LOCATION OF FILES

84. All files associated with the “credit to existing levees” are within the project directory located on the server. The folder structure is such that each type of information is located in its own folder, which is appropriately named. The folders which can be seen in Figure 3 can be found using the link below.

Figure 4: Location and Folder Structure Credit to Existing Levees Files



Link:\\mvd\mvp\PROJECTS\SA\SA_Fargo-Moorhead_Metro_FDR-153866\02FeasibilityFEA\Geotech\Credit_2_Existing_Levees\Report\2010-03_Revision_2\z_working_docs\ \mvd\mvp\PROJECTS\SA\SA_Fargo-Moorhead_Metro_FDR-153866\02FeasibilityFEA\Geotech\Credit_2_Existing_Levees\ \\mvd\mvp\projects\sa\SA_Fargo-Moorhead_Metro_FDR-153866\02FeasibilityFEA\Geotech\Credit_2_Existing_Levees

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Reference 1: U.S. Army Corps of Engineers 1991 (December 23). Policy Guidance Letter No. 26, *Benefit Determination Involving Existing Levees*.

Reference 2: U.S. Army Corps of Engineers. 1999 (May 28). “Risked-Based Analysis on Geotechnical Engineering for Support of Planning Studies,” *ETL 1110-2-556*.

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Reference 4: U.S. Army Corps of Engineers. 1994 (June 30). “Hydraulic Design of Flood Control Channels.” *EM 1110-2-1601*