SECTION 205 FEASIBILITY REPORT

ADA, MINNESOTA

WILD RICE AND MARSH RIVERS, MINNESOTA

APPENDIX C

GEOTECHNICAL ANALYSIS

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GEOLOGY AND GEOTECHNICAL DESIGN

1. PURPOSE:

This appendix presents the general geology and specific geotechnical analysis for the Ada, MN Flood Risk Management project.

2. TOPOGRAPHY and PHYSIOGRAPHY

The Red River of the North drainage basin is located within the Red River Valley Section of the Central Lowlands Physiographic Province of North America. Ada, Minnesota, the proposed project site, is centrally located between the Red River of the North and the eastern edge of the Red River Valley in central Norman County. Ada is located on the north bank of the Marsh River. Approximately 15 miles west of Ada lies the Red River of the North which marks the center of Lake Agassiz basin. The Pembina Escarpment marks the boundary between the Red River Valley and the Glaciated Plains sub-sections of the Central Lowlands Physiographic Province to the west in North Dakota.

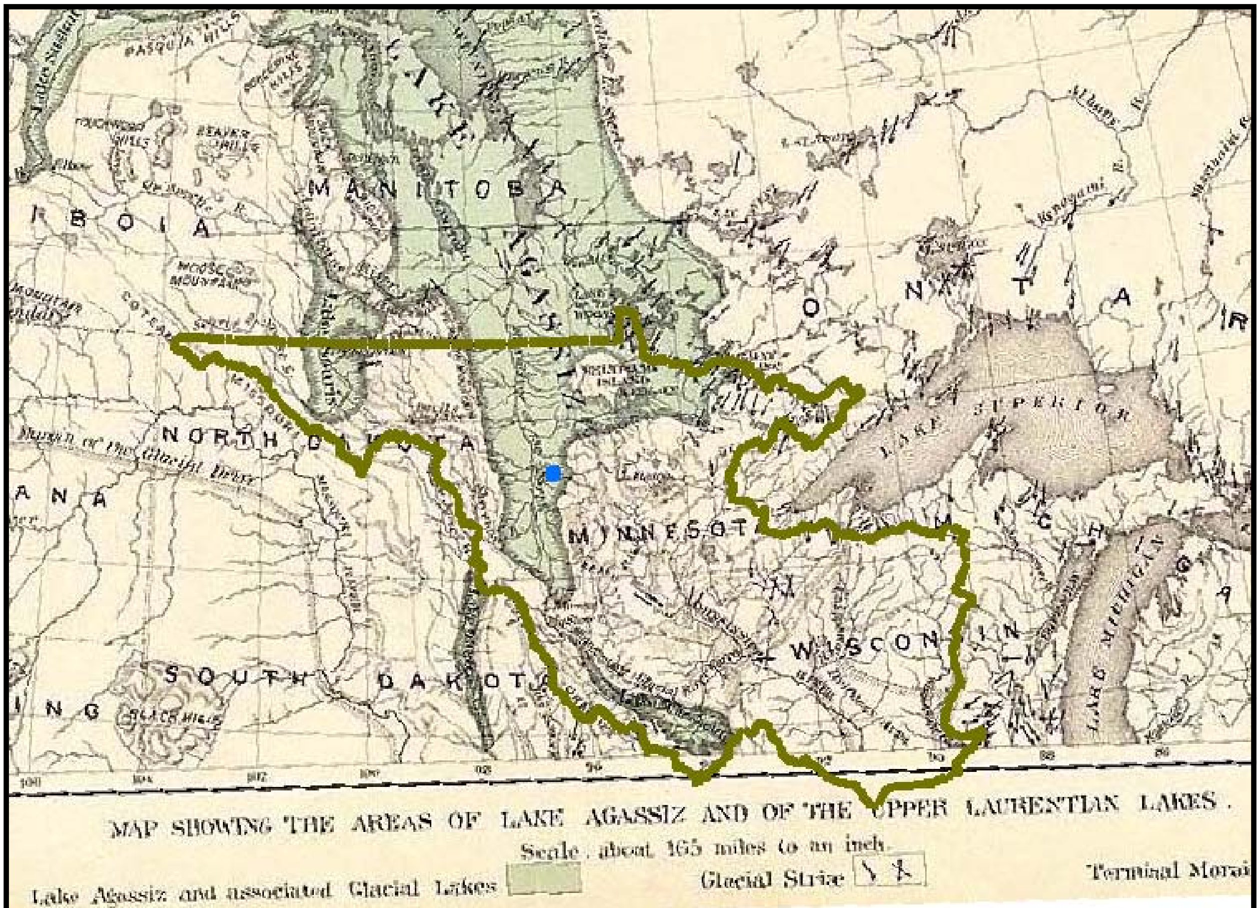

Figure C-1: Ada is shown by the blue dot.

The Red River valley is not a traditional “river valley” of erosional origin, but a nearly level featureless lake plain that was once the bottom of glacial Lake Agassiz. 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 Marsh River flows northwest where it joins the Red River of the North. The Red River of the North flows 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. Drainage of Norman County via the Wild Rice and Marsh Rivers is mainly Westward, or perpendicular, to the trend of The Red River of the North.

3. REGIONAL GEOLOGY and STRATIGRAPHY

The geology influencing the Red River Valley Section 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 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. 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 Valley of the North established itself on top of the lake sediments. The basis for most of the stability analysis prepared for this report is a direct result of the geologic setting. 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 Miscellaneous Series No. 44 (Moran, 1972),
North Dakota Geological Survey Bulletin 57 (Bluemle et al, 1973),
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.

Bedrock. Bedrock lies at an estimated depth greater than 200 feet beneath the glacial sediments in the region. The bedrock is likely composed of Paleozoic Era, Cretaceous Period sedimentary rock or granitic intrusive rocks. The bedrock lies well below the influence of the proposed project.

Red Lake Falls Formation. The lowest foundation unit of interest is the Red Lake Falls Formation. Typically, this is a very stiff to hard, sandy clay till. The formation was likely deposited by the Pre-Caledonian Advance of the Lostwood Glaciation (Wisconsin Episode) approximately 14,000 years BP. Locally the unit may be composed entirely of sand and gravel.

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, wet, soft to very soft, 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. The contact with the overlying Sherack is an erosional unconformity. The upper 5 to 10 feet of the Brenna Formation may be variably harder and more consolidated, probably due to desiccation during sub-aerial exposure.

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. 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.

Present period sediments. As the northeastern outlets for the lake opened for the final time, it is estimated that Glacial Lake Agassiz retreated from Minnesota 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 such as the Wild Rice and Marsh Rivers flowing from the high ground to the east. The present day Wild Rice and Marsh Rivers watershed is the result of this post-glacial erosional activity. Overland flood sediments from the Wild Rice and Marsh Rivers blanket the area surrounding the project. These surface sediments may be characterized generally as soft to medium stiff, fluvial or alluvial, silty clay or clayey silt. Near the Marsh River, the sediment contains some thick deposits of sand or organic matter. Adjacent to urban development, fill and rubble may be present in the upper sediments. The river exhibits no well defined flood plain. The depths of these surface sediments are highly variable and may range widely in thickness.

The Red Lake Falls, Brenna, and Sherack Formations may be combined geologically as part of an assemblage known the Coleharbor Group. All of these deposits are the result of processes directly related to Glacial Lake Agassiz and associated Wisconsin Age glacial deposition. The present period sediments may be classified geologically as the Walsh Formation. These soils are the result of post-glacial river, wind, or other erosional process.

4. SEISMIC RISK and EARTHQUAKE HISTORY

According to Corps of Engineers Regulation ER 1110-2-1806, Earthquake Design Analysis for Corps of Engineers Projects, Ada, Minnesota is located within earthquake Seismic Risk Zone 0. The Uniform Building Code of the International Conference of Building Officials assigns every location in the United States to a four-grade Seismic Risk Zone (0 = least risk, 3 = greatest risk).

The Ada area in the Red River Valley Section of the Central Lowlands Physiographic Province is one of the least seismically active places in the United States. 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 220 miles west of Ada The fault separates the stable Wyoming and Superior Cratons of the tectonically-inactive Canadian Shield. An earthquake occurred along this fault south of Bismarck, North Dakota, in 1968. It had a magnitude of 4.4 on the Richter Scale (IV-V Mercalli Intensity). Northwest of the project, 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.

In Minnesota there are few faults that could possibly affect the project. The Morris fault extends diagonally from the town of Morris, Minnesota to the Brainerd area in west-central Minnesota, roughly 110 miles southeast of Ada. The Morris 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. However, it was not felt north of Grand Forks, North Dakota. 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 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 Ada, Minnesota.

5. SUBSURFACE INVESTIGATIONS

A total of thirteen machine and one hand auger soil borings were advanced by the St. Paul District in the project area in the year 2000 and 2006. The boring logs for the 14 COE borings are presented on Plates C-2 through C-5 of this appendix. The soil borings ranged in depth from 30 to 80 feet below ground surface. The boring locations are presented on Plate C-1.

Limited index testing was completed to delineate the contact between the different geologic units. Tests taken from samples consist of atterberg limits and natural moisture content. Results of the all the laboratory tests taken in the Ada area are shown on the boring log plates. Table C-1 below summarizes the soil testing results. Results confirm the borings, showing consistently higher LL, PL, PI, Liquidity, water content, C_c, and e_0 for the Brenna Formation (generally, existing at depths greater than 13 feet). The testing results on the samples taken from the subsurface investigation were as follows:

Table C-1: Testing Summary

Formation	LL	PL	PI	Liquidity	ω_0	C_c	e_o	γ_{sat}	γ_{moist}	γ_{sub}
Sherack	71.2%	25.4%	45.8%	0.42	37.1%	0.63	1.78	102	101	40
Brenna	108.4%	32.8%	75.6%	0.52	72.6%	1.16	2.06	98	98	36

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6. SITE STRATIGRAPHY

Most of the observed conditions that are the basis of this report are closely related to the geologic setting within the proposed project site. Although the general stratigraphic sequence in the Red River Valley Section is more or less understood, this sequence can be altered within the meander belt of a given tributary or main stem river. Material found in the project area is similar in characteristics and engineering properties with other regions of the Red River of the North basin. The borings show that the soils are comprised mostly of silts and clays. The proposed project would be founded on weak glacio-lacustrine sediments throughout its length. These glacio-lacustrine clays are referred to as the Brenna Formation and the Sherack Formation in the General Reevaluation Report for East Grand Forks Minnesota and Grand Forks North Dakota, 1986. This designation will be used for this report also. Some of the surface excavation will be in fluvial/alluvial deposits (present period sediments), which are the youngest in the region. These soils blanket the project area and are thickest in the meander belt of the Marsh River. The stratigraphic units will be discussed from bottom-most to ground surface.

<u>Red Lake Falls Formation.</u> The lowest unit of interest for any foundation work proposed thus far is the Red Lake Falls Formation. On the basis of soil borings this unit is characterized as a stiff to hard, variably pebbly or sandy, low plasticity, moist, silty clay till. No site specific testing is available, but testing elsewhere in the Red River Valley indicates that the plastic limits vary from 17 to 26, liquid limits from 29 to 35 and moisture contents between 20 and 32 percent. In the Red River Valley area caissons or piles are typically set into the Red Lake Falls Formation. The top of the Formation exhibits a gently undulating surface with an approximate elevation of 635 feet (+/ - 10 feet), (NGVD 1929 adj.). No soil boring taken for this project penetrated the entire unit so the total thickness of the unit at each specific site is unknown. All available literature indicates that the unit averages approximately 45 feet in thickness.

<u>Brenna Formation</u>. The next upper-most unit is the Brenna Formation. This glacio-lacustrine

clay is notoriously unstable and is the acknowledged cause of most of the soil stability problems encountered in the Red River Valley Section. The sediment source for this formation was eroded Pierre Shale bedrock. On the basis of soil borings this unit has been classified as soft, mostly massive, highly plastic, wet, dark grey clay that often shears with a distinct slickensided appearance. Usually, the upper portion of the Brenna Formation has been exposed to sub-aerial weathering which has altered its physical characteristics. This upper desiccation zone does not exist everywhere, but is quite common and has an average thickness of about 3-5 feet throughout the project reach. The desiccation zone is variably harder and more consolidated than the bulk of the Brenna Formation, but is not thick enough to substantially alter the basic weakness inherent within the formation. The contact with the overlying Sherack Formation is a sharp, erosional unconformity. Laboratory testing in other areas of the valley indicate that plastic limits vary from 21 to 42 percent, liquid limits vary from 54 to 134 percent, and moisture content varies from 26 to 87 percent. Thickness of this unit is in the range of 60 to 70 feet. In the project area, the top of the Brenna Formation ranges in elevation from 896 feet at boring 06-13M to a low of 869.7 at boring 06-7M which is located near the Marsh River, (NGVD 1929 adj.) and exhibits a gently undulating surface dipping to the northwest. Nearer to the river course, the top elevation may be more variable due to the existence of erosional scars.

<u>Sherack Formation</u>. Typically, the stratigraphic unit encountered above the Brenna Formation is the Sherack Formation. Like the Brenna Formation, the Sherack Formation is a glacio-lacustrine deposit. The source material for the Sherack sediments was the glacial uplands, instead of shale bedrock. It is fairly stable when excavated, but is easily eroded where nonplastic silts are exposed. Almost all borings indicated a saturated nonplastic silt seam in the lower portion of this formation near the contact with the Brenna. This silt seam averages 1 to 2 feet in thickness. Based on the soil borings this unit can be classified as medium stiff, medium to high plasticity, laminated to medium bedded, wet to saturated, silty clay and clayey silt with minor amounts of fine sand, gypsum and calcite crystals, and organics. The unit is usually brown to yellow-brown with frequent iron oxide or calcareous concretions. Tests taken samples throughout the valley indicate that the plastic limits vary from 18 to 42, liquid limits from 24 to 84 and moisture contents between 14 and 53 percent. In the project area the unit thickness is approximately 10 feet. Nearer to the river course, the top elevation may be more variable or it may have been totally removed by scour.

<u>Present period sediments</u>. The ground surface sediments blanketing the area today are derived primarily from alluvial and/or fluvial sedimentary processes. Also found in the uppermost deposits within the proposed project area are weathered Sherack Formation with little to no cover, and fill or topsoil. These surface sediments may be characterized generally as soft to medium stiff, silty clay or clayey silt. Variably, the unit may contain sand, gravel, or organic matter and range from massive to weakly laminated. Moisture content ranges from dry to saturated. In the project area, present period sediment thickness is variable and can range from about 1 to 30 feet, with an average thickness of between 1-3 feet. The only practical method for evaluation is to reference a boring location.

7. STRUCTURE

Evidence of sliding along Judicial Ditch 51 is prevalent. Obvious surficial evidence of slide activity noted includes braking of drainage utilities, scarps and hummocky topography within the ditch, and leaning trees. The above evidence was used to determine which criteria were appropriate for any slope stability analysis in the project reach.

8. SITE HYDROGEOLOGY

The generally low permeability of the soils within the proposed project boundaries makes determination and prediction of groundwater levels challenging. Occasionally some fluvial seams near the river are sufficiently pervious to allow a confident measurement, however this does not yield much useful information about the interaction between the river water surface and the overbank groundwater conditions.

Currently insufficient data exists for a detailed site specific groundwater characterization at the Ada project site. Commonly, groundwater levels in the project area are high. Groundwater will be located within ten feet below the ground surface. Water levels fluctuate seasonally, with fall /winter conditions exhibiting the lowest measured water levels as might be expected. Water levels were most frequently, but not exclusively, measured in the silt portion of the Sherack Formation.

9. CONSTRUCTION MATERIALS

<u>Concrete Aggregate, Riprap, and Bedding</u>. Sources for fine and coarse concrete aggregate, bedding, and riprap should be available locally. Most commercial aggregates in the Ada vicinity are obtained from the beach ridges of Glacial Lake Agassiz east of the Red River. Additional material may be available from field stone piles in farm fields. The material consists primarily of rounded, wave-washed boulders, cobbles, and sand. If large quantities of riprap size material are required, producers will need adequate lead time in order to stockpile material. Outside sources of quarried, angular, stone should also be available approximately 200 miles east of the proposed project in western and central Minnesota. It is an established fact with local construction contractors that concrete aggregate may be obtained from beach ridges on eastern edge of Glacial Lake Agassiz. Additional investigations will be necessary prior to plans and specifications in order to accurately quantify the amount as well as the quality of stone product available within a reasonable radius of the area.

<u>Levee Borrow</u>. The levee borrow will be obtained from the excavation of the new ditch and should not be a problem. Archeological investigations must be completed before any borrow sites may be used for the project. Geotechnical parameters to be defined prior to approval

include the thickness of topsoil, presence or absence of saline soils, thickness and suitability of alluvial/fluvial soils and their susceptibility to cracking, water bearing seams and water table conditions, natural moisture content, and Procter density. Much of the excavated soil to be used as borrow for the proposed levee will be from the Sherack Formation which is known to be susceptible to cracking. This cracking will have to be dealt with because adequate quantities of borrow from other non-cracking formations are not available within a reasonable haul distance.

10. GENERAL GEOTECHNICAL DESIGN:

The Geotechnical Design philosophy used for section 205 projects is no different than that used for other flood risk management projects.

11. SELECTED PLAN SUMMARY:

The selected plan is shown in the main report. Table C-2 below lists the quantities of the various features of the selected plan with its geotechnical aspect(s).

Table C-2

Feature	Quantity	Geotechnical Aspect
Topsoil	40,000 yd^2	-Locate borrow area
Stripping	31,000 yd^2	-Locate disposal area if excess
Reroute of Judicial Ditch No 51 Channel	360,000 yd^3	-Locate disposal area -Compute stable side slopes
Levee Fill	220,000 yd^3	-Compute stable side slopes -Compute possible settlement -Find and control areas of high gradients -Locate borrow or disposal area
Rock for Riprap/Rock Structures	1,000 yd^3	-Rock gradation -Rock source -Type and design of filter

12. SLOPE STABILITY:

A slope stability analysis was completed using criteria in EM 1110-2-1913 which describes the following Cases that could be analyzed:

a. Case I - End of construction. This case represents undrained conditions for impervious embankment and foundation soils; i.e., excess pore water pressure is present because the soil has not had time to drain since being loaded. Results from laboratory Q (unconsolidated-undrained) tests are applicable to fine-grained soils loaded under this condition while results of S (consolidated-drained) tests can be used for pervious soils that drain fast enough during loading so that no excess pore water pressure is present at the end of construction. The end of construction condition is applicable to both the riverside and landside slopes.

b. Case II - Sudden drawdown. This case represents the condition whereby a prolonged flood stage saturates at least the major part of the upstream embankment portion and then falls faster than the soil can drain. This causes the development of excess pore water pressure which may result in the upstream slope becoming unstable. This case is not considered because as a flood dissipates, the soils will have enough time to drain.

c. Case III - Steady seepage from full flood stage (fully developed phreatic surface). This condition occurs when the water remains at or near full flood stage long enough so that the embankment becomes fully saturated and a condition of steady seepage occurs. This condition was critical for levee landside slope stability.

d. Case IV - Earthquake. Earthquake loadings will not be considered in analyzing the stability of levees because, as mentioned earlier, the probability of an earthquake occurring in this locality is very low.

Soils parameters for various formations are shown below in Table C-3 for both the Grand Forks/East Grand Forks project and the Ada borings. The soils parameters used for the Grand Forks/East Grand Forks project are based on many more tests, so the unit weights were used in this analysis. However, strengths for the Upper Brenna Formation and levee fill from Ada project borings were used because they govern and they are site specific, for the Brenna case. In the case of levee fill, borrow for the Ada project will come entirely from Alluvial Deposits and the Sherack Formation, which have an insitu internal-angle-of-friction of 30 degrees. Stability required 21 degrees and having the compacted strength 10 degrees less then the insitu strength seemed overly conservative.

Table C-3

Grand Forks/East Grand Forks (Ada) testing results	UNIT WEIGHTS		Q-STRENGTHS (UU)		S-STRENGTH (CD)	
	MOIST	SATURATED	C in psf	ϕ in degrees	c' in psf	ϕ' in degrees
LEVEE FILL/SPOIL	122	122	700	0	0	20 (21)
ALLUVIAL DEPOSITS	122	122	1000	0	0	30
SHERACK FORMATION	102	103	1000	0	0	30
POPLAR RIVER FORMATION	112	112	1000	0	0	22
UPPER BRENNA FORMATION	97	97	720 (345)	0	0	13 (12.7)

Cases I and III apply to the new levee and only Case III (during low water which would be the normal and the worst case) applies to the rerouted Judicial Ditch No. 51 (JD 51). These were analyzed using the computer program SLOPE/W with the soil stratigraphy from the closes boring to the site being analyzed. Slope stability was done using the strengths shown in Table C-3 above. For Case I (end-of-construction), Q-strengths were used; for Case III (long-term-seepage), S-strengths were used for both the new levee side slopes and the JD 51 side slopes. Results of the stability analysis are shown on Plate C-6. The steepest stable slope computed for the levee was 1V:4H and for the excavated slope of JD 51 was 1V:6H both of which are not unusual in the Red River Valley. Currently, the slopes on JD 51 are not stable because they're steeper then 1V:6H. JD 51's existing alignment does not have adequate room for stable slopes which is why rerouting it was required. Various alignments were investigated to select the least costly. The factor-of-safety (FS) against stability failure for the both the levee and JD 51 slopes for applicable Cases are shown in Table C-4 below. These factors-of-safety were checked and confirmed by the computer program UTEXAS 4.

Table C-4

Levee Slopes			computed	required	JD 51			computed	required
A. Case I for 1V:4H slope	Q-strengths	FS=	4.8	1.3	A. Case I for 1V:6H slope	Q-strengths		Not Applicable	
B. Case III for 1V:4H slope	S-strengths	FS=	1.4	1.4	B. Case III for 1V:6H slope	S-strengths	FS=	1.4	1.4

13. SETTLEMENT AND DISPLACEMENT:

The potential settlement of the levee was estimated using two equations in the Second Edition of "Principles of Foundations" by Braja Das. In areas where the existing ground could be considered flat, the vertical stress increase caused by the construction of the levee was computed using equation 3.97 on page 179 and shown below. Using this stress increase, the one-dimensional consolidation settlement was computed with the equations on page 168 and shown below. This was computed for levee heights varying from two to 13 feet. The polynomial regression was then done, resulting in a sixth-degree polynomial which would yield the expected amount of settlement given the height of levee. The computer program CSETT was used to check the results and, also, to compute the settlement in areas where two-dimensional affects are large (where the existing ground is not flat). The five consolidation tests that were done for this project resulted in C_c and e_0 that varied by the formation sampled, as shown in the testing summary above in Paragraph 5 Table C-1 above. The values are consistent with testing done in other areas of the Red River valley. Soil stratigraphy from boring no. 06-11M was considered representative. The ultimate primary settlement was used with an over-consolidation-ratio of 5.0 for the Sherack Formation and 2.0 for the Brenna Formation which is less-then or equal to what was used for the East Grand Forks project, according to the DDR. The levee depth was taken as the maximum amount of fill added for a given reach of levee. When the fill depth for a reach resulted in an expected settlement of greater-than 4.1 inches, the levee height for the whole reach was overbuilt by 6-inches as shown in Table C-5 below. Less than 4.1 inches was considered a maintenance issue. Settlement was computed in this simplified way to compare costs of various plans at various levels of protection. Thus, settlement could be considered without spending large amounts of time required for a site specific analysis on many plans which would not be selected. Much of the foot print of the proposed levee alignment is either currently being farmed or has an existing levee on it. For this reason, it was assumed that no displacement would occur during the construction of this project.

Stress Increase Under an Embankment

Figure 3.39 shows the cross section of an embankment of height H. This is a two-dimensional loading condition. The vertical stress increase caused by the embankment loading condition can be expressed as

$$\boxed{\Delta p = \frac{q_o}{\pi}\left[\left(\frac{B_1 + B_2}{B_2}\right)(\alpha_1 + \alpha_2) - \frac{B_1}{B_2}(\alpha_2)\right]} \tag{3.97}$$

where $q = \gamma H$

γ = unit weight of the embankment soil

H = height of the embankment

$$\alpha_1 \text{ (radians)} = \tan^{-1}\left(\frac{B_1 + B_2}{z}\right) - \tan^{-1}\left(\frac{B_1}{z}\right) \tag{3.98}$$

$$\alpha_2 = \tan^{-1}\left(\frac{B_1}{z}\right) \tag{3.99}$$

$$S_c = \frac{C_c H_c}{1 + e_o} \log \frac{p_o + \Delta p_{av}}{p_o} \quad \text{(for normally consolidated clays)} \qquad (1.65)$$

$$S_c = \frac{C_s H_c}{1 + e_o} \log \frac{p_o + \Delta p_{av}}{p_o} \quad \text{(for overconsolidated clays with } p_o + \Delta p_{av} < p_c\text{)} \qquad (1.67)$$

$$S_c = \frac{C_s H_c}{1 + e_o} \log \frac{p_c}{p_o} + \frac{C_c H_c}{1 + e_o} \log \frac{p_o + \Delta p_{av}}{p_c} \quad \text{(for overconsolidated clays with } p_o < p_c < p_o + \Delta p_{av}\text{)} \qquad (1.69)$$

where p_o = average effective pressure on the clay layer before the construction of the foundation
Δp_{av} = average increase of pressure on the clay layer caused by the foundation construction
p_c = preconsolidation pressure
e_o = initial void ratio of the clay layer
C_c = compression index
C_s = swelling index
H_c = thickness of the clay layer

Table C-5 :Proposed Levee Overbuild

Levee Reach	50-yr.	100-yr.	200-yr.	500-YR.
1	NO Overbuild	NO Overbuild	NO Overbuild	NO Overbuild
2	6 inch Overbuild	6 inch Overbuild	6 inch Overbuild	6 inch Overbuild
3	NO Overbuild	NO Overbuild	NO Overbuild	NO Overbuild
4	NO Overbuild	NO Overbuild	NO Overbuild	NO Overbuild
5	NO Overbuild	NO Overbuild	NO Overbuild	NO Overbuild
6	NO Overbuild	NO Overbuild	NO Overbuild	NO Overbuild
7	NO Overbuild	6 inch Overbuild	6 inch Overbuild	6 inch Overbuild
8	NO Overbuild	NO Overbuild	NO Overbuild	NO Overbuild

14. SEEPAGE

Seepage is a concern in the area of boring no. 06-7M which is the only boring which contains sands. Seepage calculations are shown on Plate C-7. The FS against gradients exceeding the critical gradient is 1.4. Uncertainty's in seepage parameters and the consequences of piping dictate that this FS be at least 3.0. Additions to the design of the levee to increase the FS, as mentioned by EM 1110-2-1913 chapter 5, include seepage control measures such as a (a) cutoff trench, (b) riverside impervious blanket, (c) landside seepage berm, (d) pervious toe trench, and (e) pressure relief well(s).

15. CONSTRUCTABILITY:

Excavation of the rerouted portion of JD 51 will have to be done carefully so as to not disturb the soil making up the side slopes and bottom of the ditch. This would weaken the insitu soils, reducing the FS against slope failure.

16. ROCK GRADATION:

The calculation of the minimum weight of the 50 percent-less-than-by-weight rock for the rockfill is explained in the Hydraulic Appendix. Layer thickness is 18-inches with the gradation shown on Plate C-8 and in the table below.

Table: C-6

Percent Less-than-by-Weight:	Maximum (lbs.)	Minimum (lbs.):
100	300	100
50	120	40
15	25	8

17. FUTURE WORK:

Now that a plan is selected and degree-of-protection established, the following will have to be done for plans and specifications:

1.) A site specific settlement analysis will have to be completed:
   a. Consolidation settlement will be accounted for in the final pipe grades of storm water outfalls through the levee.
   b. Consolidation settlement will be added to the levee height where needed, instead of by reaches of levee.
2.) Layout and complete additional borings to:
   a. More precisely define the extent of under seepage concern.
   b. Define the soil parameters at all structures.
   c. Investigate HTRW concerns.
3.) Design gradient control measures to increase the FS to 3.0 against exceeding the critical gradient.
4.) Additional investigations will be necessary prior to plans and specifications in order to accurately quantify the amount as well as the quality of stone product

available within a reasonable radius of the area.
5.) Work with Hydraulics to decide where to place of riprap.
6.) Decide what to use for a filter/bedding for riprap.

18. CREDIT TO EXISTING LEVEES

Ada, MN Feasibility Study

31 OCTOBER 2007

Introduction

1. This document is part of the Ada, MN Flood Risk Management Feasibility Study. The town of Ada currently has a system of levees protecting against the Marsh River on the south side of town and Judicial Ditch 51 (JD 51) on the north side of town. The purpose of this document is to assess the condition of the existing levees, and to determine the baseline level of protection that the existing levees provide to the town.

Existing Conditions

2. There are 2 levees protecting the town of Ada. For the purposes of this report, they will be called the south levee and the north levee. The south levee is broken into two reaches divided by Hwy 9. The west reach runs from high ground near South Jamison Dr on its west end to Hwy 9 on its east end. The east reach runs from Hwy 9 on its west end to the golf course on its east end. The north levee starts at high ground by Hwy 9 on its west end, runs east alongside JD 51 to just east of 9th St East, then turns south, terminating at high ground north of Hwy 200 behind some businesses. See Figure 1 after the appendix Plates for the location of the existing levees.

3. The south levees were initially constructed under flood emergency conditions. These levees have been improved in the time since their construction, with the most recent improvements made in 2003. Improvements consisted of adding fill to raise the levee crest and to flatten the levee side slopes to 1V:4H. The new levee has a top width of at least 10ft for its entire length. In 2004, existing culverts that were damaged in the 2002 flood were replaced with new sluice gate control structures. Flap gates were also installed on all new culvert outlets. At no time was an inspection trench constructed to help locate possible underground pipes or buried culverts below the footprint of the levee. Chapter 7-2.f of EM 1110-2-1913 discusses the need for inspection trenches prior to levee construction. Both reaches of this levee are well maintained.

4. The majority of the existing north levee was constructed in 1998. A portion of the levee, from the MNDOT shop building to the apartment building at the intersection of Lily Lane and Daisy Lane, consists of spoil material placed on top of the bank during the construction of Judicial Ditch 51. It is not known when the spoil material was placed on top of the bank here. It is also not known whether or not the portions of levee constructed in 1998 were built on existing

spoil material. The levee has 1V:4H side slopes and a 10ft top width. In 2004, 4 new control structures with sluice gates were constructed on this levee to replace old ungated culverts. The north levee has a few problems. The biggest problem is the presence of landslides in 2 different areas on the levee. One of the slides has a vertical drop of about 5ft or 6ft. The slides were most likely caused by placing too much fill on top of the bank near JD 51. The slide areas have affected 2 of the new control structures making them unable to perform as designed. The levee also runs through the backyards of private residences for a long stretch. In this area, the levee has many trees and private encroachments (sheds, gardens, fences, etc.) on it.

Analysis

5. The town was divided into 6 areas for the level of protection analysis, 3 for the north levee and 3 for the south levee. For simplicity, the areas were named similar to that for the economic analysis model. Figure 1 shows the town of Ada divided into the analysis areas. The following is a short description of each area:

Area 1A: North levee, between 4th St E and 9th St E, follows hydraulic reference pt D (from initial feasibility report dated 14 August 2001)
Area 1B: North levee, between Hwy 9 and 4th St E, follows hydraulic reference pt C
Area 2A: South levee, between 4th St E and 9th St E, follows hydraulic reference pt D
Area 2B: South levee, between Hwy 9 and 4th St E, follows hydraulic reference pt C
Area 3: South levee, between South Jamison Dr and Hwy 9, follows hydraulic reference pt B
Area 4: North Levee, east of 9th St E, follows hydraulic reference pt E

6. <u>North Levee.</u> To analyze the effectiveness of the north levee, the existing condition of the levee comes into play. As stated before, the levee has many deficiencies. The biggest concern is the presence of land slides as the levee follows the alignment of Judicial Ditch 51. Since landslides have occurred on this levee in the past, this means that no credit can possibly be given to the existing north levee. Because assigning a PNP/PFP elevation would imply some credit be given to levees 1A and 1B, a PNP/PFP elevation will not be assigned and levees 1A and 1B will be treated like they do not exist. For reference, PNP is the probable non-failure pt, or the elevation at which the levee is highly likely to not fail. The PFP, or probable failure point, is the elevation at which the levee is highly likely to fail.

7. To estimate the PFP, the existing topography was used to find the lowest point on the landward toe for Area 4. The PNP elevation will coincide with the PFP. See Figure 1 for locations of the low points for each area. The existing topography used for this analysis was the 1-foot contour aerial mapping obtained in 1999 for the initial feasibility study. No changes to the north levee have occurred since the mapping was obtained. The analysis yielded these results: Area 1B = el. 902.0ft, Area 1A = el. 904.0ft, and Area 4 = el. 905.0ft.

8. <u>South Levee.</u> Descriptions of the analysis for Areas 2A, 2B, and 3 on the south levee are described below. The geometry and foundation conditions for Areas 2A and 2B are considered

to be similar, so a combined analysis was conducted for these two areas.

9. Areas 2A and 2B. Areas 2A and 2B are both far enough away from the river channel that, by inspection, slope stability is not a concern. Also, reviewing the generalized stratigraphy developed from borings shows that no sand is present in the foundation in this area. During the 2002 flood, on-site Corps of Engineers personnel encountered no sand in the upper foundation of this area while constructing emergency levees. No seepage problems were encountered during the 2002 flood in this area either. For these reasons, there are no seepage concerns for these areas. The top of the levee in these areas is at el. 905.5ft. Assuming that there was some settlement of the levee crests after construction, and some variability in the top elevation of the levee, assume that the top of the levee is effectively at el. 905.0ft. Therefore the PFP for these areas is 905.0ft. It is assumed that the PNP for the existing levees in these areas will coincide with the PFP since no failure mode besides overtopping is reasonable.

10. Area 3. The existing conditions for Area 3 are a little different than those for Areas 2A and 2B. First, there is a short stretch where the existing levee is close to the bank of the Marsh River, which is a slope stability concern. Also, while constructing the emergency levee in 2002 across the farm field on the west side of Area 3, a sand seam was discovered below the ±1ft of topsoil that was being stripped. This sand seam could mean that seepage is an issue in this area.

11. For the stability analysis, the long-term (drained, steady state seepage) design condition was used, and phreatic surfaces were assumed to be fully developed between the design water level levee on the riverside, and the levee toe on the landside. The section was taken about 400ft west of the railroad tracks at a point where the existing levee is closest to the Marsh River. Soil parameters were assumed to be as follows (MLV = most likely value, V = coefficient of variation, σ = value of 1 standard deviation):

				ϕ (degrees)				
Material	**Top El., ft**	**γ_m (pcf)**	**γ_{sat} (pcf)**	**MLV**	**V[1] (%)**	**σ**	**MLV+σ**	**MLV-σ**
Clay Fill	903.0	111	116	30	9	2.7	32.7	27.3
Alluvium	900.0	123	123	30	9	2.7	32.7	27.3
Sherack Formation	898.0	115	116	30	9	2.7	32.7	27.3
Upper Brenna Formation	890.0	100	100	13	9	1.2	14.2	11.8

[1] The value of V was taken from ETL 1110-2-556, Appendix B, Table 1, pg. B-30
Note: The unit weight of soil was not varied as this parameter has little effect on the results of a stability analysis, plus we have good information for these soils from the abundance of Grand Forks testing data available.

12. Slope stability analysis on the landward side of the levee, using most likely values and a

water surface at the top of the levee on the riverward side, results in a factor of safety of 3.75. Based on this one result, the possibility of a slide on the landward side of the levee during a flood is almost non-existent. There is no need to look further at the landward toe. The location of the slope stability analysis is shown on Figure 1 after the appendix Plates.

13. A second analysis was run in the same location to simulate a slide on the riverward side of the levee during extreme low water conditions. This is a scenario that has caused many slides in the Red River valley, so it would not be surprising for the levee to experience a slide during a non-flood situation that would render it ineffective. The analysis was run using most likely values for phi angles, the water surface was put at el. 891.0ft, and the bottom of the Marsh River was set at el. 890.0ft. The analysis produced a critical slip surface with a factor of safety of 0.84, although the critical slip surface did not pass through the levee prism. However, there were a large number of slip surfaces that passed through the levee prism with factors of safety ranging from 0.95 to 1.0. For this reason, it is safe to assume that a slide through the levee at this location is likely if water in the Marsh River was at an extremely low stage. Based on this conclusion, the levee in this area should not receive any credit, and the ground elevation at the landward toe, el. 900.0ft, should be applied as the PNP/PFP elevation.

14. Before a final PNP/PFP for Area 3 can be determined, the seepage concerns in the area about 350ft from the west end of the levee need to be considered. However, since the ground elevation at the landward toe for the levee in this area, and for this entire east/west stretch of levee in general, is at or above el. 900.0ft, there is no need to perform the analysis since this is also the controlling elevation from the slope stability analysis. However, because of possible seepage concerns (location shown on Figure 1 which comes after the appendix Plates) in this Area, no credit will be assigned and it will be treated as nonexistent.

15. Looking at the topographic layout of the entire town of Ada, the railroad embankment serves as the elevation divide for the town. The lowest elevation on the railroad throughout town that would allow water from Area 3 to spill over to areas east of the railroad is roughly el. 903.0ft. At this elevation water from Area 3 would begin to spill into Areas 2B, and 2A, effectively lowering the PNP/PFP for these areas to el. 903.0ft. However, because all these areas are connected be storm sewers, no credit will be given to these levees either.

16. Reviewing the most current storm water system map for the city, there is a storm water connection under the railroad. This creates the possibility of flood waters traveling through the storm water system from Area 3 to areas east of the railroad. There also is a storm water connection from Area 1B to Area 2B that runs south along 2nd St E. The possibility exists that water entering the storm water system in Area 1B could travel to Area 2B using this conduit. This would effectively lower the credit given to levee 2B to no credit.

Summary

17. Summary of Analysis. The following table summarizes the credit to existing levees for each area shown on figure 1 after the appendix Plates.

AREA	PNP/PFP (ft)
1A	No Credit*
1B	No Credit*
2A	No Credit*
2B	No Credit*
3	No Credit*
4	No Credit*

* - Should be treated like they do not exist

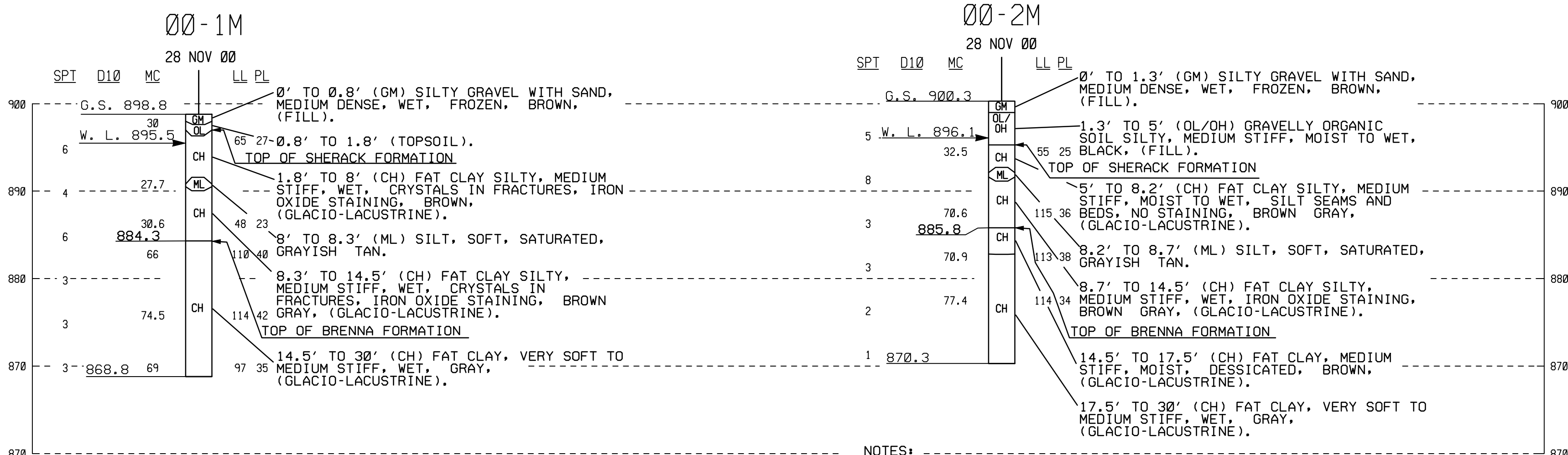
Ada Sec. 205 Flood Control Project

St. Paul District
US Army Corps of Engineers®

Boring Location Map

0 500 1,000 2,000
Feet

PLATE C-1

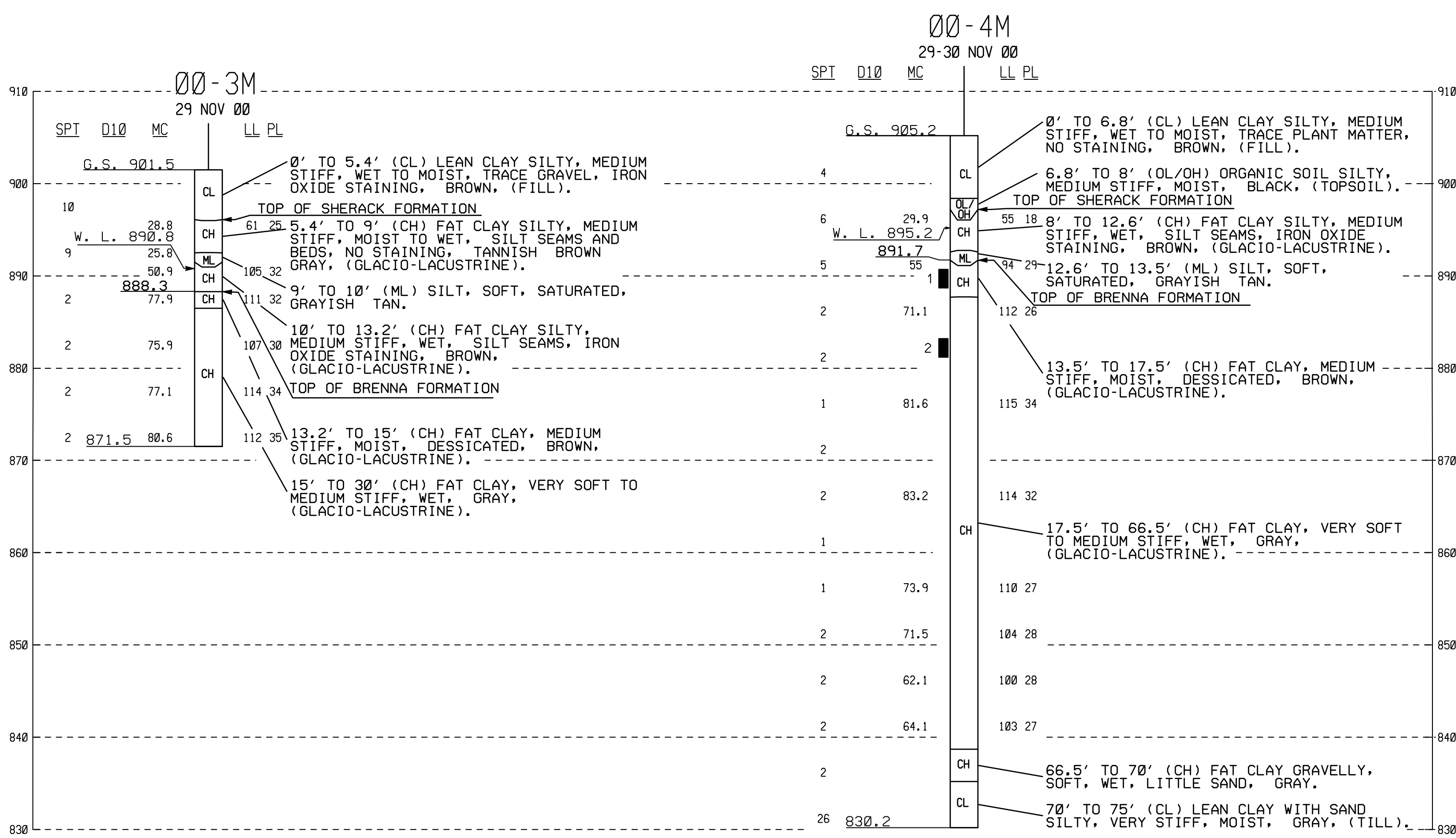


NOTES:

1. WATER LEVEL DETERMINED IN OFFSET HOLE DRILLED TO 15-FEET. AFTER 3 HOURS HOLE CAVED TO 4.9-FEET
2. 3 ¼ I.D. HOLLOW STEM AUGER SET TO ELEVATION 874.9.
3. BORING BACK FILLED WITH MEDIUM BENTONITE CHIPS.

NOTES:

1. WATER LEVEL DETERMINED IN OFFSET HOLE DRILLED TO 15-FEET. AFTER 18 HOURS HOLE CAVED TO 6.8-FEET
2. PETROLEUM ODOR ENCOUNTERED FROM 1 TO 5 FEET.
3. 3 ¼ I.D. HOLLOW STEM AUGER SET TO ELEVATION 875.3.
4. BORING BACK FILLED WITH MEDIUM BENTONITE CHIPS.



NOTES:

1. WATER LEVEL DETERMINED IN OFFSET HOLE DRILLED TO 15-FEET. AFTER 2.5 HOURS HOLE CAVED TO 12.1-FEET
2. 3 ¼ I.D. HOLLOW STEM AUGER SET TO ELEVATION 876.5.
3. BORING BACK FILLED WITH MEDIUM BENTONITE CHIPS.

NOTES:

1. WATER LEVEL DETERMINED IN OFFSET HOLE DRILLED TO 15-FEET. AFTER 21 HOURS HOLE CAVED TO 12.6-FEET
2. 3 ¼ I.D. HOLLOW STEM AUGER SET TO ELEVATION 835.2.
3. BORING BACK FILLED WITH TREMIED CEMENT-BENTONITE GROUT.

GENERAL BORING LEGEND

84-1M	YEAR OF BORING-BORING NUMBER, BORING TYPE (EG: M=MACHINE, A=AUGER, TP=TEST PIT, P=PIEZOMETER).
1 MAY 1984	DATE OF BORING
W.S. 1026.7	WATER SURFACE ELEVATION ON DAY OF BORING
G.S. 1020.2	GROUND SURFACE ELEVATION AT BORING
GW	WELL GRADED GRAVELS, GRAVEL - SAND MIXTURE, LITTLE OR NO FINES
GP	POORLY GRADED GRAVELS, LITTLE OR NO FINES
GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
SM	SILTY SANDS, SAND - SILT MIXTURES
SC	CLAYEY SANDS, SAND - CLAY MIXTURES
ML	INORGANIC SILTS, LIQUID LIMIT LESS THAN 50
MH	INORGANIC SILTS, LIQUID LIMIT GREATER THAN 50
CL	INORGANIC CLAYS, LOW TO MEDIUM PLASTICITY, LIQUID LIMIT LESS THAN 50
CH	INORGANIC CLAYS, HIGH PLASTICITY, LIQUID LIMIT GREATER THAN 50
OL	ORGANIC SILTS OR CLAYS, LOW PLASTICITY, LIQUID LIMIT LESS THAN 50
OH	ORGANIC SILTS OR CLAYS, MEDIUM TO HIGH PLASTICITY, LIQUID LIMIT GREATER THAN 50
PT	PEAT
SP-SM	BORDERLINE MATERIAL
SP& SM	STRATIFIED MATERIAL
I	LOCATION AND SAMPLE NUMBER FOR UNDISTURBED SAMPLE
	NO RECOVERY
W.L. 726.7	WATER LEVEL ON DATE OF BORING
700.1	ELEVATION AT BOTTOM OF BORING
(238.56)	ELEVATION IN METERS

GENERAL BORING NOTES

1. GENERAL: THE UNIFIED SOIL CLASSIFICATION SYSTEM IS USED TO IDENTIFY BASIC SOIL TYPE. THE LEGEND REPRESENTS ONLY THE BASIC SOILS. TO COMPLETE THE CLASSIFICATION, PERTINENT INFORMATION IS ADDED TO THE RIGHT OF THE BORING STAFF. NOTES PERTAINING TO A SPECIFIC BORING ARE SHOWN BELOW THE BORING STAFF.

2. MOISTURE CONTENT: THE NATURAL MOISTURE CONTENT IN PERCENT OF DRY WEIGHT (MC) IS SHOWN TO THE LEFT OF THE BORING STAFF.

3. BLOW COUNT (SPT): BLOW COUNTS ARE SHOWN TO THE LEFT OF THE BORING STAFF AND, EXCEPT AS NOTED, ARE THE NUMBER OF BLOWS NECESSARY TO DRIVE THE SAMPLER USED A DISTANCE OF 12-INCHES. STANDARD BLOW COUNTS ARE FOR A STANDARD PENETRATION TEST (SPT) USING A 1⅜ X 2-INCH SAMPLER, 140 LB. HAMMER, AND A 30-INCH DROP. FOR NON-STANDARD BLOW COUNTS SAMPLER SIZE, HAMMER WEIGHT, AND HEIGHT OF DROP ARE AS SHOWN.

4. ATTERBERG LIMITS: LIQUID LIMIT (LL) AND PLASTIC LIMIT (PL) ARE SHOWN TO THE RIGHT OF THE BORING STAFF.

5. D 10 SIZE: THE GRAIN SIZE IN MILLIMETERS OF WHICH 10% OF THE SAMPLE IS FINER IS SHOWN TO THE LEFT OF THE BORING STAFF.

6. RQD: ROCK QUALITY DESIGNATION (RQD) IS SHOWN TO THE LEFT OF THE PERCENT RECOVERY COLUMN. RQD IS THE PERCENT RECOVERY CONSISTING OF UNBROKEN PIECES LONGER THAN 4-INCHES.

7. % RECOVERY: PERCENT CORE RECOVERY IS SHOWN TO THE LEFT OF THE BORING STAFF. PERCENT RECOVERY IS LENGTH OF CORE RECOVERED/LENGTH OF CORE CUT X 100. UNLESS SPECIFIED OTHERWISE, ALL CORE IS 4-INCH DIAMETER.

8. ELEVATIONS REFERENCED TO N.G.V.D. 1929 ADJ. UNLESS SPECIFIED OTHERWISE.

9. THE BORINGS SHOW SUMMARIES OF INFORMATION RECORDED ON THE ORIGINAL FIELD LOGS. THESE LOGS ARE AVAILABLE FOR INSPECTION AT THE ST. PAUL DISTRICT OFFICE. ARRANGEMENTS TO INSPECT LOGS CAN BE MADE BY CALLING (651) 290-5599.

US Army Corps of Engineers® St. Paul District

MARK	DESCRIPTION	DATE	APPR

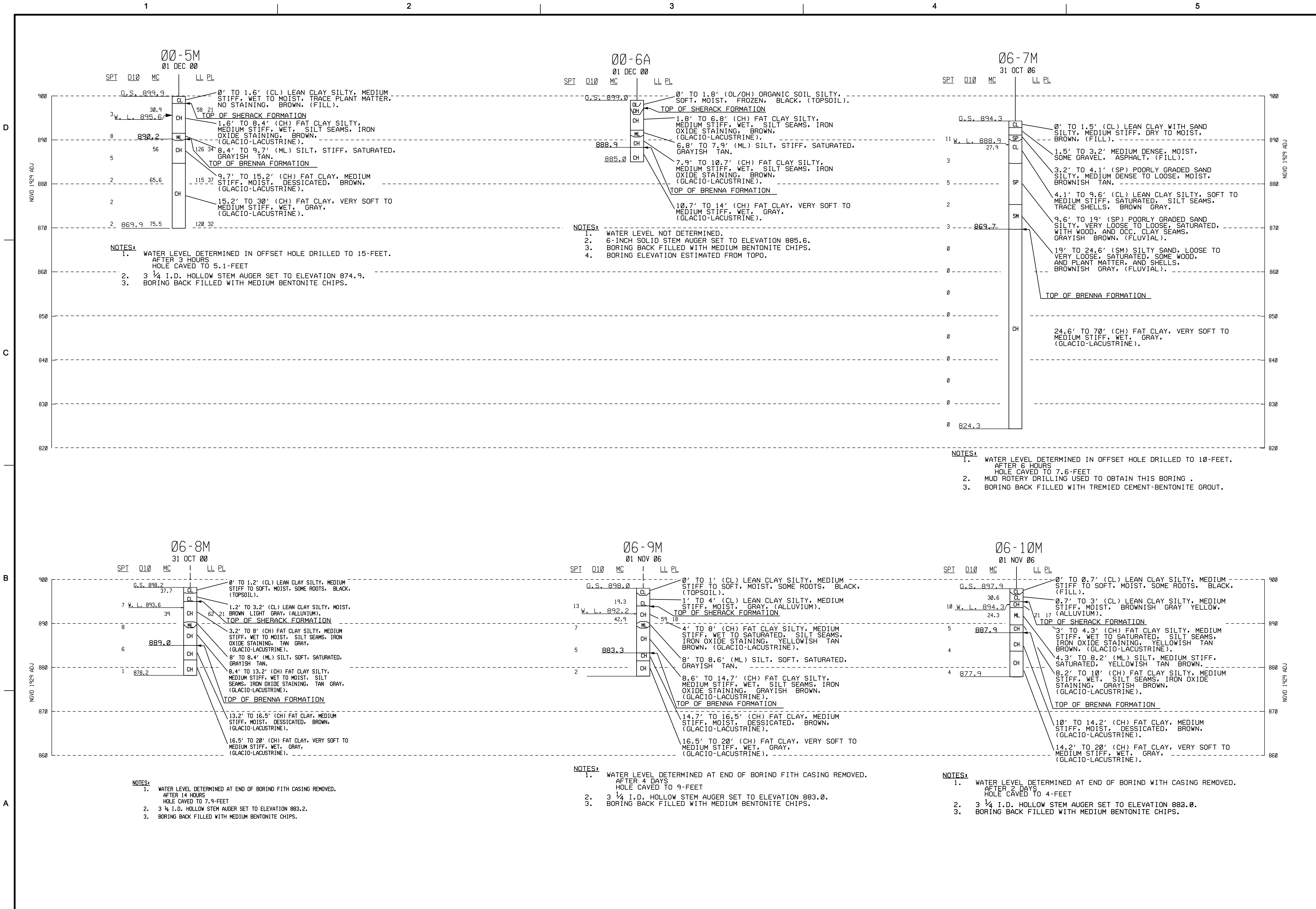
DESIGNED BY: KSN | DWN BY: GRS | CKD BY: KSN/GAR | SUBMITTED BY: | PLOT SCALE: 1:1 | PLOT DATE: No | SIZE: | FILE NAME: ADA\NED\04B2002_B-200200.dgn | DATE: 2006/03/21 | SOLICITATION NO.: | CONTRACT NO.: | FILE NUMBER:

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

FEASIBILTY STUDY - ADA, MINNESOTA
WILD RICE & MARSH RIVERS
ADA, MINNESOTA

BORING NOTES, LEGEND, & GENERAL BORING LOGS: 00-1M, 2M, 3M & 00-4M NED PLAN

SHEET IDENTIFICATION
B-002



US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

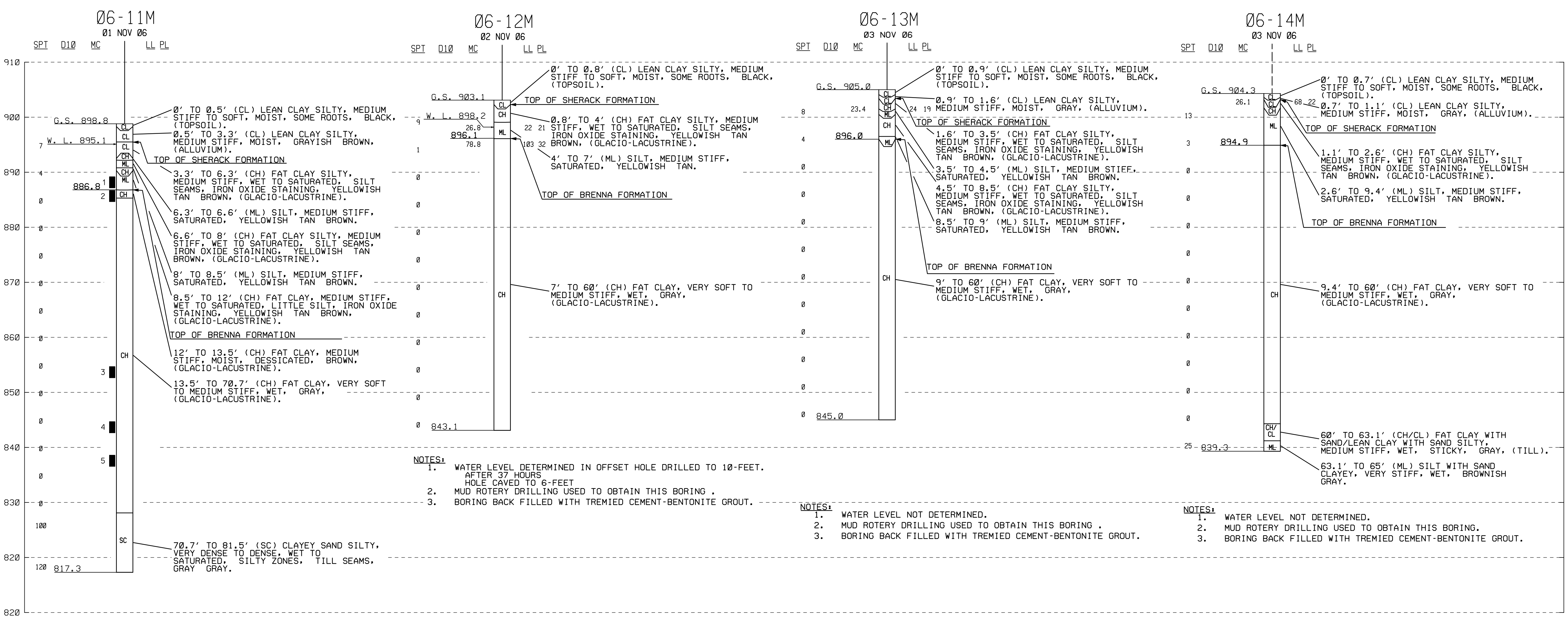
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT

DESIGNED BY: KSN		DATE: 2008/03/21
DWN BY: GRS	CKD BY: KSN/GAR	SOLICITATION NO.:
SUBMITTED BY:		CONTRACT NO.:
PLOT SCALE: 1:1	PLOT DATE: No	FILE NUMBER:
SIZE:	FILE NAME: ADANED04B2003__B-200300.dgn	

FEASIBILTY STUDY - ADA, MINNESOTA
WILD RICE & MARSH RIVERS
ADA, MINNESOTA

BORING LOGS: 00-5M, 6A, 06-7M, 06-8M, 9M, 10M & 11M
NED PLAN

SHEET IDENTIFICATION
B-003



NOTES: (06-11M)
1. WATER LEVEL DETERMINED IN OFFSET HOLE DRILLED TO 10-FEET. AFTER 18 HOURS HOLE CAVED TO 4.6-FEET
2. MUD ROTERY DRILLING USED TO OBTAIN THIS BORING .
3. BORING BACK FILLED WITH TREMIED CEMENT-BENTONITE GROUT.

NOTES: (06-12M)
1. WATER LEVEL DETERMINED IN OFFSET HOLE DRILLED TO 10-FEET. AFTER 37 HOURS HOLE CAVED TO 6-FEET
2. MUD ROTERY DRILLING USED TO OBTAIN THIS BORING .
3. BORING BACK FILLED WITH TREMIED CEMENT-BENTONITE GROUT.

NOTES: (06-13M)
1. WATER LEVEL NOT DETERMINED.
2. MUD ROTERY DRILLING USED TO OBTAIN THIS BORING .
3. BORING BACK FILLED WITH TREMIED CEMENT-BENTONITE GROUT.

NOTES: (06-14M)
1. WATER LEVEL NOT DETERMINED.
2. MUD ROTERY DRILLING USED TO OBTAIN THIS BORING.
3. BORING BACK FILLED WITH TREMIED CEMENT-BENTONITE GROUT.

US Army Corps of Engineers® St. Paul District

MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: KSN | DATE: 2006/03/21
DWN BY: GRS | CKD BY: KSN/GAR | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
PLOT SCALE: 1:1 | PLOT DATE: No | FILE NUMBER:
SIZE: | FILE NAME: ADA\NED\NED2004_B-200400.dgn

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

FEASIBILTY STUDY - ADA, MINNESOTA
WILD RICE & MARSH RIVERS
ADA, MINNESOTA

BORING LOGS: 06-11M 06-12M, 06-13M & 06-14M
NED PLAN

SHEET IDENTIFICATION
B-004

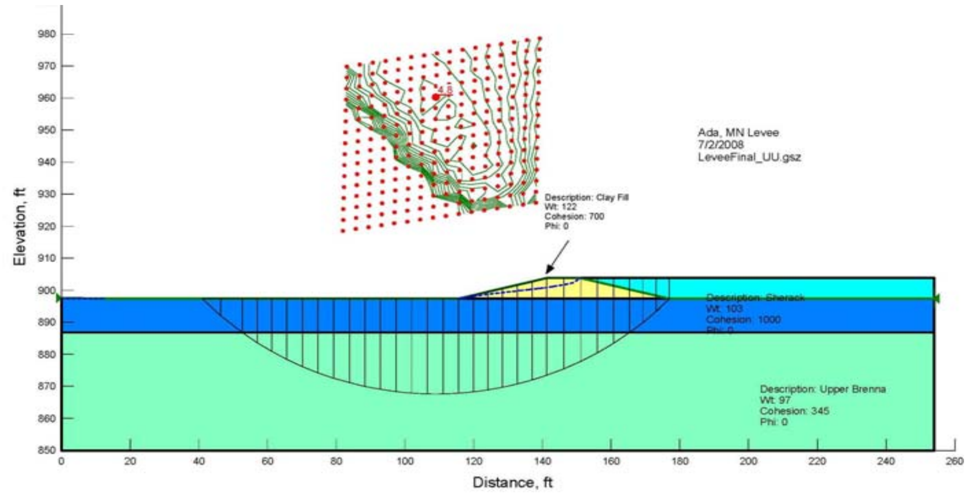
Ada Flood Control Project

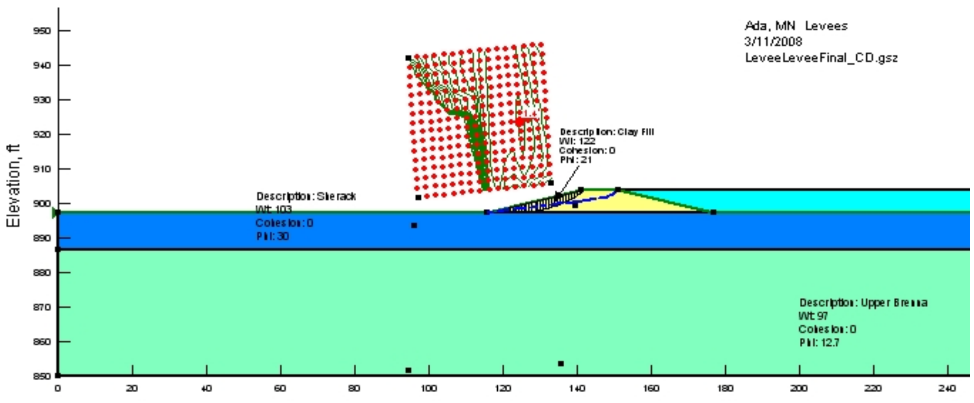
Summary of FS Results

Levee Slopes			Computed	Required	JD 51			Computed	Required
A. Case I	Q-strengths	FS=	4.8	1.3	A. Case I	Q-strengths		Not Applicable	
B. Case III	S-strengths	FS=	1.4	1.4	B. Case III	S-strengths	FS=	1.4	1.4

Summary of Soil Shear Strengths

	Unit Wieght		Q-Strengths		S-Strengths	
Soils	Saturated	Moist	Cohesion	Phi angle	Cohesion	Phi angle
Clay Fill	122	122	700	0	0	21
Sherack	103	102	1000	0	0	30
Upper Brenna	97	97	345	0	25	12.7





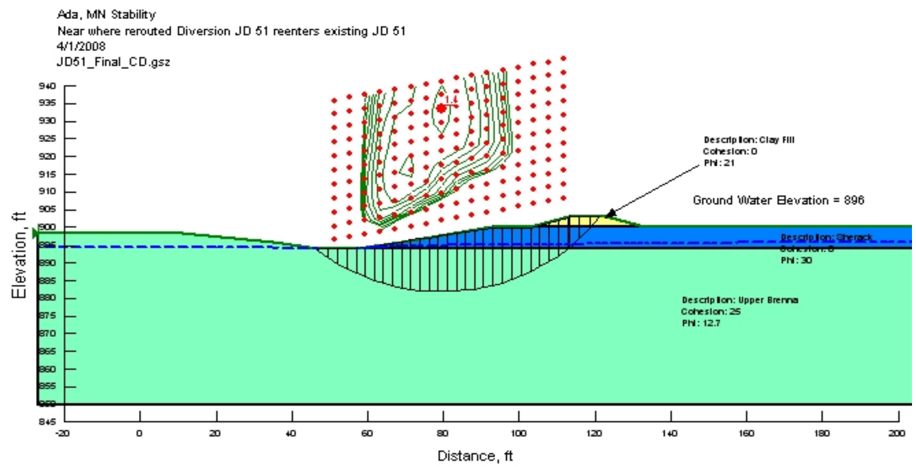


PLATE C-5

ADA MN Seepage

from ETL 1110-2-555 "Design Guidance on Levees" Nov. 1997, p. 2-1

Sherack: $\gamma_{sat} =$ 122 lbs./ft^3

$\gamma_{water} =$ 62.4 lbs./ft^3

$i_{cr} = (\gamma_{sat} - \gamma_{water})/\gamma_{water} =$ 0.96

$FS = i_{cr} / i_y$ $\quad i_{yrequired} = i_{cr} / FS_{required} = 0.96 / 3 =$ 0.318

$FS_{required} = 3$

$i_{yAct.} = 0.68$

$FS_{act} = i_{cr} / i_y =$ 0.96 / 0.318 = 1.40

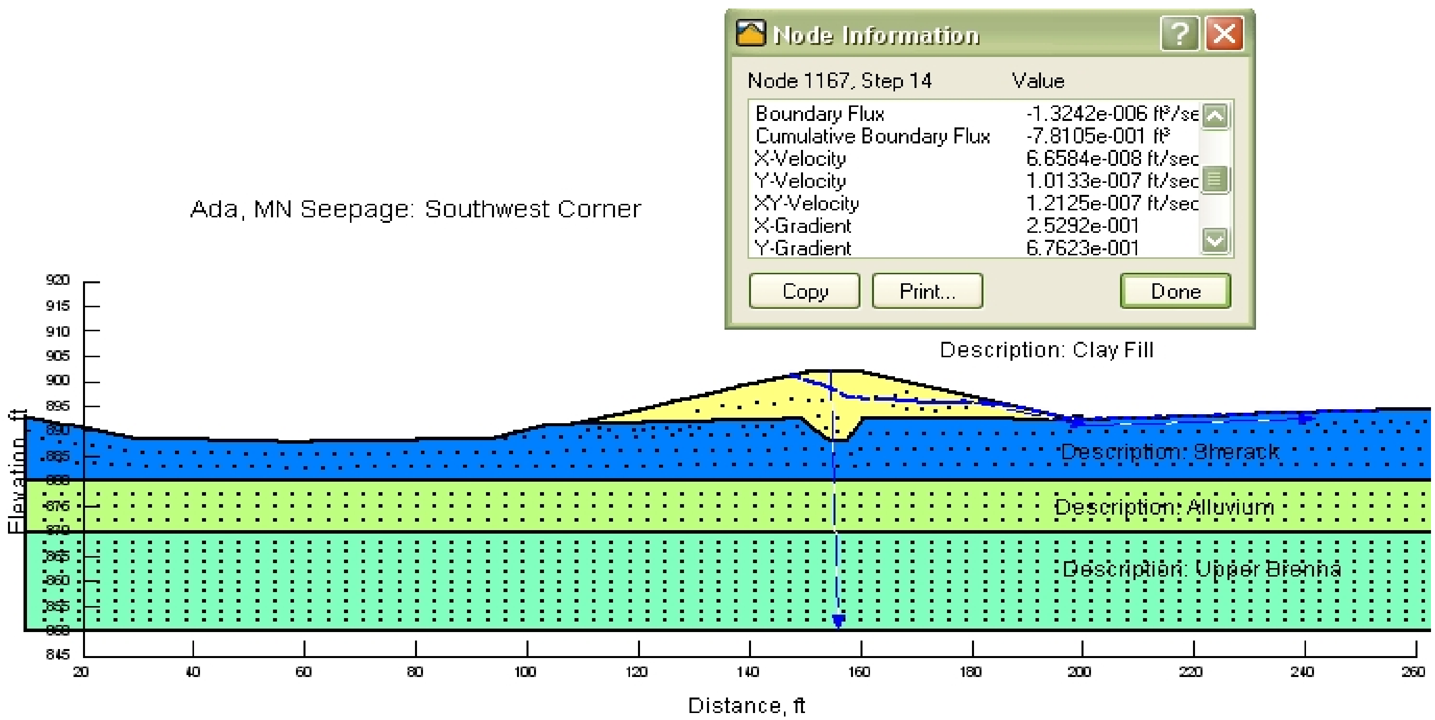
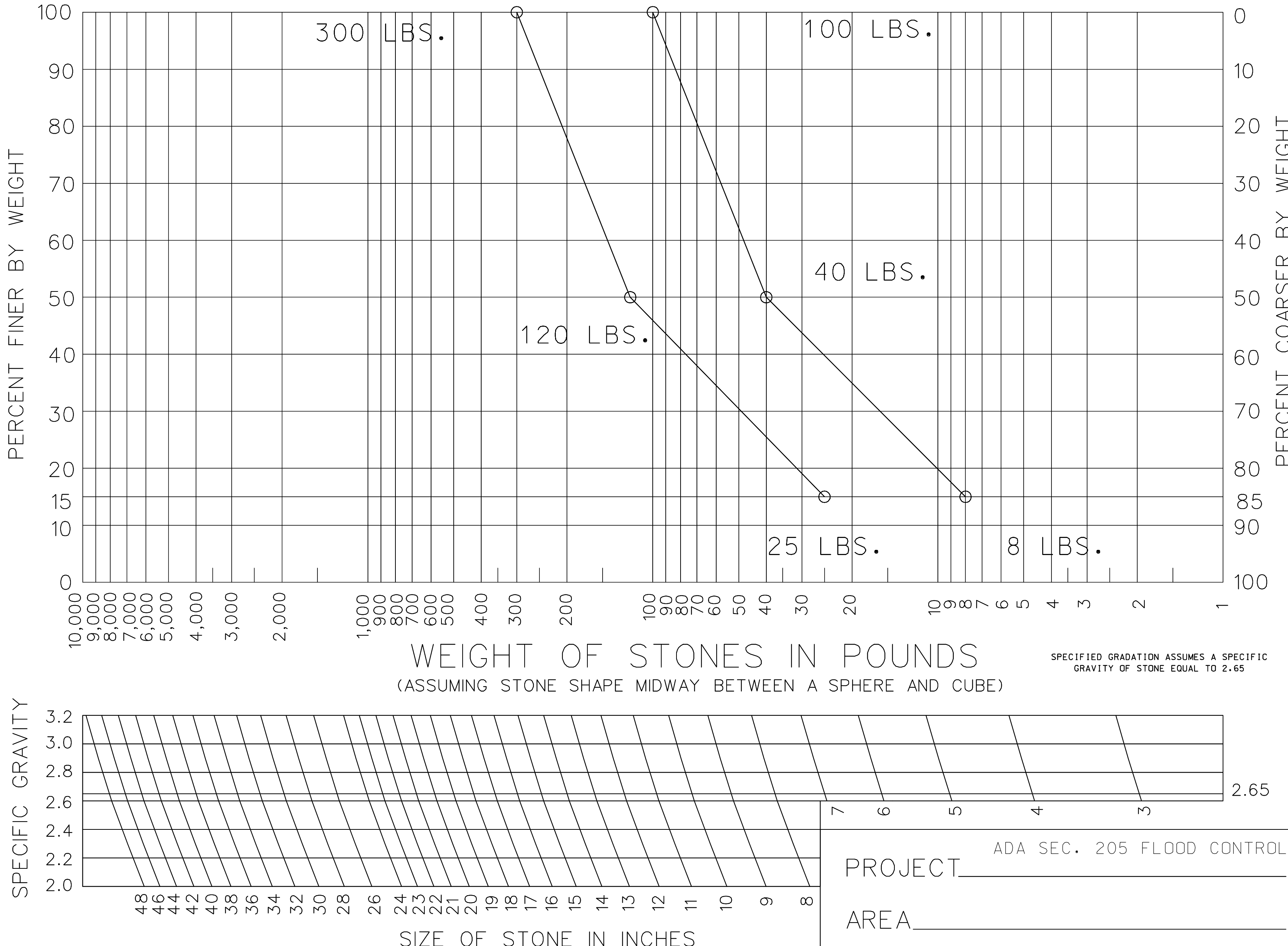


PLATE C-6



SPECIFIC GRAVITY OF STONE =

PROJECT ADA SEC. 205 FLOOD CONTROL

AREA

DATE

RIPRAP/ROCKFILL GRADATION CURVE

ENG FORM 4055 *COMPUTER GENERATED*
APRIL 2003

PLATE C-7