Van White Memorial Boulevard
Minneapolis, Minnesota

Geotechnical Engineering Design Analysis Report

Phase II Environmental Site Assessment

US Army Corps
of Engineers ®
St Paul District

May 2004

Geotechnical Engineering Design Analysis Report

Bassett Creek Valley Brownfields Redevelopment
Van White Memorial Boulevard
Minneapolis, Minnesota

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April 2004

EXECUTIVE SUMMARY

The U.S. Army Corps of Engineers, St. Paul District completed a geotechnical exploration program and subsequent design analysis report for a proposed road in Minneapolis, Minnesota. The work completed for this report was done under an agreement between the U.S. Environmental Protection Agency and the city of Minneapolis. A series of eleven machine soil borings were advanced along the alignment of the proposed Van White Memorial Boulevard Project. Boring locations were selected to provide a general geologic profile under the proposed road. A total of six undisturbed thin-walled tube samples were obtained in 2003 to evaluate the compressibility and shear strength of the fine-grained soils. The results from the 2003 undisturbed testing were combined with 1987 and 1988 undisturbed testing from the Corps of Engineer's Bassett Creek Flood Control Project that was located along or close to the proposed road alignment.

The primary geotechnical design considerations were the effects of embankment settlement on pavement and bridge structures and the ability to construct a stable embankment configuration. Based on the subsurface exploration and soil testing completed, conventional road construction would produce total and differential settlement values well beyond tolerable levels. For a typical embankment cross-section calculated consolidation settlement would take about 10-years to complete. To occupy a large area of urban real estate with an earth fill and wait 10-years until construction can begin is unrealistic in this day and age. A secondary problem occurs when the embankment height exceeds about 23-feet, the factor of safety against an undrained slope failure drops below an accepted value (1.3). To mitigate against slope failure, staged construction; flatter side slopes; stability berms; or lightweight fill could be used.

A properly designed surcharge load with prefabricated vertical drains could effectively reduce the total and differential settlement of the road embankment to tolerable levels. Assuming a 5-foot surcharge for an 18-foot finished height embankment, with 5-foot pvd spacing, and the surcharge in place until primary consolidation is completed (2-years based on preliminary pvd design), the computed post surcharge settlement was about one-inch. As an option, EPS-geofoam could be incorporated through the highest embankment reaches (Station 713+55 –716+00) to avoid slope stability problems and achieve very small post surcharge settlements. To avoid surcharging the project alignment for 0.5-2 years, an EPS block geofoam option exists. If 4-feet of subgrade weighing 115 pcf is removed, this would offset the pavement, EPS, and soil cover weight resulting in no net load on the foundation soils. Potential disadvantages would include: additional EPS cost, remediation cost of contaminated excavation, and chemical degradation of EPS due to contaminated soil and groundwater.

Finally, for the proposed south bridge location the bearing capacity for a single 12-inch diameter pipe pile of varying depth was determined. The computed allowable pile bearing capacity generally varied from about 35-tons for a 90-foot long pile to 70-tons for a 130-foot long pile founded in the clay till (unit 7B). The pile lengths and bearing stratum assumed for design were consistent with other bridges in the vicinity.

VAN WHITE MEMORIAL BOULEVARD – MINNEAPOLIS, MN

GEOTECHNICAL ENGINEERING DESIGN ANALYSIS REPORT

TABLE OF CONTENTS Page Number

List of Tables

List of Plates

List of Appendices

Van White Memorial Boulevard

Geotechnical Engineering Design Analysis Report

1 PROJECT DESCRIPTION

1.1 Introduction

1.1.1 The U.S. Army Corps of Engineers, St. Paul District completed a geotechnical exploration program and subsequent design analysis report for a proposed road in Minneapolis, Minnesota. The work completed for this report was done under an agreement between the U.S. Environmental Protection Agency and the city of Minneapolis. Ultimately, the city and their civil design consultants are the end users of the geotechnical work completed by the Corps of Engineers.

1.2 Location

1.2.1 Van White Memorial Boulevard is a proposed two lane, parkway style, road that will connect the Harrison Neighborhood, including the Heritage Park Redevelopment with the Lowery Neighborhood near Dunwoody Institute. The proposed project corridor is located in Hennepin County, immediately south-west of downtown Minneapolis in T 29 N, R 24 W, SE ¼ Section 21 and the NE ¼ Section 28 (see Plate 1). The proposed roadway alignment extends north / south about 3,600 feet, from approximately the junction of Fourth Avenue North and Fremont Avenue to the junction of Dunwoody Boulevard and Interstate-394.

1.3 Alignment

1.3.1 The proposed road climbs at a 4% grade beginning just north of the Dunwoody/394 junction to a planned bridge location that spans the existing railroad tracks. The road, located atop an embankment averaging 20-feet high, ascends at a 4% grade through the Minneapolis impound lot. A grade change to-2.4% occurs near the northern side of the impound lot ascending to a second planned bridge location to cross Basset Creek. The road embankment transitions to existing grade at the C.P. Railroad. Plate 2 shows the majority of the project alignment, the proposed project continues to the north along the existing Fremont Avenue.

2 SUBSURFACE EXPLORATION

2.1 General

2.1.1 A series of eleven machine soil borings were advanced along the alignment of the proposed Van White Memorial Boulevard Project. Plate 2 shows the project alignment and the associated boring locations. Boring locations were selected to provide a general geologic profile under the proposed road. Deeper borings were completed at the planned bridge locations to try to identify an adequate bearing stratum.

2.2 Drilling

2.2.1 The soil borings were advanced using continuous sampling methods. A 4-1/4-inch I.D. hollow-stem auger or mud rotary drilling was used to advance the pilot boring. Continuous sampling consisted of repetitions of the following standard sequence: a 2-inch I.D. by 2-1/2-inch O.D. split-barrel sampler was driven or pushed 3-feet. This sampler was then withdrawn from the soil boring, opened, and logged; with jar samples obtained every five feet or change in soil unit. A standard 1-3/8-inch I.D. by 2-inch O.D. split-spoon sampler was then driven 2-feet. A 140-pound hammer dropping a distance of 30 inches drove the standard split-spoon sampler. An automatic hammer was used for all of the soil borings. The number of blows required to drive the standard split-spoon sampler one foot, ignoring the first 6 inches and the last 6 inches of the sample, is termed the Standard Penetration Test (SPT) Number, designated as N. The standard split-spoon sampler was then withdrawn from the soil boring, opened, and logged, with jar samples obtained as described above. The soil boring was then cleaned out to the bottom of the five-foot interval and the sequence repeated. Upon completion, soil borings were backfilled with cement bentonite grout.

2.3 Sampling

2.3.1 The soil samples contained within the split-barrel samplers were visually classified using the Unified Soil Classification System (USCS) and described by a St. Paul District Geologist. Soil unit contacts were generally logged to an accuracy of 0.1-feet. Jar samples from the split-barrel samplers were obtained to complete laboratory index testing and verify the field USCS classifications.

2.3.2 Undisturbed samples were obtained from holes adjacent to the soil boring pilot hole. The undisturbed samples were five-inch diameter thin-walled tubes whenever possible; otherwise 3-inch diameter tubes were obtained. This sampling procedure permits selecting sampling depths on the basis of information shown on the pilot boring log and therefore, can provide samples more representative of a given material type. Where this procedure was used the offset boring number normally appears on individual laboratory test results; however, the pilot boring log shows subsurface conditions. The boring number of the pilot hole is used to show the location of both borings in plan view.

3 GROUNDWATER

3.1 Groundwater Observation Holes

3.1.1 Groundwater elevations were usually determined by drilling an offset groundwater observation hole at each soil boring location. This consisted of setting 4-inch hollow stem auger to a depth of 10 to 15-feet and allowing the water level inside the casing to stabilize over a time period as long as practical for that location.

3.2 Project Groundwater Levels

3.2.1 Groundwater levels generally varied between 5 and 10-feet below grade along the project alignment when measured in Unit (1). The groundwater elevations identified in the Van White soil borings appear to be consistent with the peizometric surface identified in the geologic profile for the Bassett Creek Flood Control Project, Design Memorandum No. 4 (1988) for soil units (1) through (7). Groundwater elevations along the project alignment generally correspond to and vary with the prevailing water surface elevation in Bassett Creek.

4 GENERAL GEOLOGY AND TOPOGRAPHY

4.1.1 The Twin Cities metropolitan area is located within the Central Lowland Physiographic Province of North America. Regional topography is characterized by a flat to gently rolling ground surface composed of a mantle of glacial drift. Prior to the glacial advances of the Pleistocene, the area consisted of broad uplands dissected by deep bedrock valleys. The final two glacial advances that occurred during the Wisconsin Stage of the Pleistocene Epoch filled the pre-glacial valleys with a variety of soils.

4.1.2 The city of Minneapolis lies atop a pile of Paleozoic Era sedimentary rocks. These rocks are arranged in a broad, shallow, almost circular basin that dominates the regional subsurface structure. The tilting of the rock strata is so gentle that primary bedding at the site may be considered essentially horizontal. The bedrock is likely composed of Ordovician Period St. Peter Sandstone or dolomite from the Prairie du Chein Formation. The uppermost bedrock aquifer in the area is the Prairie du Chein – Jordan aquifer with a general groundwater flow direction to the east or towards the Mississippi River.

4.1.3 Above the Paleozoic rocks, and completely covering them in many places, are glacial sediments left behind during the Pleistocene Epoch (1.8 million – 11,000 years ago). Glacial advances between 20,000 and 14,000 years ago left numerous moraines and lake basins giving the area its special hummocky topographic character.

4.1.4 The primary surface drainage feature along the proposed project alignment is Bassett Creek, which is an east/northeast flowing minor tributary of the Mississippi River. After the retreat of the glaciers, the Bassett Creek basin consisted of a hummocky till and outwash plain with scattered ice-block lakes and marshes many of which are now filled with peat, silt and clay. Bassett Creek meanders through this glaciated terrain in a shallow channel less than 50 feet wide. For most of its length, it flows through low areas rather than through a definable valley.

5 SITE GEOLOGY

5.1 General

5.1.1 The geology pertinent to the proposed road alignment is described using a classification system based on data presented in the 1979 U.S. Geological Survey publication Miscellaneous Investigation Series Map 11157, Geologic and Hydrologic Aspects of Tunneling in the Twin Cities Area by Norvitch and Walton. For the Van White Memorial Boulevard Project, the St. Paul District subdivided the basic soils into definable units based on their engineering characteristics and geologic origin, generally consistent with the Bassett Creek Flood Control Project, Design Memorandum No. 4 (1988). This information is summarized in Table 3. Soil units are numbered one (1) through eight (8) with some of the units divided into A and B subunits based on correlative geotechnical properties. Unit 3 encountered during the Bassett Creek subsurface exploration was not encountered during the Van White Memorial Boulevard exploration and was therefore not discussed in this report.

5.1.2 Plate 3 contains the geologic profile along the centerline of the proposed road alignment. Users of the geologic profile and the summarized geotechnical parameters should keep in mind that variations within each soil unit exist due to variables such as: topographic and stratigraphic position, previous and existing land use of the area, and the soils' relationship with groundwater. The variables just mentioned as well as others within this report can influence a soil's behavior. The soil units and their numeric identification follow.

5.2 Fill (1)

5.2.1 Unit (1) is composed of clean to dirty sand and gravel with abundant silt and clay beds, concrete slabs, demolition debris, glass and bricks. The sand is fine to coarse grained, loose, dry to saturated and brown to black colored. It varies in thickness along the alignment from 7-feet to 20-feet, with the thickest deposit located beneath the city's impound lot.

5.2.2 During urbanization, the low-lying swampy areas were filled with materials that were readily available at the time. Groundwater that is present in the fill is typically perched on the underlying natural silts and clays. The dewatering characteristics of the fill are likely poor due to the silty, clayey and variable nature of this unit.

5.3 Undifferentiated Alluvium

5.3.1 Undifferentiated alluvium is composed of clean to dirty sand with occasional silt and clay beds. The sand is fine to coarse grained, loose, dry to saturated, brown colored, and averages approximately 7-feet in thickness. Undifferentiated alluvium is likely a lateral equivalent to the Younger Coarse-Grained Lower Terrace Deposits (Unit 3) encountered along the Bassett Creek Project Alignment.

5.4 Recent Bassett Creek Sediments (2)

5.4.1 Unit (2) is composed of fat to lean clay and clayey silt that is very soft to soft, commonly saturated, organic and highly plastic. Beds with abundant shell fragments are milky gray colored while the remaining unit is green gray to black. The unit was deposited in low swampy areas from post-glacial to recent times. It is associated with the ancient and modern day Bassett Creek as well as the natural and gradual filling of the depressions left by the last glacier. These soils are very compressible and exhibit poor load bearing characteristics. The unit supports a perched water table and is commonly saturated. Unit (2) terminates near the southern start of the road embankment and increases in thickness to the north, reaching a maximum of 32-feet thick at the south side of the city impound lot.

5.5 Lower Terrace Clay (4a & 4b)

5.5.1 Units (4a) and (4b) were deposited on a lower terrace of the Mississippi River when the river flowed at a higher level or were deposited in backwater trapped by a riverward terrace or natural dike. Unit (4a) was not encountered along the project alignment.

5.5.2 Unit (4b) is composed of fat clay with minor beds of silty clay and silt. This unit is soft to very soft, saturated, highly plastic and green gray to dark gray colored. Samples have a massive structure, shiny appearance, commonly form slickensides when sheared, and are extremely sticky. The unit averages 8-feet in thickness and thickens slightly to the north. Unit (4b) terminates south of the impound lot within the railroad right of way. This unit supports a perched water table and in some cases is a confining bed for water in the underlying sands.

5.6 Older Coarse-Grained Lower Terrace Deposits (5)

5.6.1 Unit (5) was deposited on a lower terrace of the Mississippi River when its flow was at a higher level. Unit (5) is composed of silty sand with occasional gravelly, clayey and clean sand beds. This sand is fine to medium grained, loose to medium dense, moist to saturated and gray to brown colored. This unit averages about 20-feet in thickness and thins near the northern terminus of the road embankment.

5.7 Glacioaqueous Clay (6a & 6b)

5.7.1 Units (6a) and (6b) are interpreted to have been deposited during glacial times in an ice contact lake or river environment. Unit (6a) is composed of silty to fat sandy clay with pockets of sand and silt. The clay is medium stiff to stiff, medium to highly plastic and is occasionally laminated. Samples have a distinctive dull appearance and feel moist to the touch. Medium to dark gray color is most common. This unit generally has a higher moisture content and higher plasticity than unit (6b). It averages 10-feet in thickness except near the north side of the impound lot where it approaches 30-feet thick.

5.7.2 Unit (6b) is composed of silty clay with minor beds of silt and fat clay. The clay contains dispersed sand and gravel with interbeds of silty, clayey and sandy glacial drift. The clay is medium stiff to stiff, has medium plasticity, is sparsely laminated and medium gray colored. Samples have a distinctive dull appearance and feel moist to the touch. This clay was intermittently present along the project alignment.

5.8 Glacial Till (7a & 7b)

5.8.1 Units (7a) and (7b) were laid down by the Wisconsin aged glaciers that are responsible for most of the area's topographic features. Unit (7a) was not encountered during the subsurface exploration for the proposed road project. Unit (7b) is composed of gravelly sandy clay with occasional clayey sand beds and boulders. The clay is stiff to very stiff and has low plasticity and moisture content. Fresh samples are gray with abundantly scattered zones that readily oxidize to a red-brown color when exposed to air.

5.9 Glacioaqueous Sand and Gravel (8)

5.9.1 Unit (8) is composed of clean to silty sand and gravel with occasional silt beds and boulders. This unit is fine to coarse grained, dense, saturated, and brown colored. This soil unit was only encountered in boring 03-5M at an approximate elevation of 668-feet.

5.10 Bedrock

5.10.1 Bedrock along the proposed alignment was not encountered in any borings taken by the USACE for this work effort. Available geologic literature and borings by the Minnesota Department of Transportation (MNDOT) indicate that the bedrock is likely composed of Ordovician Period St. Peter Sandstone or dolomite from the Prairie du Chein Formation at depths of at least 200-feet below the ground surface.

6 GEOTECHNICAL DESIGN PARAMETERS

6.1 Index Testing

6.1.1 Geotechnical testing of selected jar samples included Atterberg limits and moisture content of fine-grained soils and mechanical analysis testing of coarse-grained soils. These index tests assist in soil classification, aid in determining the demarcation of the geologic units, and correlate to other geotechnical properties, such as undrained shear strength and consolidation settlement parameters. Appendix A contains a complete set of the soil testing results.

6.2 Undisturbed Testing

6.2.1 Table 1 contains a summary of the undisturbed laboratory soil testing. A total of six undisturbed thin-walled tube samples were obtained in 2003. The results from the 2003 undisturbed testing were combined with 1987 and 1988 undisturbed testing from the Bassett Creek Flood Control Project that was located along or close to the proposed road alignment. The 1987 and 1988 samples were 5-inch diameter piston samples, while the more recent sampling consisted of 5-inch diameter thin-walled tubes and 3-inch diameter thin-walled tubes for the deeper samples.

6.3 Average Index Properties and Unit Weights

6.3.1 The Atterberg limits, moisture content, specific gravity, void ratio, and dry, moist, and saturated unit weights from all applicable undisturbed testing was combined into a single table for the soil formations investigated. Plate 8 through Plate 11 contain the average, maximum, and minimum index properties and unit weights for soil formations 2, 4B, 6A, and 7B. The average unit weights from these soil formations were used in subsequent engineering calculations.

6.4 Effective Shear Strength Parameters

6.4.1 Undisturbed shear strength testing consisted of direct shear, istotropically, consolidated-undrained triaxial compression testing with pore pressure measurements, unconsolidated-undrained triaxial compression testing, and unconfined compression tests. Plate 12 shows the notation and graphically depicts how the shear strength testing was plotted for the drained and undrained shear strength envelopes. The direct shear and ICU triaxial tests with pore pressure measurements were used to define effective (drained) shear strengths. Plate 13 through Plate 16 show the effective shear strength envelopes for soil formations 2, 4B, 6A, and 7B. For soil formations 2, 4B, and 6A, which are normally to lightly overconsolidated, fine-grained soils, peak shear strength envelopes were selected. For soil formation 7B, glacial till, which may be more overconsolidated, the ultimate or post-peak shear strength envelope was selected as more representative of the available drained shear strength. Plate 17 through Plate 22 compile the index properties, unit weights, ultimate shear strength, and peak shear strength information from the direct shear and ICU triaxial data.

6.5 Undrained Shear Strength Parameters

6.5.1 Unconfined compression and UU triaxial tests were used to define total stress (undrained) shear strengths. Plate 23 through Plate 26 display the undrained shear strength envelopes for soil formations 2, 4B, 6A, and 7B. Undrained shear strength data was plotted versus depth of the samples tested. In general, the undrained shear strength envelopes were plotted from the smallest depth to the maximum depth of the formations encountered along the project alignment. Plate 27 through Plate 30 list the index properties, unit weights, and shear strength information from the unconfined compression and UU triaxial data.

6.6 Compressibility

6.6.1 The compressibility of the fine-grained soils was determined by one-dimensional consolidation tests (oedometer). Six consolidation tests were completed in 2003 and grouped with the three consolidation tests completed in 1987 and 1988. Three consolidations tests were completed in soil formation 2, two in 4B, two in

6A, and two in 7B. Plate 31 summarizes the index properties, unit weights, preconsolidation pressure, compression indices, recompression indices, and coefficients of consolidation. The preconsolidation pressure was determined graphically from the laboratory e-log p' plot using the procedure developed by Casagrande (1936). The field consolidation plot used to estimate Cr and Cc was determined by a graphical procedure (Schmertmann 1953). The coefficient of consolidation was estimated by the logarithm of time method at various load increments. Plate 32 and Plate 33 contain estimates of the effective vertical overburden pressure for the 2003 undisturbed soil samples and were used to compute the overconsolidation ratio.

Table 1 - Summary of Undisturbed Laboratory Soil Testing

Boring Number	Sample Number	Depth (feet)	Sample Type(1)	Uncon-fined Compres-sion	Triaxial Compression		Direct Shear	Consol-idation
					UU	CU/pp(2)		
03-6MU	1	20-21.3	5 TWT		X	X		X
03-6MU	2	43-44.5	5 TWT		X	X		X
03-6MU	3	97-99	5 TWT		X		X	X
03-8M	1	75-76.8	3 TWT		X		X	X
03-9M	1	55-56.4	3 TWT		X		X	X
03-9M	2	70-72	3 TWT		X		X	X
88-148MU	1	16-18	5 P		X		X	X
88-148MU	2	22-24	5 P	X			X	
88-148MU	3	27-29	5 P	X			X	X
88-148MU	4	34-36	5 P	X			X	
87-129MU	1	12-14	5 P		X		X	
87-129MU	2	15-17	5 P		X			X

(1) 5 TWT = 5-inch diameter thin-walled tube sample, 3 TWT = 3-inch diameter thin-walled tube sample, and 5 P = 5-inch diameter piston sample.
(2) CU/pp = Isotropically, Consolidated-Undrained Triaxial Compression Test with Pore Pressure Measurement.

7 GEOTECHNICAL DESIGN CONSIDERATIONS

7.1 Settlement

7.1.1 Settlement may take three different forms: immediate, primary consolidation, and secondary consolidation. Immediate or elastic settlement occurs in moist, fine-grained soils and coarse-grained soils in response to an external load. As the name implies this form of settlement occurs quickly, usually during construction and is normally not quantified in design. Primary consolidation settlement results from a volume change (decrease in void ratio) in saturated, fine-grained soils as pore water is expelled in response to an external load, such as embankment fill. Due to the relatively low permeability of fine-grained soils, it may take a significant amount of time (5-10-20+ years) for primary consolidation to occur. With good quality subsurface exploration and soil testing, fairly accurate predictions of the settlement magnitude can be made. The limitations of consolidation theory and difficulty characterizing small or randomly occurring subsurface features (e.g., permeable seams, joints, fissures) make predicting the rate of consolidation settlement more difficult. Secondary consolidation settlement occurs after the excess pore-water pressures generated from an external load have dissipated and results from a plastic adjustment of soil fabrics. Secondary consolidation settlement may become important in certain organic soils and when the time to complete primary consolidation is shortened by vertical drains (Terzaghi et al. 1996).

7.1.2 The geologic profile for the proposed road alignment (Plate 3) shows that soil type 2, the organic, very plastic, silt/clay initiates close to the southern beginning of the road embankment fill. It also indicates that soil type 4B, the soft, fat, clay beneath soil type 2, begins near midspan of the proposed south bridge location over the B.N RR tracks. The geologic profile essentially shows that the thickest sequence of soft, compressible, fine-grained soils corresponds with the highest road embankment fill (north abutment of the south bridge through the city impound lot to Bassett Creek). This means the largest magnitude of settlement should occur near the north bridge abutment of the south bridge. On the north side of Bassett Creek, the geologic profile indicates the soft, compressible soil thickness again increases; however, the road embankment height is decreasing from 12-feet at the proposed north abutment of the north bridge to grade over a distance of 350-feet. Settlement will decrease as the embankment height decreases and the thickness of compressible soils remains constant or decreases.

7.1.3 Primary consolidation settlement of two road embankment cross-sections was analyzed for this report with the computer program CSETT. Plate 34 shows the 28-foot high embankment cross-section located at the proposed north abutment of the south bridge (maximum embankment height) including the soil stratigraphy, settlement parameters, and assumed drainage conditions for consolidating layers. Plate 35 shows the computed ultimate primary consolidation settlement profile beneath the embankment. A limitation of the embankment settlement analysis at this location is the assumption of a plane strain condition. The embankment section ends at the bridge abutment and decreases in height to the north, resulting in an over prediction of stress increases beneath the embankment at this location and a corresponding over prediction of computed settlement.

7.1.4 Plate 36 shows an 18-foot high embankment cross-section located near the center of the impound lot, while Plate 37 shows the computed ultimate primary consolidation settlement profile beneath the embankment. This cross-section should represent more typical or average conditions for the proposed road embankment in terms of embankment height and thickness of compressible soils (types 2 and 4B). While the embankment height decreased by about one-third, the computed settlement decreased by about two-thirds (see Table 2 for settlement results). Plate 35 and Plate 37 also indicate the time for consolidation to occur beneath the centerline of the embankment using Terzaghi's one-dimensional consolidation theory. After 10-years the 28-foot high embankment reached about 50% of the primary consolidation settlement, while the 18-foot high embankment reached about 90% of the ultimate computed value.

Table 2 - Summary of Selected Settlement Results

Approximate Station of Proposed Road	Embankment Height (feet)	Primary Consolidation Settlement at Embankment Centerline (feet)	Percent of Total Settlement in Soil Types 2 and 4B
713+55	28	6.65	91
716+00	18	2.15	91

7.1.5 It's evident from Plate 38 and Table 2 that the majority of settlement, about 90%, takes place in the soft, shallow deposits, soil types 2 and 4B. Settlement of the road embankment presents a number of problems. Because the road embankment height varies and the thickness of the compressible soil deposits varies, the settlement along the road alignment will not be uniform. This will lead to differential displacements causing cracking in the pavement and premature pavement failure. Settlement at the bridge abutments due to the large earth fills will lead to the development of negative skin friction on the pile foundations. This in turn could lead to expensive foundation modifications to mitigate the effects of increased drag loading or downdrag (an unacceptable level of settlement of the pile foundation). The settlement must be dealt with in one of two ways; wait for the settlement to occur or significantly reduce the embankment loads. A combination of both methods might also prove effective.

7.2 Slope Stability

7.2.1 Building a large earth fill embankment over soft, fine-grained soils typically presents two related geotechnical problems. The first, already discussed, is consolidation settlement. The second problem is the stability of the embankment or the ability of the foundation soils to support the proposed embankment. The soft foundations soils (soil types 2 and 4B) have relatively low undrained shear strengths (see Table 3, Plate 23, & Plate 24). The inability of saturated, fine-grained foundation soils to drain quickly in response to an external load leads to an undrained loading response, the classic total stress $\phi = 0$ condition. Normally the undrained shear strength of fine-grained soils is identified through laboratory testing of undisturbed samples or an insitu undrained shear test, completed prior to embankment construction. The assumption then made in a total stress stability analysis is that the fine-grained soils do not have time to consolidate during embankment construction and the stress path to failure would be undrained.

Table 3 - Geotechnical Parameters Summary for the Van White Memorial Boulevard Project

												S (CD)		Q (UU)	
Soil Type	Soil Description	USCS Classification	LL	MC	PL	PI	LI	Void Ratio	γ_d (pcf)	γ_m (pcf)	$\gamma_{sat.}$ (pcf)	c' (psf)	ϕ' (deg.)	$c_{(3)}$ (psf)	$\Delta c/d_{(4)}$ (psf/ft.)
Fill	COMPACTED FILL(1)	---	---	---	---	---	---	---	---	122	---	0	30	---	---
Unit 1	FILL – Variable composition; gravel, sand, rubble, and clay	SP, CL, GP-GC	---	---	---	---	---	---	---	115	120	0	30	---	---
Unit 2	RECENT BASSETT CREEK SEDIMENTS Silt, clayey; organic rich; soft; abundant shells; green-white-gray	CH/MH/OH	152	116	66	86	0.66	3.06	40	85	87	0	33	290 @6'	12
Unit 4B	LOWER TERRACE CLAY Soft, fat, clay; occasional organics, gray	CH	73	63	24	49	0.81	1.71	62	101	101	100	25	350 @16'	12
Unit 5	COARSE-GRAINED LOWER TERRACE DEPOSITS Sand, clayey, silty; loose-med. dense; gray	SP, SP-SC	---	---	---	---	---	0.55	104	120	123	0	33	---	---
Unit 6A	GLACIOAQUEOUS CLAY - Clay, silty; med. stiff; laminated; gray	CH	63	35	25	38	0.25	0.95	87	116	117	200	32	700 @40'	45
Unit 6B	GLACIOAQUEOUS CLAY - Clay, silty; med. stiff; laminated -massive; rare f. gravel; gray	CL, SP-SC	47	27	18	29	0.34	0.74	98	124	125	200	24	2500	0
Unit 7B	GLACIAL TILL Clay, sandy, gravelly; med. stiff-v. stiff; red-gray	CL	26	16	14	15	0.47	0.41	118	137	137	0(2)	35(2)	1500 @70'	30
Unit 8	GLACIOAQUEOUS SAND & GRAVEL Sand, gravelly; dense; brown	SP-SM	---	13	---	---	---	0.42	118	133	136	0	37	---	---

Notes:
(1) Shear strength parameters and unit weights are assumed.
(2) These parameters are based on triaxial compression and direct shear tests with the failure criterion defined at ultimate deviator stress. This equates to the smaller of the minimum deviator stress (or shear stress) after peak or the deviator stress at 15% axial strain (or shear stress at 0.4-inch shear displacement).
(3) Undrained shear strength at the depth below grade indicated. The depth shown represents the top of the soil unit encountered along the road alignment.
(4) Increase in undrained shear strength per foot of depth.
(5) Values shown in blue were taken directly from U.S. Army Corps of Engineers, *Design Memorandum No. 4 Bassett Creek Flood Control*, 1988.

7.2.2 The computer program UTEXAS4 was used to analyze the stability of the proposed embankment slopes. Plate 40 shows the 28-foot high embankment cross-section located at the proposed north abutment of the south bridge (maximum embankment height) including the soil stratigraphy, shear strength parameters, unit weights of soil and the critical shear surface with computed safety factor. The computed factor of safety at this location for an undrained or end of construction design condition was 1.15, below the normally accepted standard of 1.3. When the same problem was rerun with a 20-foot high embankment, the computed factor of safety was near 1.4 (see Plate 42). These results indicate that when the embankment height exceeds about 23-feet the computed factor of safety would fall below a normal design standard. Table 4 summarizes the results of the slope stability analyses for the two cross-sections considered.

7.2.3 To limit undrained shear distortion the embankment construction should be staged or embankment loads reduced when the embankment height exceeds about 23-feet. Under the current road profile this would consist of a relatively short length of road embankment, about the first 150-feet of embankment north of the north abutment of the south bridge. The embankment in this reach can be built to a safe height or at a rate that would allow the soft foundation soils to consolidate, gain shear strength, and produce a stable embankment configuration. The long-term stability of the 1V on 4H side sloped, 28-foot high embankment was also checked (see Plate 43). The computed factor of safety, 2.45, exceeded the minimum recommended design factor of safety, 1.5, by a healthy margin.

Table 4 - Summary of Slope Stability Results

Approximate Station of Proposed Road	Embankment Height (feet)	End of Construction		Long-term	
		Computed Factor of Safety	Minimum Recommended Factor of Safety	Computed Factor of Safety	Minimum Recommended Factor of Safety
713+55	28	1.15	1.30	2.45	1.50
713+55	20	1.39	1.30	Not Analyzed	1.50

7.3 Pile Foundations

7.3.1 The abutment fills placed at the bridge abutments cause consolidation settlement of the foundation clays and produce a drag load on the abutment piles. The drag load results from negative skin friction that occurs when the soil around the pile moves downward relative to the pile itself. The consequences of negative skin friction can be excessive settlement (beyond tolerable limits) of the pile foundation, termed downdrag, or producing a drag load that exceeds the structural capacity of the pile. If the bridge abutments are adequately preloaded to remove the settlement that would have occurred under the finished road embankment without a preload or if the embankment loads are dramatically reduced, the potential problems associated with the development of negative skin friction can be effectively removed.

8 DESIGN ALTERNATIVES

8.1 Preloading

8.1.1 Preloading involves covering an area with a fill or weight to consolidate the fine-grained foundation soils sufficiently within the available time frame (Terzaghi, 1996). In the case of Van White Memorial Boulevard, the road embankment fill would become part of the preload. Additional fill would be added to surcharge the foundation soils in order to minimize postconstruction secondary consolidation. In effect the surcharge load would overconsolidate the compressible foundation soils. The main problem with this approach is the time required for consolidation to occur. For a typical embankment cross-section the estimated time for 90% of the primary consolidation settlement to occur is about 10-years (Plate 37). To occupy a large area of urban real estate with a surcharge fill and wait for a period of years until pavement and bridge construction can begin is unrealistic. A second problem with this approach is the surcharge embankment would require staged construction, stability berms, or perhaps flatter embankment side slopes to preclude an undrained slope failure when the embankment height exceeds about 23-feet. Staged construction adds extra time to wait for the foundation soils to gain strength, while berms or flatter slopes would require additional right of way. The surcharge embankment itself would also likely require additional temporary right of way.

8.2 Preloading with Vertical Strip Drains

8.2.1 If time were not a design constraint, then preloading along with staged embankment construction where necessary for stability, would be a pretty obvious choice. Time is a very important design constraint and the speed at which construction can proceed often dictates the choice of solution. When the time to primary consolidation is too slow, installing vertical drains can speed up the process dramatically. Vertical drains may take the form of sand drains, pack drains (geotextile-encased sand drains), or prefabricated vertical strip drains. Vertical drains introduce a horizontal drainage path, are typically spaced 3 to 9-feet apart, and may penetrate or partially penetrate the consolidating layer(s).

8.2.2 Prefabricated vertical drains (pvd) today typically consist of a plastic core with grooves or studs wrapped by a nonwoven geotextile. The plastic core is designed to resist installation stresses and lateral earth pressures, while providing an internal flow path along the drain length. The geotextile acts as a filter between the fine-grained soil and the inner plastic core. The installation process normally involves pushing a pvd inside a mandrel to the desired depth, where an anchor plate then holds the pvd in place as the mandrel is withdrawn. This installation method provides equipment flexibility, is very efficient, and quite simple.

8.2.3 A properly designed surcharge with vertical drains should accomplish the following:

- Complete primary consolidation within an acceptable time frame (depends on the project owner's requirements)
- Design the surcharge to limit secondary settlement after surcharge removal to an acceptable level
- Surcharge should be staged or constructed in a stable configuration (slope failure would likely render the installed drains ineffective)

8.2.4 In order to evaluate prefabricated vertical drain (pvd) spacing versus time, the equation developed by Hansbo (1979) was solved for varying drain spacing and average degrees of consolidation. The following assumptions were made for a preliminary pvd design:

- $c_h = c_v$, where c_v was obtained from oedometer tests at a load increment of 2 tons/ft.^{2}, c_v = 10 ft.^{2}/yr.
- Effects of soil disturbance and drain resistance were not included in the analysis
- For practical purposes, t_{90} equates to completion of primary consolidation.
- Pvd length would vary from 30-50-feet along the project profile and penetrate soil types 2 and 4B.

Plate 44 through Plate 46 shows the design computations including graphical results, while Table 5 summarizes the key results. Due to the lack of quality consolidation testing for soil types 2 and 4B, a single value of c_v was used.

Table 5 Summary of Prefabricated Vertical Drain Spacing vs. Time

Approximate Time to Complete Primary Consolidation, t_{90} (years)	Approximate PVD Spacing, Triangular Pattern (feet)	Typical PVD Length (feet)
2	5	30-50
1	3.8	
0.5	3	

8.2.5 Drain length is essentially dictated by the fact that roughly 90% of the primary consolidation takes place in soil types 2 and 4B. Vertical drains penetrating soil types 2 and 4B would allow excess pore water in these soils to flow horizontally to the drains, then to the surface or to higher permeability soils above and below the fine-grained compressible soils. The vertical drains would be used to remove the primary consolidation from soil types 2 and 4B; a surcharge could be used to minimize secondary compression in these soils and to a lesser degree reduce primary consolidation in the deeper, less compressible, fine-grained soils. Plate 47 through Plate 50 contain surcharge design computations for the "typical" embankment section (Station 716+00). Assuming a 5-foot surcharge, with 5-foot pvd spacing, and the surcharge in place until primary consolidation is completed after 2-years, the post surcharge settlement was about one-inch (see Table 6).

8.2.6 For the maximum height embankment at the north abutment of the south bridge, Plate 51 and Plate 52 show the surcharge design computations. As indicated in Table 6 about 3-inches of settlement was computed after the 10-foot surcharge was removed. It's interesting to note, essentially all of the settlement in this case is the result of primary consolidation of soil type 7B, the clay till. If the till was slightly more overconsolidated, say the OCR =1.4 instead of 1.1 value used in design computations, the primary consolidation settlement would drop below an inch. It should be noted in the case of the maximum embankment height, the surcharge load (38-foot high embankment) would require flatter side slopes, stability berms, or staged construction to avoid an undrained slope failure.

8.2.7 A second option exists that would avoid special consideration of slope stability when the embankment height exceeds 23-feet. The surcharge embankment could be constructed to 23-feet at the north abutment of the south bridge, then extend to the north until the surcharge and finished embankment height would begin decreasing. At the north abutment of the south bridge, an adequate surcharge amount could be removed and then a lightweight fill, such as EPS-geofoam, could be used to bring the embankment to design grade. The EPS-geofoam would then step down as the finished embankment height decreased to the north. The surcharge amount and EPS-geofoam could be designed to limit total settlement and differential settlement to an established level.

Table 6 Summary of Post Surcharge Settlements

Approximate Station of Proposed Road	Finished/ Surcharge Embankment Height (feet)	Post Surcharge Primary Consolidation Settlement at Embankment Centerline (inches)	Secondary Consolidation Settlement at Embankment Centerline (inches)	Total Post Surcharge Settlement at Embankment Centerline (inches)
713+55	28/38	3.04	0.06	3.1
716+00	18/23	0.75	0.25	1.0

8.2.8 As the design computations indicate, a surcharge load used in conjunction with prefabricated vertical drains can effectively remove the primary consolidation settlement component from the shallow compressible soil deposits. In addition, the surcharge can be used to reduce secondary compression settlement in the same soils to a tolerable level. Consolidation settlement in the deep, fine-grained soils turns out to be a small component of the overall total settlement. A properly designed surcharge load with pvd can effectively reduce the total and differential settlement of the road embankment to tolerable levels. As an option, EPS-geofoam could be incorporated through the highest embankment reaches (Station 713+55 –716+00) to avoid slope stability problems and achieve very small post surcharge settlements.

8.3 Lightweight Fill – EPS Block Geofoam

8.3.1 Expanded polystyrene (EPS) block geofoam is a cellular-geosynthetic used in lightweight fill applications. The concept of lightweight fill over soft ground is simple; reduce the external load until computed settlement reaches a tolerable level. The additional benefits of EPS block geofoam include rapid construction, avoiding an undrained embankment failure, and significantly reduced lateral loads on bridge abutments. The first design decision would involve embankment cross-section: sloped sides or vertical sides (geofoam wall). Vertical sides produce a cross-section with a smaller footprint (smaller fill volume), however; the exposed vertical sides requires a structural facing material and a Portland cement concrete slab on top of the EPS blocks to anchor road hardware (guardrails, lighting, signage, etc.). This would offset some of the EPS savings and could reduce aesthetics and recreation opportunities.

8.3.2 As a starting point, the two design cross-sections analyzed throughout this report were again used with EPS block geofoam replacing the embankment core. Plate 53 shows a typical EPS block geofoam embankment cross-section and design assumptions. This cross-section assumes EPS geofoam placed directly on the soil subgrade up to the pavement layer with soil cover on the sides to establish grass cover. Plate 54 and Plate 55 contain the results of the consolidation settlement analyses for the 28-foot and 18-foot embankment heights, while Table 7 summarizes the key results. EPS geofoam weighs 1-2 pcf so the external load of the embankment is greatly reduced. For practical purposes the load on the foundation soils results from the pavement atop the EPS and the soil cover on the side slopes. As indicated by the results, primary consolidation settlement is reduced by 87% for a 28-foot embankment and 79% for an 18-foot embankment compared to a conventional earthfill embankment. The problem is the total and differential settlement values are still well above tolerable levels.

Table 7 - Settlement of EPS Block Geofoam Fill Embankments

Approximate Station of Proposed Road	Embankment Height (feet)	Primary Consolidation Settlement at Embankment Centerline (feet)	Percent of Total Settlement in Soil Types 2 and 4B
713+55	28	0.85	95
716+00	18	0.45	95

8.3.3 In order to reduce these settlement values, a portion of the subgrade could be excavated and replaced with geofoam. Essentially if 4-feet of subgrade weighing 115 pcf is removed, this would offset the pavement, EPS, and soil cover weight. If construction proceeded rapidly, there would be little opportunity for the subgrade to rebound and the net effect would be little to no load on the foundation soils resulting in very small total and differential settlement values. The disadvantages of this type of solution would include: additional EPS cost, potential remediation cost of contaminated soils requiring excavation, and evaluation of potential chemical degradation of EPS due to contaminated soil and groundwater.

8.4 Other Alternatives

8.4.1 Another alternative to mitigate the geotechnical problems of settlement and slope stability is a bridge over the railroad tracks, city impound lot, and Bassett Creek Diversion Channel instead of an embankment. The obvious disadvantage to this solution is the cost. The advantages include immediate construction (no waiting for settlement to occur), avoiding the geotechnical and environmental problems by using a pile foundation, and leaving much of the city impound lot intact. A second choice that would be less costly than a bridge is a column supported embankment (CSE). CSE consist of an embankment on top of a load transfer platform that in turn rests on top of closely spaced vertical columns or piles that ultimately transfer the loads to competent foundation material. The key advantage of the CSE solution is the immediate construction of the road when compared to preload alternatives; the primary disadvantage is usually initial construction cost.

8.4.2 Other methods of ground improvement considered more exotic were not considered or discussed. Table 8 below summarizes the advantages and disadvantages of alternatives considered to mitigate the problems associated with weak, compressible foundation soils along the proposed road alignment.

Table 8 - Alternatives Considered to Mitigate Geotechncial Problems

Alternative	Advantages	Disadvantages
Bridge	- Immediate Construction - Pile Foundation avoids Settlement, Stability, and Potential Environmental Problems	- Cost - Potential Difficulty Incorporating Recreational Features
Column-Supported Embankment	- Immediate Construction - Pile Foundation avoids Settlement, Stability, and Potential Environmental Problems	- Cost (Between Bridge and Preload with Vertical Drains)
Preload with Vertical Drains	- Preload Embankment Incorporated into Final Road Embankment	- Time Delay, Typically 6 months – 2 years - PVD Draining Potentially Contaminated Groundwater
Lightweight Fill – EPS Block Geofoam	- Immediate Construction - Variable Embankment Footprint (Vertical Sides Possible) - Minimal Lateral Load at Bridge Abutments	- Cost (Typically Slightly more than Preload with Vertical Drains) - Possible Excavation of Contaminated Soils - Possible Chemical Degradation of EPS - Difficult to Include Utilities - Performance History Somewhat Limited (20-25 years in U.S.)
Preload	- Lowest Cost - Preload Embankment Incorporated into Final Road Embankment	- Large Time Delay (years) - Preload Embankment Causes Slope Stability Problems (Staged Preload or Additional Real Estate Required for Stability Berms)

9 BRIDGE FOUNDATIONS

9.1 General

9.1.1 A brief survey of MN/DOT bridges in the project vicinity shows bridges supported on 12-inch diameter cast-in-place, cased, concrete piles. Steel casing with a conical shaped driving shoe is driven to the design tip elevation and then backfilled with concrete. Table 9 summarizes the pile type, structural loads, and bearing stratum for selected piers from three bridges. Table 9 indicates the piles from the three bridges in the area are founded in soil type 7B, sandy, gravelly, clay till.

Table 9 - Existing Bridge Foundations in the Proposed Project Vicinity

Bridge	Pile Type	Estimated Pile Length/Tip Elevation (feet)	Approximate Maximum Pile Loads (tons/pile)	Soil Type Piles Founded in Based on [MN/DOT Boring No.]
Pedestrian Walkway (Between Linden and Laurel Ave.) No. 27866	12" ϕ CIP Concrete	80/740	39 (Pier No. 3)	Unit 7B Till [T-1]
Trunk Highway No. 12 No. 27831	12-3/4" ϕ CIP Concrete	125/690	63.5 (Pier No. 35)	Unit 7B Till [52741]
Trunk Highway No. 394 No. 27776	12" ϕ CIP Concrete	70 (test piles)/ 745 (test piles)	Not Available (Pier No. 16)	Unit 7B Till [52739]

9.2 Design Pile Capacities

9.2.1 The ultimate axial bearing capacity of a single driven pile was determined using the program APILE Plus, Version 3.0 developed by ENSOFT, Inc. The ultimate pile capacity was computed by the program using four different methods: API (American Petroleum Institute), U.S. Army Corps of Engineers, Federal Highway Administration, and the Revised Lambda Method. Refer to the ENSOFT documentation, appropriate government guidance, or engineering text for a detailed discussion of each analysis method. The ultimate axial pile capacity was computed using the unconsolidated-undrained (UU) shear strength for the fine-grained soil layers, and consolidated-drained (CD) shear strength for the sands. Each of the four methods of analysis is theoretical in nature and can yield very different ultimate capacities for a given set of soil conditions. As a result, the ultimate capacities from the four methods were averaged and the average value was reported as the ultimate pile capacity. The theoretical ultimate pile capacities are then reduced by a safety factor of 3.0 to determine the allowable pile bearing capacities.

9.2.2 For design purposes the following assumptions were made: a nominal outside pile diameter of 12-inches, pile wall thickness of 0.25-inches, and the skin friction in the upper 5-feet of the soil profile was ignored. The pile capacities at two separate locations were analyzed, the north and south abutments of the proposed south bridge location located roughly 550-feet apart. Since the south abutment location shows a lesser thickness of weak, soft foundation soils and a higher till elevation relative to the north abutment location, larger computed pile capacities would be expected at the south abutment location. The north and south abutments of the south bridge location should effectively bracket anticipated subsurface conditions and pile capacities.

9.2.3 Plate 56 contains the geologic stratigraphy at the south abutment, south bridge used in computing pile capacities. Plate 57 shows a plot from APILE Plus, of the computed pile capacity versus depth and Plate 58 shows results from the four methods of analysis at selected depths. Plate 59 through Plate 61 contain the same respective results of the north abutment, south bridge. Table 10 and Table 11 summarize the allowable pile capacities for pile lengths of 90, 110, 130, and 150-feet for the south and north abutment respectively. The computed allowable bearing capacity generally varies from about 35-tons for a 90-foot long pile to 100-tons for a 150-foot long pile. Based on boring 03-5M piles less than 150-feet in length will be founded in the clay till (unit 7B) and will develop about 95% of their capacity from skin friction (friction piles). Beneath the clay till a dense sand layer exists that would provide 5-6 times greater tip resistance than the till.

9.2.4 Based on AASHTO bridge design guidance, reductions in the axial load carrying capacity of a pile group (group efficiency) result when using friction piles in cohesive soils and/or when pile spacing < 3 pile diameters. This should be accounted for in the final pile group layout of the bridge piers and abutments.

Table 10 – South Bridge, South Abutment, Allowable Pile Bearing Capacity (Single Pile)

Pile Tip Elevation	Pile Length (feet)	Allowable Pile Bearing Capacity (tons)	Percent of Total Capacity from Skin Friction	Percent of Total Capacity from Tip Resistance
728	90	38	94	6
708	110	55	94.5	5.5
688	130	75	95	5
668	150	111	82	18

Table 11 – South Bridge, North Abutment, Allowable Pile Bearing Capacity (Single Pile)

Pile Tip Elevation	Pile Length (feet)	Allowable Pile Bearing Capacity (tons)	Percent of Total Capacity from Skin Friction	Percent of Total Capacity from Tip Resistance
722	90	34	93	7
702	110	50	94	6
682	130	70	95	5
662	150	106	81	19

9.3 Drag Load

9.3.1 The abutment fills placed both abutments of the south bridge cause consolidation settlement of the foundation clays and produce a drag load on the abutment piles. The drag load results from negative skin friction that occurs when the soil around the pile moves downward relative to the pile itself. The consequences of negative skin friction can be excessive settlement (beyond tolerable limits) of the pile foundation, termed downdrag, or producing a drag load that exceeds the structural capacity of the pile. The pile settlement (settlement of the pile head neglecting pile compression from the dead and drag loads) is equal to or less than the settlement value at the neutral axis. The neutral axis is defined as the pile depth where the relative displacement between the pile and the adjacent soil is zero.

9.3.2 Since the abutment piles will likely be founded relatively deep into the glacial till, the neutral axis of the pile would be expected to be in the till as well. At the north abutment where the embankment height is the greatest, the computed settlement from a 28-foot embankment is on the order of 3-4 inches in the glacial till. Even adjacent to the highest embankment fill, excessive settlement of the piles should not be a concern. In the case of long piles (> 100 pile diameters), the additional stress from the drag load could be a problem. The drag load plus the dead load on the pile should not exceed the pile's structural compressive capacity. This load combination should be checked in addition to the dead load plus live load combination.

9.4 Pile Load Testing

9.4.1 For the bridge piling, a dynamic pile load testing program is recommended, preferably in conjunction with static load testing. Using pile driving formulas tends to result in added conservatism and extra costs. In addition to determining pile load capacity, dynamic pile testing provides information about the driving system efficiency, driving stresses that develop in the pile, and the pile integrity. Depending on the logistics of the pile driving operations, high-strain dynamic testing should be conducted upon restrike after sufficient time has elapsed to allow for setup in the fine-grained soils and relaxation in the coarse grained soils. Then a wave equation analysis can be performed on the initial and restrike dynamic measurements to attempt to quantify the potential presence of increased skin friction due to soil setup. Finally, prior to driving piling the construction contractor should submit a wave equation analysis of the proposed driving system to show the hammer-pile system can efficiently and effectively drive the piling to the required bearing capacity.

10 BASSETT CREEK FLOOD CONTROL PROJECT

10.1 General

10.1.1 The Van White Memorial Boulevard proposed alignment will cross the current Bassett Creek Diversion Channel at about Station 720+00 and cross the old Bassett Creek Diversion Channel (now an overflow channel) at about Station 722+00. The preliminary Van White Memorial Boulevard plan (dated October 9,

2002) indicated a bridge over the current diversion channel and a road embankment over/filling the old diversion channel. Both channels must remain open and provide the same conveyance as pre-road project. This can be accomplished by a bridge over both channels or perhaps by cast-in-place box culverts.

11 CONSTRUCTION CONSIDERATIONS

11.1 General

11.1.1 Clearing and grubbing operations should be minimal with only a few trees and minor vegetation present near the creek diversions. No topsoil was present in the geotechnical borings completed along the proposed road alignment. Due to the absence of topsoil, little to no stripping is anticipated. An offsite source of topsoil may need to be located for finished cover on the embankment side slopes. If a traditional earthfill embankment will be constructed, a borrow source will have to be located. The availability of a borrow source close to the project site could dictate the selected method of ground improvement.

12 REMAINING WORK FOR FINAL DESIGN

12.1 Subsurface Exploration

12.1.1 Standard DOT practice is to complete a boring at each bridge pier. Due to the difficulty in obtaining rights of entry to the railroad property and since the bridge layout was not complete, borings at each bridge pier were not completed. This work should be accomplished as part of the final design. Also depending on the choice of bridge or culverts at the Bassett Creek Diversion Channels, additional bridge pier borings for a potential north bridge location should be completed. Finally, if prefabricated vertical drains are chosen as the primary method of ground improvement a better estimate of the horizontal coefficient of consolidation should be obtained. This could be done by obtaining quality undisturbed tests and conducting vertical and horizontal permeability testing or by conducting pore pressure dissipation tests with an electrical piezocone (CPTU).

12.1.2 The piezo cone penetrometer is a good exploration tool for some key soft ground engineering design parameters. In addition to determining soil stratigraphy, the CPTU provides good correlations to overconsolidation ratio and undrained shear strength. Consideration should be given to using the CPTU as supplemental exploration tool during final design of the Van White Memorial Boulevard Project.

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