I-395 Project

Exhibit C-3

Technical Requirements

Attachment 3.4a

395 Express Lanes Geotechnical Data Report

GEOTECHNICAL ENGINEERING DATA REPORT FOR 395 EXPRESS LANES PRELIMINARY DESIGN

RFP Conceptual Plans

Arlington County, Fairfax County,
& City of Alexandria, Virginia

Prepared for:

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Prepared by:

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REV. 1 – September 22, 2016

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APPENDICES

1.0 INTRODUCTION

The purpose of this document is to present the results of the preliminary geotechnical investigation and to supplement the geotechnical investigation and design requirements contained in Chapter III of the VDOT Materials Division's Manual of Instructions for the 395 Express Lanes project. This document also contains supplemental requirements relating to the project specific pavement design requirements. In cases of conflict, the requirements of this document supersede the requirements of the VDOT Materials Division's Manual of Instructions.

2.0 PROJECT AND SITE DESCRIPTION

The project extends from the current northern terminus of the 95 Express Lanes at Turkeycock Run near Duke Street in the south, to the vicinity of Boundary Channel Drive near the Pentagon in the north, and is located within Arlington County, Fairfax County, and the City of Alexandria. Referenced to the preliminary design baseline, the project starts in the south at Station 1533+50 and may extend (depending on options constructed) as far north as Station 1925+00, for a total length of approximately 7.4 miles. At approximate Station 1876+00, the reversible HOV lane facility splits into separate northbound (NB) and southbound (SB) lanes divided by a barrier. The separated lanes extend to the northern terminus of the project. South of the split, the facility consists of two travel lanes and two shoulders. North of the split, the facility consists of two travel lanes and one outside shoulder in each direction. Figure A-1 in Appendix A shows the general site vicinity of the project in addition to approximate stationing referenced at several locations along the alignment.

In general, the two travel lanes and two shoulders that make up the current High Occupancy Vehicle (HOV) lanes along the project alignment will be converted to three travel lanes and two narrower shoulders. The two existing travel lanes have an average total width of approximately 23.5 feet. The shoulders, which are located on either side of the travel lanes over most of the alignment, generally range from approximately 8 feet to 12 feet in width and in limited cases are narrower or wider. The existing shoulder sections will be removed and replaced full depth, while the existing main travel lanes will be milled and overlaid to support the proposed improvements to the HOV facility. The new travel lanes will be 11 feet wide, and the new shoulders will generally be two and ten feet wide. Joints in the existing concrete pavement beneath the main travel lanes will also require repairs during construction.

3.0 OBJECTIVE AND SCOPE

The objective of this preliminary geotechnical investigation is to collect, document, and report geotechnical data sufficient to support the preparation of 30% plans, Technical Requirements, and design-build bidding documents for the 395 Express Lanes project. To accomplish this objective, HDR completed the following general scope of services:

- Drilled 168 pavement cores and shallow soil test borings to identify existing pavement thickness and soil subgrade conditions,
- Drilled two soil test borings to support preliminary design of a retaining wall along the entrance ramp from South Eads Street,
- Conducted laboratory testing of selected soils from the test borings to aid in soil classification and establishing subgrade support parameters for preliminary design,
- Conducted a condition assessment of the existing pavements and joints to characterize the number and type of joint repairs that will be required during the construction phase,
- Conducted a Ground Penetrating Radar (GPR) survey to aid in determining average asphalt concrete pavement section thickness at regular intervals along the project alignment,
- Conducted Falling Weight Deflectometer (FWD) testing to evaluate existing pavement section and subgrade soil stiffness properties for preliminary design,
- Prepared a Pavement Condition Assessment Report (provided under separate cover) to summarize HDR's assessment of joint repair locations and conditions, surface pavement condition, and existing pavement section thickness,
- Prepared this Geotechnical Engineering Data Report to summarize the subsurface exploration and present the results of the field and lab testing programs, as well as considerations for design and construction.

4.0 GEOTECHNICAL EXPLORATIONS

4.1 Preliminary Geotechnical Investigation

HDR planned and executed a subsurface exploration program to collect geotechnical data along the alignment. Traditional soil test borings were used to complete pavement cores and obtain physical samples for visual/manual classification, laboratory testing, and observation of subsurface water levels. Non-destructive testing (NDT), including a GPR survey and FWD testing were conducted to aid in evaluation of the existing pavement section and underlying subgrade. The following sections provide details relative to the preliminary subsurface exploration program.

4.1.1 Soil Test Borings

DMY Engineering Consultants, Inc. (DMY) of Dulles, Virginia and GET Solutions, Inc. (GETS) of Virginia Beach, Virginia drilled 170 subsurface explorations along the proposed alignment from January 19 to March 7, 2016. The subsurface explorations include 160 Standard Penetration

Test (SPT) borings and 10 pavement cores. Table 1 provides a brief summary of the subsurface explorations drilled.

Table 1 – Summary of Subsurface Explorations

Exploration Purpose	Exploration Depth (feet)	Number of Exploration Drilled	Total Drilled (feet)
Roadway SPT Boring	3 to 14	158	1,482
Retaining Wall SPT Boring	10 to 29	2	80 [1]
Pavement Core only	N/A	10	0
	Total	170	1,562
[1] Total includes depth drilled for offset borings.			

Roadway test borings were generally advanced to a depth of approximately 8 feet below the pavement subbase using hollow stem drilling techniques. Standard Penetration Tests (SPT) with split-barrel spoon sampling of soils were conducted in accordance with ASTM D1556 using an automatic hammer. Typically, four 24-inch long SPT samples were collected in the upper 8 feet below the pavement subbase of each borehole.

Retaining wall test borings were advanced to depths ranging from 10 to 29 feet below the ground surface using the drilling techniques discussed above. Typically, five 24-inch long SPT samples were collected in the upper 10 feet of each borehole and at five-foot intervals below a depth of ten feet.

DMY, GETS, and HDR personnel monitored the drilling in the field, which included overall coordination of drilling activities, visual-manual classification of soil samples, preparation of field exploration logs, monitoring conformance with drilling and sampling criteria, and observation of general site conditions. Groundwater observations were made in open boreholes upon completion of drilling and prior to backfilling. In general, because most borings were located within the HOV travel lanes and shoulders, 24-hour water level measurements were not made in order to backfill the holes immediately after drilling.

Pocket penetrometer tests were typically performed on cohesive soil samples at the time of sample collection. The results of the pocket penetrometer tests are shown on the exploration logs provided in Appendix B.

DMY and GETS personnel collected bulk soil samples beneath the pavement subbase from 28 of the soil test boring locations. The samples were collected from auger cuttings to depths of up to approximately six feet below the pavement subbase. The samples were collected for laboratory testing, including classification, moisture-density relationship, California Bearing Ratio (CBR), and resilient modulus. Bulk sample locations are indicated on the applicable test boring logs in Appendix B and laboratory test results are provided in Appendix C.

Borings advanced through existing pavements were typically backfilled with pea gravel to a depth of approximately two feet below the pavement surface. The remaining two feet were backfilled with grout. Borings not advanced through the existing pavement were backfilled with auger cuttings.

HDR personnel laid out proposed exploration locations in the field along a field location baseline surveyed by Rice Associates. Locations were marked in roadway shoulders and moved to proposed locations during drilling. SPT borings and pavement cores were surveyed by Rice Associates for location and surface elevation after drilling was completed.

Station/Offset and Latitude/Longitude are provided on the exploration logs. Survey data are referenced to North American Datum 1983 (NAD-83) State Plane Virginia (feet). Vertical datum is referenced to North American Vertical Datum 1988 (NAVD-88), also in feet. A summary of the soil test borings is provided in Table A-1 in Appendix A. Test boring logs are provided in Appendix B. As-drilled boring locations are shown on Figure A-3 in Appendix A.

4.1.2 Pavement Cores

DMY and GET drilled 170 subsurface explorations along the proposed I-395 alignment. One hundred sixty-eight of these explorations were drilled through existing pavement sections between January 19 and February 22, 2016. DMY and GETS cored the existing pavement sections at a total of 163 of these locations in the main travel lanes and shoulders. Pavement cores were not collected in 16-PC-019-SH, 16-PC-022-SH, 16-PC-025-SH, 16-PC-028-SH, and 16-PC-031-SH; however, pavement thicknesses were measured after augers were advanced through the pavement. HDR personnel measured and photographed the pavement cores. One hundred fifty-three of the pavement cores were collected at roadway SPT boring locations. The remaining 10 pavement cores were collected at tie-in locations or locations where roadway SPT boring locations were not required or could not be drilled due to utility conflicts.

The pavement section thickness data is summarized in Table A-2, and photographs (with observational annotation) of the pavement cores are provided in Appendix D. Pavement core locations are depicted on Figure A-3.

4.1.3 Laboratory Testing

DMY, GETS, and Boudreau Engineering conducted laboratory testing on soil samples (jar and bulk samples) collected from the subsurface explorations. HDR personnel evaluated the field exploration logs and assigned specific samples for testing. Testing was performed to aid in the classification of soils encountered in the explorations and to support development of geotechnical engineering parameters for preliminary design. Table 2 summarizes the completed laboratory testing.

Table 2 – Summary of Laboratory Testing

Laboratory Test	Specification Referenced		Laboratory	No. of Tests
	VTM	AASHTO		
Moisture Content		T 265	DMY and GETS	604
No. 200 Wash	25	T 11		124
Grain Size Analysis	25	T 88		124
Atterberg Limits	7	T 89 / 90		88
Standard Proctor	1	T 99		28
California Bearing Ratio	8	T 193		28
Resilient Modulus		T 307	Rick Boudreau	26

A laboratory index table, indicating which testing was performed from each test boring, is provided in Table A-3 in Appendix A. Index testing (natural moisture content, Atterberg Limits, and fines content) results are shown on the test boring logs in Appendix B. A summary of laboratory test results (except for moisture content only samples) is provided in Table C-1 in Appendix C. The laboratory test results are provided in Appendix C.

4.1.4 Ground Penetrating Radar Survey (GPR)

Infrasense, Inc. of Woburn, Massachusetts completed a Ground Penetrating Radar (GPR) survey of the project alignment on February 6, 2016. The study was conducted between approximate Station 1510+30 on the south end to 1941+00 on the north end. (Note: a portion of the dedicated southbound lane from approximate Station 1928 to 1941 (northern project extent) was not tested due to MOT restrictions) The objective of the GPR survey was to aid in determining the asphalt concrete pavement thickness at regular intervals along the project's length.

The GPR data was collected with a GSSI 1-GHz horn antenna GPR system, center-mounted behind the survey vehicle providing continuous data along the centerline of each lane. The vehicle was equipped with an electronic distance-measuring instrument (DMI) mounted to the rear wheel, providing synchronous distance data as the GPR data was collected. A Trimble GPS unit provided high resolution, differentially corrected geo-spatial information. The data collection and recording was controlled by the SIR-20 GPR system operated from within the survey vehicle. The data was collected at a rate of 1 scan per foot of travel. The pavement layer thicknesses were calculated at 25-foot increments, and represent the average of individual readings calculated over an interval of ± 12 feet on either side of the reported location.

HDR provided the results of the pavement core measurements to Infrasense to ground truth the GPR data before their report was finalized. Infrasense's report titled "Ground Penetrating Radar (GPR) Pavement Evaluation of the I-395 Corridor HOV Lanes," dated April 8, 2016 is provided as Appendix E.

4.1.5 Falling Weight Deflectometer (FWD) Testing

PTS collected NDT data for the roadway pavements using their in-house Dynatest Model 8000 Falling Weight Deflectometer (FWD) during weekend closure of the facility on February 6, 7, 13, 20, and 21, 2016. The FWD used for the testing operation is capable of simulating a moving wheel load of up to 34,000 pounds.

The FWD data may be used to assess the overall support conditions of the in-place pavements and subgrades by providing data for:

- Development of pavement deflection profile plots that are helpful in delineating design sections for long stretches of roadway, and
- Back-calculation of pavement layer strengths and subgrade support parameters for use in structural analysis of the pavements.

NDT tests were conducted in accordance with ASTM D4694-09, “Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device” and ASTM D4695-03, “Standard Guide for General Pavement Deflection Measurements”. NDT tests were typically spaced 100 feet longitudinally in each lane and staggered by 50 feet for adjacent lanes of testing, which provided a final longitudinal spacing of 50 feet over the alignment. An exception to this was at approximate Station 1876+00, where the reversible HOV lane facility splits into separate northbound and southbound lanes divided by a barrier. Since each side consisted of two travel lanes and one shoulder, the NDT spacing was increased to 200 feet in each lane and staggered at 100 feet. This provided a final longitudinal spacing of approximately 50 feet for the facility from 1875+00 to the north project limit. (Note: a portion of the dedicated southbound lane from approximate Station 1928 to 1941 (northern project extent) was not tested due to MOT restrictions)

PTS’s data report containing the FWD data is provided in Appendix F.

4.2 Design Phase Geotechnical Explorations

Geotechnical explorations were not completed to support preliminary design of bridge structures, minor structures (pipes and culverts greater than 36 inches in diameter), cut or fill slopes, storm water management ponds, sound barrier walls, toll gantries, and sign foundations.

A design phase geotechnical investigation shall be performed by the design-builder in accordance with Chapter III of the VDOT Materials Division’s Manual of Instructions to investigate existing subsurface geotechnical conditions in the areas of the proposed improvements. A boring location plan must be approved by the Northern Virginia District Materials Engineer prior to initiation of the design geotechnical investigation. The final geotechnical engineering report must be reviewed and approved by the Northern Virginia District Materials Engineer prior to initiation of construction activities.

5.0 SUBSURFACE SUMMARY

The following sections summarize the results of the subsurface explorations completed at the project site. Specific observations, remarks, and comments are reflected on the exploration logs provided in Appendix B.

5.1 Site Geology Review

The project site is located near the western edge of the Coastal Plain Physiographic Province. The Coastal Plain Province consists of an eastward-thickening wedge of unconsolidated river/deltaic and marine sediments. The interbedding of fine- and coarse-grained sediments is complex due to irregular deltaic and alluvial deposition, as well as the cyclic marine deposition associated with transgressions and regressions of the sea. Strata unconformities (gaps in the geologic record) due to periods of erosion and regional faulting are common within the western Coastal Plain. As a result, strata composition and thicknesses can vary greatly over short horizontal or vertical distances.

HDR reviewed a number of local geology references for information relative to mapping of geology units along the project alignment. The references included the Fairfax County Soils Map (2015), the City of Alexandria Marine Clay Area Map (1976), the Arlington County Geologic Map (2006), and the Annandale quadrangle geologic map (1986). Refer to Section 9.0 for the specific reference information. Table A-4 in Appendix A summarizes the approximate surface mapping of geologic units along the project alignment using the local geology references summarized above.

Potomac Formation soils, which consist of both clays and sands, are present within the project site. The quartzo-feldspathic sands are light gray to pinkish and greenish gray, fine to course grained, poorly sorted, and commonly thick bedded. The sands are interbedded with gray to green, massive to thick-bedded clay and silt, commonly mottled red or reddish brown. Low to highly plastic silts and clays (ML/MH, CL/CH) of variable thickness underlie surface silts, sands, and gravels. The soils of the Potomac Formation occur on side slopes and hilltops, and within the older or buried floodplains of the Coastal Plain. The thickness ranges from feather-edge at western limit of outcrop to more than 3,500 feet in the subsurface of the outermost, eastern Coastal Plain.

The high-plasticity silt and clay deposits of the Potomac Formation (also referred to as marine clays, Potomac clays, or Marumsco soils on the geologic maps) are highly fractured and broken, and contain fissures and discontinuities. They are considered “unsuitable” and are known locally to be problematic, specifically with regards to slope stability and volumetric changes with moisture variation (shrinking and swelling).

High-plasticity Potomac formation clays can present stability issues over extended periods of time due to the potential for softening and weakening along the existing fissures in the clay, resulting from exposure of the fissures to disturbance and water from construction activities. The clays often exhibit slickensides (previously sheared surfaces characterized by residual shear strengths) along the fissures and discontinuities that may impact their overall stability.

5.2 Subsurface Soil

Subsurface conditions vary along the length of the alignment and consist of sands, gravels, silts, and clays. HDR typically observed two strata throughout the alignment.

HDR observed Stratum I (Fill and Suspected Fill Soils) in a majority of the pavement test borings below the pavement subbase aggregate. Stratum I ranged in thickness from approximately 2 feet to 8 feet within the test boring depths. Stratum I consisted primarily of sands (SP, SP-SM, SP-SC, SC-SM, SM, SC, SW-SC); however, gravels (GP, GP-GC, GW-GC, GC), silts (MH), and clays (CL, CH) were present to a lesser extent. SPT N-values ranged from 1 blow per foot (bpf) to split spoon refusal (50 blows per 6 inches or less).

Stratum I was observed in the two retaining wall borings from the ground surface to the boring termination depths (approximately 14 to 29 feet bgs). Stratum I in the retaining wall borings consisted primarily of sands (SP, SP-SC, SC, and SM); however, clays (CL, CH) were present to a lesser extent. SPT N-values ranged from 2 blows per foot (bpf) to split spoon refusal.

Suspected Fill soils were identified based on observations of soil and site conditions, including:

- The presence of foreign material such as fragments of asphalt, concrete, wood, brick, angular/sub-angular gravel, or other debris within the otherwise Coastal Plain geology
- Apparent layering formed by compaction operations, and
- The presence of fill slopes based on observations of surrounding topography.

Stratum II (Native Coastal Plain Soils - Interbedded Sands and Clays) was observed below Stratum I, where present, and ranged in thickness from approximately 2 feet to 8 feet within the test boring depths. Stratum II soils consisted of interbedded sands (SP, SW, SP-SM, SP-SC, SW-SC, SC, SM) and clays (CL, CH); however, gravels (GP-GC, GM) and silts (ML, MH) were present to a lesser extent. SPT N-values in the coarse-grained soils ranged from 2 bpf to split spoon refusal, indicating relative densities ranging from very loose to very dense. SPT N-values in the fine-grained soils ranged from 5 bpf to 30 bpf, indicating consistencies ranging from firm to very stiff.

Shallow refusal was encountered in 10 of the test borings at depths of 10 feet or less. No rock coring or excavation was conducted to evaluate the cause of the shallow refusal. In some cases, as noted on the boring logs, borings were offset a short distance with similar results. It is possible that concrete rubble or other large debris may have been used as fill material within the existing embankments. Table 3 summarizes the boring locations where shallow refusal was encountered.

Table 3 – Summary of Shallow Refusal Boring Locations

Boring	Station	Surface El. (ft)	Depth to Refusal (ft)	Refusal El. (ft)	Refusal Type
16-PC-099-SH	1761+63	48.6	5.4	43.2	Auger
16-PC-114-SH	1799+04	50.2	5.1	45.1	Auger
16-PC-147-SH	1880+73	73.9	6.1	67.8	Split Spoon
16-PC-148-SH	1883+15	71.3	7.8	63.5	Split Spoon
16-PC-149-ML	1888+11	65.1	6.8	58.3	Split Spoon
16-PC-150-SH	1889+09	62.4	3.4	59.0	Split Spoon
16-PC-155-ML	1903+08	43.9	8.6	35.3	Split Spoon
16-PC-157-SH	1907+76	30.6	4.5	26.1	Auger
16-PC-166-SH	1930+66	24.4	4.3	20.1	Auger
16-BH-RW-002	1895+63	40.4	10.0	30.4	Auger

5.3 Subsurface Water

Subsurface water was observed in 16 of the test boring explorations completed for this study at the time of drilling (in open boreholes prior to backfilling) at depths ranging from approximately 3 to 7 feet below the ground surface. In general, because most borings were located within roadways, 24-hour water level measurements were not made in order to backfill the holes immediately after drilling. Refer to Table 4 and the exploration logs in Appendix B for specific observations of subsurface water at the exploration locations.

Table 4 – Summary of Subsurface Water Observations during Drilling

Test Boring Designation	Test Boring Station	Test Boring Surface El. (ft)	Approximate Subsurface Water Depth (ft)	Approximate Subsurface Water Elevation (ft)
16-PC-008-ML	1535+44	184.3	5.0	179.3
16-PC-011-ML	1542+88	208.7	4.5	204.2
16-PC-019-SH	1562+83	236.2	6.5	229.7
16-PC-022-SH	1570+45	230.7	4.0	226.7
16-PC-033-SH	1596+62	171.3	5.0	166.3
16-PC-081-SH	1716+70	149.9	6.1	143.8
16-PC-096-SH	1754+34	54.5	3.0	51.5
16-PC-098-ML	1760+36	49.6	4.0	45.6
16-PC-145-SH	1878+12	76.0	6.0	70.0
16-PC-146-ML	1880+40	73.8	5.0	68.8
16-PC-147-SH	1880+73	73.9	5.0	68.9
16-PC-149-ML	1888+11	65.1	4.0	61.1
16-PC-155-ML	1903+08	43.9	5.0	38.9
16-PC-159-SH	1911+79	22.4	5.0	17.4
16-PC-162-SH	1920+84	38.8	7.0	31.8
16-PC-164-ML	1925+66	34.6	6.0	28.6

Note that subsurface water levels tend to fluctuate due to precipitation, season, temperature, site grading, and other factors that may be different from those prevailing at the time HDR completed its subsurface explorations.

6.0 PAVEMENTS

6.1 Existing Pavement Condition and Thickness

HDR executed a multi-faceted approach to evaluating the existing pavement thickness and condition on the project, including pavement cores, a visual condition assessment of joints and surface pavements, and non-destructive testing (NDT) to assess section thickness, stiffness, and subgrade support. HDR's Pavement Condition Assessment Report, dated June 3, 2016, describes the approach, results, and conclusions in detail. An inventory of the existing joint pavement conditions is included with the report. Joint repair details are provided in the 30% plans.

Pavement thickness information was collected at 168 locations by coring or augering. The pavement section thickness data is summarized in Table A-2 in Appendix A. Specific observational comments related to the recovered core are contained on the pavement core photographic logs contained in Appendix D.

Table 5 summarizes the approximate existing pavement section thicknesses, broken down by the lane or shoulder in which the pavement thickness was measured. In general, the reduced strength shoulders consist of approximately 4 inches of asphalt concrete (AC) pavement over 7 to 12 inches of subbase aggregate. The main travel lanes consist of approximately 4 inches of asphalt concrete pavement over 9 inches of Portland cement concrete (PCC) pavement over 7 to 8 inches of cement-treated aggregate over 5 inches of subbase aggregate. The range and average values for the layer thicknesses are provided in Table 5.

Table 5 – Summary of Typical Pavement Core Measurements

West Shoulder	West Travel Lane	East Travel Lane	East Shoulder
AC Pavement 2.5 to 8.5 inches *Average = 3.9 in*	AC Pavement 2.8 to 6.0 inches *Average = 4.2 in*	AC Pavement 2.8 to 6.1 inches *Average = 4.0 in*	AC Pavement 2.0 to 7.7 inches *Average = 3.9 in*
Aggregate Subbase 2 to 14 inches *Average = 7.4 in*	PCC Pavement 7.8 to 10.0 inches *Average = 8.7 in*	PCC Pavement 8.0 to 11.0 inches *Average = 8.9 in*	Aggregate Subbase 1 to 27 inches *Average = 11 in*
	Cement Treated Aggregate 3.0 to 9.0 inches *Average = 7.5 in*	Cement Treated Aggregate 4.0 to 9.1 inches *Average = 7.0 in*	
	Aggregate Subbase 1 to 13 inches *Average = 5.6 in*	Aggregate Subbase 1 to 10 inches *Average = 5.5 in*	

In a number of exploration locations and the interpreted GPR data, HDR observed "non-typical" pavement thicknesses that do not represent the generally observed thicknesses summarized in Table 5. Examples include: thicker than usual asphalt concrete pavement, PCC pavement beneath the asphalt concrete surface of the shoulder, or the lack of a cement-treated aggregate layer in the main travel lane. Refer to the pavement core logs in Appendix D and the GPR data in Appendix E for further information.

6.2 Minimum Pavement Section Requirements

Minimum pavement sections provided herein are for proposal preparation purposes only. The design-builder will be required to validate the adequacy of the minimum pavement sections and to notify the Concessionaire of its findings during the Scope Validation Period. If the selected design-builder confirms that the minimum pavement sections are inadequate for actual design/construction conditions, the design-builder shall notify the Concessionaire during the Scope Validation Period of the necessary changes and proposed price adjustments, if any. Acceptable changes are limited to increasing the thickness of the base or subbase layers specified below. Any changes to the pavement sections specified below must be approved by the Concessionaire. The design-builder will be responsible for the final design and construction of the pavements for this project in accordance with the Technical Requirements.

Table 6 summarizes the traffic design parameters used to establish the minimum pavement sections for the 395 Express Lanes project.

Table 6 – Summary of Design Traffic Parameters

Parameter	Value Used	Reference
Design life	30 years	VDOT MOI VI-41 for Interstate Roadway
ADT (2015)	58,000 VPD	Interchange Justification Report (IJR) dated 1-7-09
ADT (2030)	64,000 VPD	Interchange Justification Report (IJR) dated 1-7-09
Design Year ADT (2048)	72,300 VPD	Calculated, assumes open to traffic in 2019
Growth rate	0.67% per year	Calculated between 2015 and 2030 from IJR ADT's
%Passenger Vehicles	97%	Typical for 95 Xpress lanes (south of I-395, same corridor)
%2-Axle, Single Unit	3%	
%3-Axle, Tractor Trailer	0%	
ESAL Factors (Flexible Design)	0.0002 / 0.46 / 1.05	VDOT MOI VI-43
ESAL Factors (Rigid Design)	0.0003 / 0.59 / 1.59	VDOT MOI VI-48,49
Lane Distribution	70%	VDOT MOI VI-42 for 3 lane roadway
Directional Distribution	100%	Appropriate for reversible HOV facility
Design ESALs (Flexible)	7,051,126	Calculated
Design ESALs (Rigid)	9,065,085	Calculated

The minimum pavement designs are based upon the following criteria: (a) a minimum soil subgrade CBR value of 5 (therefore all imported fill material shall have a minimum CBR value of 5), (b) all subgrade is compacted in accordance with the applicable sections of the Road and Bridge specifications and applicable special provisions and, (c) that all unsuitable materials at, or below, subgrade have been removed or modified in accordance with applicable sections of Division I Amendments to the Standard Specification General Provisions for Design-Build Contracts under Part 5 of the RFP document.

As indicated on the preliminary plans, all existing paved shoulders shall be cut with a smooth vertical face to expose the original Portland cement concrete pavement beneath the main travel lanes, demolished and reconstructed with the pavement section identified below. Also as indicated on the preliminary plans, all existing main travel lanes shall be milled to a depth of 2 inches and resurfaced with 2 inches Asphalt Concrete, Type SM-12.5E. All underdrains beneath the existing paved shoulders shall be removed and replaced to the nearest available outlet outside of the striped travel lanes.

- Surface – 2 inches Asphalt Concrete, Type SM-12.5E (estimated at 236 lbs/yd^2)
- Intermediate – 2 inches Asphalt Concrete Type IM-19.0D (estimated at 244 lbs/yd^2)
- Base – 11 inches Asphalt Concrete, Type BM-25.0A
- Subbase – 10 inches Aggregate Base Material, Type I, Size No. 21B connected to a standard UD-4 edge drain located beneath the outer edge of the paved shoulder. Increase the bottom depth (thickness) of the VDOT 21B as required to properly convey water from beneath the main travel lanes into the edge drains.

  Modified UD-1 shall be provided in lieu of standard UD-4 edgedrain for pavement sub-drainage in areas of high groundwater, springs or deep (> 15 feet) cuts; the modification consists of wrapping the aggregate with geotextile drainage fabric.

  On the high side of super-elevated sections, where the new pavement will drain towards the existing pavement, provide 10 inches Cement Treated Aggregate (CTA) (Aggregate Base Material, Type I, Size No. 21A, pugmill mixed with 4% hydraulic cement by weight in lieu of 21B).

6.3 Temporary Pavement for Maintenance of Traffic

The design-builder shall be responsible for any temporary pavement design. Temporary pavements shall be designed in accordance with the AASHTO Guide for the Design of Pavement Structures (1993 edition) and the VDOT Materials Division’s Manual of Instructions. All temporary pavement designs shall be submitted to the Concessionaire for review. All temporary pavement shall be completely removed once it is no longer in service. All temporary pavement designs for the 395 Express Lanes project shall have a minimum six inches of asphalt concrete and shall meet the following minimum design criteria:

- Design Life – 6 months minimum
- Reliability – 85% minimum
- Initial Serviceability – 4.2 minimum
- Terminal Serviceability – 2.8 minimum
- Standard Deviation – 0.49 minimum
- CBR value for subgrade soils determined by laboratory tests.

Note: The existing paved shoulders shall not be used for maintenance of traffic in their present condition unless approved by the Concessionaire.

The minimum pavement sections require that proper grading be maintained to direct surface water away from paved areas and to provide for efficient runoff from surrounding areas. Control of both surface and ground water will be a very important consideration for the overall performance of these pavement designs.

Any utility excavations or excavations for storm drains within pavement areas shall be backfilled with compacted structural fill in accordance with applicable sections of the Road and Bridge specifications and applicable special provisions.

Standard UD-4 edge drains will be required for all pavements on this project. Modified UD-1 underdrain shall be provided in lieu of standard UD-4 edge drain for pavement sub-drainage in areas of high ground water, springs or cuts in excess of 15 feet; the modification consists of wrapping the aggregate with geotextile drainage fabric. Standard Combination Underdrain (CD-1) shall be provided at the lower end of cuts. Standard Combination Underdrain (CD-2) shall be provided at grade sags, bridge approaches, and at the lower end of undercut areas.

Hydraulic cement concrete sidewalk shall have a minimum base course of 4" of aggregate base material Type I, Size No. 21A or No. 21B.

7.0 GEOTECHNICAL ENGINEERING CONSIDERATIONS FOR DESIGN AND CONSTRUCTION

7.1 Earthwork

Cuts and fills are anticipated to be minimal over the length of the project and generally associated with removal of pavements, preparation of subgrades in pavement reconstruction areas, and minor re-grading. Re-grading of slopes may be required outside the HOV lanes in conjunction with sound barrier wall construction.

Earthwork issues that should be considered during design may include, but are not limited to, subgrade preparation, subdrainage, compacted fill placement, allowable fill slope angles, evaluation of stability and settlement in both design and construction for retained fills and non-retained fills, both existing and proposed, throughout the project.

It will be the design-builder's responsibility to ensure that the stability and settlements of the embankments have been designed to the tolerances specified in the RFP for this project.

For subgrades, it will be important to address appropriate methods for evaluation of subgrade suitability, and procedures for mitigating unsuitable subgrade materials. With respect to subdrainage, the design-builder shall identify areas where subdrainage is needed beyond that required by the standard VDOT specifications/special provisions and design the appropriate types of subdrainage. The design-builder shall evaluate the suitability of on-site soils for use as fill or backfill with respect to soil types, CBR values, and moisture contents as discussed in Section 7.3, below.

As discussed in Section 5.1, Potomac Formation high-plasticity clays/silts (sometimes referred to as "marine" clays) are known to exist within the limits of the proposed construction. These clays/silts have been identified by Fairfax and Prince William counties as "problem" soils that require special treatment during design and construction but they are also known to be present in Stafford County and in the northern end of Spotsylvania County.

Identification of the location, quantities and treatments of high-plasticity Potomac Formation clay and silt soils will be important to determine accurate cost estimates for this project. VDOT has successfully used these materials by compacting them in confined embankment fills and capping them with suitable subgrade fill material. This method of treatment of highly plastic soils will be acceptable on this project provided it is adequately engineered and constructed.

Twenty-eight bulk samples of subgrade soils were collected from the test borings and tested in the laboratory for natural moisture content, classification, Standard Proctor, CBR, and Resilient Modulus (26 out of 28). The test results are presented in Appendix C and summarized in Table C-1.

Moisture conditioning of the on-site soils should be anticipated and considered in design and construction of the project. The clay and silt soils will likely be difficult to compact if wet of optimum moisture content and/or during periods of wet weather due to their propensity to absorb and retain water. Therefore, it will be important that the design-builder address the potential impact that these soils could have on earthwork operations and how they should be treated during construction. As an alternative to aeration and/or mechanical drying, the design-builder may elect to use pelletized quick lime to dry soils that are excessively wet provided dust is adequately controlled.

7.2 Slope Design

Cut and fill slopes shall be no steeper than 2H:1V. All cut and fill slopes shall be analyzed and designed in accordance with the most recent version of Chapter III of the VDOT Materials Division's Manual of Instructions. All cut and fill slopes shall be designed to be stable for the interim construction stages, for the end-of-construction condition, and for design-life conditions.

7.3 Unsuitable Materials

Unsuitable materials for use as embankment fill, bedding for structures, and in cut areas for subgrade directly beneath pavements are defined as any soils with one or more of the following properties: a liquid limit greater than 45, a plasticity index greater than 25, classifies as CH, MH, OH and OL in accordance with the Unified Soil Classification System (USCS), contains more than 5% by weight organic matter, a California Bearing Ratio (CBR) value less than 5 and/or a swell greater than 5% as determined from CBR testing in accordance with VTM-8. Soils that are otherwise suitable, but are in a condition that is +/- 3% of optimum moisture content (i.e. saturated or very dry and/or very loose or very soft coarse/fine grained soils that exhibit excessive pumping, weaving or rutting under the weight of construction equipment) are also considered unsuitable unless they can be moisture conditioned to an acceptable moisture content range that allows adequate compaction to meet project specifications. Based upon the available geologic/soils mapping and local experience, unsuitable soils consisting of highly plastic clays and elastic silts, chemically aggressive soils and soils with Design CBR values less than 5 should be expected to be encountered on this project.

Unsuitable soils in the pavement subgrade that cannot be made suitable by moisture conditioning shall be removed and replaced using the 395 Design-Build Contract Unit Prices {Undercut Unsuitable Subgrade, Replace Subgrade Undercut with Select Material - Type 1 Fill (Minimum CBR 30), Subgrade Geotextile Fabric from Part 1, Attachment C and D} at locations and depths as recommended by the Design-Builder (and as approved by the Concessionaire) using the following methods:

A. Complete removal and replacement with Select Material - Type 1 Fill (Minimum CBR 30) per Section 207 of the 2016 Road and Bridge Specifications to a depth of at least two feet below final pavement subgrade. All unsuitable materials shall be disposed off-site at no additional cost to the Concessionaire.

B. In addition to the removal and replacement method as described in Item A., if unsuitable soils are deeper than two feet below pavement subgrade, at the Concessionaire's direction, the Design-Builder shall also place a layer of Subgrade Stabilization Geotextile Fabric per the VDOT Approved Product List and Section 245.03 (d) of the 2016 Road and Bridge Specifications at the bottom of the 2 feet deep of the unsuitable removal cut prior to placement of Select Material - Type 1 Fill (Minimum CBR 30) to assist in bridging the low-strength deeper unsuitable soils. UD-4 Edge drains shall be lowered as necessary to provide positive drainage for the Select Material.

Based on the test boring data, unsuitable soils should be anticipated within approximately four vertical feet of subgrade elevation at some locations. Table A-5 in Appendix A summarizes boring locations where soil samples tested in the laboratory meet the above requirements for unsuitable soils. In addition, Table A-5 summarizes locations where:

1. The water table was observed at depths of less than approximately five feet (also refer to Table 4 for subsurface water observations – Note these areas may also need additional UD-1 Underdrains),
2. Soil samples were observed to be “wet” during drilling, and
3. SPT N-values were observed to be less than 4 blows per foot (indicating very loose granular soils or soft/very soft cohesive soils).

The final determination of unsuitable soil limits and quantities is the responsibility of the Design-Builder once the final detailed design subsurface investigation is completed. Unsuitable soils in addition to those identified in this report and the preliminary subsurface investigation described herein are likely to be encountered. The design-builder should consider this when preparing his bid. Excessively soft/loose or saturated soils not located beneath and/or impacting the pavement subgrade may also be unsuitable based on the Design-Builders investigation and analysis and must be removed to provide adequate support for embankments, structures or drainage items. This work is considered to be inclusive of the 395 Design-Build Contract Price and will not be paid for under the unsuitable subgrade unit prices described above. The design-builder’s qualified geotechnical engineer must identify unsuitable materials and provide justification for the selected treatment method, or methods, to verify that there will be no adverse effect on the performance of embankments, structures or drainage items. Topsoil or other organic soils are also considered unsuitable for use in embankment fills other than as a cover for slopes for the purpose of establishing vegetative cover. When used as cover for slopes, the thickness of topsoil shall not exceed 12 inches.

8.0 LIMITATIONS

The data contained in this document has been prepared to facilitate preparation of the price proposal for this project and should not be solely relied upon for the final design and construction of this project. A design level geotechnical investigation must be performed by the design-builder to verify and augment the information included in this document. Reference should be made to the 395 Design-Build Contract for further information regarding required design investigations and the identification, resolution and responsibility for differing site conditions.

The data included in this report depict the subsurface soil, ground water, and existing pavement conditions at the specific locations where the borings were performed. These conditions may vary at other locations beyond, or between, these specific locations. Accordingly, the Concessionaire does not warrant or guarantee that the information provided on the logs, or in this report, can be projected as indicative of conditions beyond the limits of the borings, and any such projection is purely interpretive. In addition, the ground water levels recorded on the boring logs indicate the ground water conditions that existed at the time of the investigation. Ground water levels may vary considerably, with time, according to prevailing climate, rainfall, surface run-off, evaporation, construction and other factors.

The data are made available to bidders in order that they may have access to subsurface data identical to that which is possessed by the Concessionaire, and are not intended as a substitute for personal investigation, interpretation and judgment by others. Also, the information contained herein represents borings that were performed by the Concessionaire and may not represent all of the borings performed on the project, particularly if consultant designers performed work under self-contained geotechnical/design contracts.

The minimum pavement sections and discussion of geotechnical considerations as presented in this report are based on the information revealed by our exploration. We have attempted to provide for normal contingencies, but the possibility remains that unexpected conditions may be encountered during subsequent site explorations and construction. The design-builder must perform additional test borings and laboratory testing to develop the design for this project and to meet the minimum requirements outlined in Chapter III of the current VDOT Material Division's Manual of Instructions and the current AASHTO LRFD Bridge Design Specifications, 2014 and VDOT Modifications.

We have endeavored to complete the services identified herein in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions as this project.

9.0 REFERENCES

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395 EXPRESS LANES

APPENDIX A

FIGURES AND TABLES