IH-635 MANAGED LANES PROJECT

GEOTECHNICAL INTERPRETIVE REPORT FOR THE REFERENCE SCHEMATIC - SEGMENT A

prepared for:

Texas Department of Transportation
Dallas District Office

prepared by:

Lachel Felice & Associates, Inc.
Dallas, Texas

JUNE 2006
June 12, 2006

John D. Hudspeth, P.E.
Project Manager LBJ Corridor
Texas Department of Transportation
4777 E. Highway 80
Mesquite, TX 75150

Subject: IH-635 Managed Lanes Project
Geotechnical Interpretative Report for the Reference Schematic – Segment A

Dear Mr. Hudspeth:

Lachel Felice & Associates, Inc. (LF&A) is pleased to submit this Geotechnical Interpretive Report (GIR) on behalf of the LBJ Mobility Partners, for the subject project. Two hard copies and one electronic copy on CD are attached for your use.

The GIR presents a compilation of the geotechnical data developed to date from the three previous field and laboratory investigations performed and certified by others, and it provides LF&A's interpretations and recommendations with regard to design and construction considerations for the mined tunnels, cut and cover tunnels and U-wall sections of the project. We are certifying herewith only our conclusions, interpretations and recommendations which we developed pursuant to our review of the reports described above and other information available in the literature.

Please contact us if you have any questions. LF&A appreciates the opportunity to work with you on this most interesting project.

Sincerely,

Lachel Felice & Associates, Inc.

James F. Moore, P.E.
Senior Geotechnical Engineer

George R. Teetes, Ph.D., P.E.
Project Manager

7907 Roundrock Road, Dallas, TX 75248 • (972) 980-0037 • Fax (972) 980-9048
TABLE OF CONTENTS

TABLE OF CONTENTS............................................................................................. ii
LIST OF TABLES ...................................................................................................... v
LIST OF FIGURES ................................................................................................... vi
1.0  INTRODUCTION ...............................................................................................1
  1.1  Project Overview ............................................................................................ 1
  1.2  Purpose and Scope ........................................................................................ 1
  1.3  Sources of Information ................................................................................... 1
    1.3.1  General ....................................................................................................1
    1.3.2  Site Specific Investigations and Reports .................................................2
  1.4  Limitations and Disclaimer ............................................................................. 4
2.0  PROJECT SETTING .........................................................................................6
  2.1  General ........................................................................................................... 6
  2.2  Segment A Configuration ............................................................................... 6
  2.3  Physiography and Regional Geology ............................................................. 7
  2.4  Stratigraphy .................................................................................................... 7
    2.4.1  Overburden ..............................................................................................7
    2.4.2  Bedrock ....................................................................................................9
  2.5  Geologic Structure ........................................................................................ 10
  2.6  Groundwater Conditions from a Regional Geologic Perspective ................. 11
  2.7  Regional Seismicity ...................................................................................... 11
3.0  GROUND CHARACTERIZATION AND SITE CONDITIONS ..........................13
  3.1  Engineering Properties of Soil and Rock...................................................... 13
    3.1.1  Overburden Soils ...................................................................................13
    3.1.2  Bedrock ..................................................................................................17
  3.2  Hydrogeologic Setting .................................................................................. 26
    3.2.1  Hydraulic Conductivity ...........................................................................26
    3.2.2  Groundwater Elevations ........................................................................27
  3.3  In Situ Stress Conditions .............................................................................. 30
  3.4  Environmental Considerations ..................................................................... 30
  3.5  Geologic Conditions along Segment A......................................................... 30
    3.5.1  Station 67+00 to 92+00 (Josey Lane to Webb Chapel Road) ...............31
    3.5.2  Station 92+00 to 130+00 (Webb Chapel to 1,800 ft West of Marsh Lane) .....................................................................................................32
    3.5.3  Station 130+00 to 163+00 (1,800 ft West of Marsh Lane to 1,500 ft East of Marsh Lane) ..............................................................................32
    3.5.4  Station 163+00 to 191+00/195+00 (1,500 ft East of Marsh Lane to West Mined Tunnel Portals) ...........................................................................33
    3.5.5  West Mined Tunnel Portals to Station 264+00 (Mined Tunnel) ..............34
    3.5.6  Station 264+00 to 305+00 (Mined Tunnel) ............................................35
    3.5.7  Station 305+00 to 334+00 (Mined Tunnels to East Mined Tunnel Portals) .....................................................................................................35
    3.5.8  Station 334+00 to 356+00 (East Mined Tunnel Portals to 700 ft West of Hillcrest Road) ..............................................................................36
    3.5.9  Station 356+00 to 360+00 (End Below Grade Sections to West of Hillcrest Road) ..................................................................................36
4.0  PREVIOUS CONSTRUCTION EXPERIENCE ..................................................37
  4.1  General .......................................................................................................... 37
  4.2  Mined Tunnels in Austin Chalk .......................................................................37
  4.3  Vertical Cuts in Austin Chalk .........................................................................37
4.3.1 DART Mockingbird Station .................................................................37
4.3.2 Addison Airport Toll Tunnel ...............................................................38
4.4 Vertical Cuts in Eagle Ford Shale ............................................................39
4.4.1 President George Bush Turnpike, Carrollton Section .........................39
4.4.2 President George Bush Turnpike, Las Colinas Section .......................40
4.4.3 One Main Place, Downtown Dallas ....................................................40
4.4.4 I-30 Widening, Dallas, Loop 12 to Sylvan .........................................40
4.4.5 Landfill Case History, Slope Failure ..................................................41
4.4.6 DFW Airport, Slope Failure During Construction of Utility Corridor .......41
5.0 DESIGN CONSIDERATIONS FOR MINED TUNNELS .........................42
5.1 General ....................................................................................................42
5.2 Ground Response .....................................................................................42
5.3 Initial Ground Support .............................................................................43
5.3.1 Mined Tunnels and Associated Underground Structures ....................43
5.3.2 Portal Design .......................................................................................43
5.4 Final Lining Design ..................................................................................44
5.5 Argillaceous Zones ..................................................................................44
5.6 Bentonite Seams ......................................................................................44
5.7 Groundwater and Seepage Control ..........................................................45
5.8 Combustible and Toxic Gases ..................................................................46
5.9 Corrosion Potential ..................................................................................46
5.10 Recommended Additional Subsurface Investigations .............................46
6.0 DESIGN CONSIDERATIONS FOR CUT-AND-COVER TUNNEL AND U-WALL SECTIONS .................................................................47
6.1 General ....................................................................................................47
6.2 Long-term Earth Pressure Loading Basis .................................................47
6.2.1 At-rest Earth Pressures (Ko) in Clay and Shale ....................................47
6.2.2 Special Considerations Regarding Lateral Swell Pressures due to Swelling Clays and Clay Shales ..........................................................48
6.2.3 Methods of Stabilizing Vertical Cuts in Weathered and Fresh Austin Chalk ..........................................................................................49
6.2.4 Groundwater Level at the Ground Surface ...........................................50
6.2.5 Surcharge Loads ..................................................................................50
6.3 Vertical Swell Design Considerations ......................................................50
6.4 Long-term Loading on Walls (by Station) ................................................51
6.4.1 Station 67+00 to Station 92+00 (Josey Lane to Webb Chapel Road) ..........51
6.4.2 Station 92+00 to Station 130+00 (Webb Chapel to 1,800 ft West of Marsh Lane) ................................................................................52
6.4.3 Station 130+00 to Station 163+00 (1,800 ft West of Marsh Lane to 1,500 ft East of Marsh Lane) ......................................................52
6.4.4 Station 163+00 to Station 191+00/195+00 (1,500 ft East of Marsh Lane to West Tunnel Portals) ..........................................................53
6.4.5 Station 334+00 to Station 356+00 (East Portals to 700 ft West of Hillcrest Road) ..............................................................................54
6.4.6 Station 356+00 to End Below Grade Sections Station 360+00 ..........54
7.0 CONSTRUCTION CONSIDERATIONS FOR MINED TUNNELS ..................56
7.1 Mechanical Excavation ..........................................................................56
7.2 Muck Handling .......................................................................................56
7.3 Groundwater and Seepage Control (Mined Tunnel Construction) ............57
7.4 Geotechnical Instrumentation and Monitoring .......................................57
8.0 CONSTRUCTION CONSIDERATIONS FOR CUT-AND-COVER TUNNEL AND U-WALL SECTIONS ............................................................... 58
8.1 Temporary Excavation Support........................................................................ 58
8.2 Temporary Excavation Support; Offsite Impacts.............................................. 59
8.3 Excavation Equipment and Methodology..................................................... 59
8.4 Compaction Control...................................................................................... 60
8.5 Right-of-Way Issues..................................................................................... 60
8.6 Construction Access..................................................................................... 60
8.7 Groundwater Inflow...................................................................................... 60
  8.7.1 Flow From Soils in the Cut-and-Cover Tunnel and U-wall Sections..... 61
  8.7.2 Fracture Flow in the Cut-and-Cover Tunnel and U-wall Sections...... 61
APPENDIX A - Overview of Swelling Soils and Clay Shales............................. 62
  A.1 Theoretical Considerations and Laboratory Indicators of Swell Potential... 62
  A.2 Practical Considerations.............................................................................. 64
APPENDIX B - REFERENCES ............................................................................. 66
APPENDIX C - FIGURES ..................................................................................... 71
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-1</td>
<td>Roadway Configuration and Section Lengths for “Segment A&quot; per the Reference Schematic</td>
<td>6</td>
</tr>
<tr>
<td>2-2</td>
<td>Height of Bentonite Marker Bed above the Eagle Ford Shale</td>
<td>10</td>
</tr>
<tr>
<td>3-1</td>
<td>Soil/Overburden Engineering Properties</td>
<td>16</td>
</tr>
<tr>
<td>3-2</td>
<td>Austin Chalk Engineering Properties</td>
<td>20</td>
</tr>
<tr>
<td>3-3</td>
<td>Bentonite Marker Bed Engineering Properties</td>
<td>22</td>
</tr>
<tr>
<td>3-4</td>
<td>Bentonite Beds along Mined Tunnel Alignment</td>
<td>22</td>
</tr>
<tr>
<td>3-5</td>
<td>Eagle Ford Shale Engineering Properties</td>
<td>25</td>
</tr>
<tr>
<td>3-6</td>
<td>Summary of Field Hydraulic Conductivity Testing</td>
<td>26</td>
</tr>
<tr>
<td>3-7</td>
<td>Standpipe and Vibrating Wire Piezometer Data (April 1, 2005)</td>
<td>29</td>
</tr>
<tr>
<td>3-8</td>
<td>Sections of Similar Geologic Conditions Along Segment A</td>
<td>31</td>
</tr>
<tr>
<td>6-1</td>
<td>Preliminary Design Earth Pressures and Assumptions</td>
<td>55</td>
</tr>
</tbody>
</table>
# LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure 1.</td>
<td>Regional Geology and Site Location Map</td>
<td>72</td>
</tr>
<tr>
<td>Figure 2.</td>
<td>Boring Location Plan (Figures 2a - 2k)</td>
<td>73</td>
</tr>
<tr>
<td>Figure 3.</td>
<td>Site Geology and Topography</td>
<td>84</td>
</tr>
<tr>
<td>Figure 4.</td>
<td>Geotechnical Profile (Figures 4a - 4c)</td>
<td>85</td>
</tr>
<tr>
<td>Figure 5.</td>
<td>Plasticity Charts</td>
<td>88</td>
</tr>
<tr>
<td>Figure 6.</td>
<td>Shear Strength Test Results</td>
<td>89</td>
</tr>
<tr>
<td>Figure 7.</td>
<td>Activity Indices (Volume Change Potential)</td>
<td>90</td>
</tr>
<tr>
<td>Figure 8.</td>
<td>Unconfined Compressive Strength in Austin Chalk along Mined Tunnel Alignment (Figures 8a - 8b)</td>
<td>91</td>
</tr>
<tr>
<td>Figure 9.</td>
<td>Argillaceous Content along Mined Tunnel Alignment (Figures 9a - 9b)</td>
<td>93</td>
</tr>
<tr>
<td>Figure 10.</td>
<td>Discontinuity Data along Mined Tunnel Alignment (Figures 10a - 10b)</td>
<td>95</td>
</tr>
<tr>
<td>Figure 11.</td>
<td>Locations of Previous Construction Projects in Similar Geology</td>
<td>97</td>
</tr>
</tbody>
</table>
1.0 INTRODUCTION

1.1 Project Overview

The IH-635 Managed Lanes Project (the Project) consists of improvements to the existing IH-635 (LBJ Freeway) in the Dallas-Fort Worth metropolitan area and encompasses a portion of the corridor that is approximately 24 miles long extending from Luna Road on the west to IH-30 on the east. This report applies specifically to Segment A of the Project (Figure 1), which extends from just east of the IH-35E interchange at the west end to the IH-635/US-75 Dallas High Five (DHF) interchange at the east end.

1.2 Purpose and Scope

This Geotechnical Interpretive Report (GIR) is intended to provide a summary of geotechnical data and information gathered along Segment A for the "Reference Schematic". The term "Reference Schematic" is used herein to describe a separate document contained in the Reference Information Documents (RID) that depicts one potential solution for configuration of Segments A, B and C. The Reference Schematic was developed by the Procurement Engineering (PcE) team during the preliminary engineering phase of procurement and was developed consistent with the Project constraints identified in the Reference Schematic Criteria column of Exhibit C presented in the Request For Qualifications (LBJMP 2005).

The GIR summarizes the factual geotechnical data gathered to date and presents interpretations of the data along with design and construction considerations based on a range of anticipated construction means, methods and sequences of excavation. During construction, the soil and rock response to excavation and the corresponding ground structure interaction will be dependent upon these items, and as such, the ground response may differ from discussion presented herein. In all cases the Developer shall obtain the necessary geotechnical information and evaluate the soil, rock and groundwater conditions in light of the Developer's Project configuration and planned means and methods.

1.3 Sources of Information

1.3.1 General

Data used in preparing this report were obtained from a wide variety of sources. Much of the general geologic information came from publications provided by state and local associations, universities, and federal geologic surveys. Data from previous open cut and mined tunnel projects in the Dallas area were also used for comparison with this Project.
1.3.2 Site Specific Investigations and Reports

Three separate geotechnical investigation programs have been conducted for this Project, the results of which were presented in reports described below. These include a 1998 investigation by Terra-Mar, Inc., and two subsequent investigations by Fugro Consultants LP, starting in 2003 and extending into 2005. The Terra-Mar, Inc. investigation was performed to evaluate the LBJ corridor for specific managed lane configurations at the feasibility level during 1998, while the Fugro investigations were performed in a phased approach to expand the coverage of available geotechnical information along the proposed alignment and to gather necessary geotechnical parameters to be used for preliminary design. The locations of all borings from the three investigation programs are presented in Figure 2.

The geotechnical reports from each of the previous investigations are as follows:

- Geotechnical Data Report, LBJ Corridor Study Project, Dallas, Texas, Terra-Mar, Inc., December 29, 1998
- Phase 1 Geotechnical Data Report, IH-635 (LBJ Freeway), Section 4 – West, Fugro South, Inc., April 6, 2004
- Phase 1 Geotechnical Baseline Report, IH-635 (LBJ Freeway), Section 4 – West, Fugro Consultants LP, May 12, 2004
- Phase 2 Geotechnical Data Report, IH-635 (LBJ Freeway), Section 4 – West, Fugro Consultants LP, May 18, 2005

1.3.2.1 Feasibility Level Investigation

The 1998 Terra-Mar investigation includes the results of 14 test borings drilled in October 1998 along the centerline of the proposed project, which essentially followed the centerline alignment of existing IH-635 from the US-75 interchange on the east to the intersection with IH-35E on the west. The field-testing included Standard Penetration Testing (SPT) and sampling in unconsolidated material, and determination of percent core recovery and Rock Quality Designation (RQD), of NX size rock core. Laboratory tests on soils included basic classification tests, and limited strength testing (unconfined compressive strength of selected soil samples). Laboratory testing for rock samples included unconfined compression strength and slake durability on selected rock core specimens.

1.3.2.2 Phase 1 and Phase 2 Geotechnical Investigations

The Phase 1 geotechnical investigation performed by Fugro South, Inc. was an extensive boring and testing program involving drilling and sampling of 93 test borings along the proposed alignment described in the 2004 Environmental Assessment (TxDOT 2004). The boring program was laid out typically with one boring or pair of borings approximately every 500 feet along the alignment, and the test borings were arranged in "left", "right" and "center" profiles. The left profile essentially followed the proposed
westbound HOV/Toll lanes from just east of existing IH-35E to just west of US-75, and the right profile followed similarly along the proposed eastbound HOV/Toll lanes. The center profile of borings followed the mined tunnels for the central 1.6 miles of their overall length and was essentially drilled along the existing IH-635 centerline.

Borings for the Phase 1 investigation included SPT's and penetration testing in accordance with test method Tex-132E, Texas Cone Penetration (TxDOT 2005). Groundwater observations were recorded in eight standpipe piezometers and eleven vibrating wire piezometers at the time the borings were drilled, and for up to one year following installation. Field permeability testing was performed by means of packer tests within selected rock strata in 13 of the test borings. Laboratory testing for this program included classification and index testing, moisture content, unconfined compression and triaxial strength testing of representative samples intended to represent several key soil strata including alluvial clays, residual clays and weathered rock. A limited number of simple swell tests were also performed. For the rock core obtained during the investigation, the core was logged for structure descriptions and visual observation of bentonitic and argillaceous content. Laboratory testing included unconfined compression and unconsolidated undrained (UU) triaxial testing in both the weathered and fresh portions of the bedrock units encountered. Field and laboratory test data from the investigation were summarized in the Phase 1 Geotechnical Data Report (Phase 1 GDR).

In addition to the Phase 1 GDR, a Phase 1 Geotechnical Baseline Report (Phase 1 GBR) was prepared by Fugro Consultants LP and subconsultant Brierley Associates to present engineering interpretations of the above described test data relevant to design of the mined tunnel, cut-and-cover tunnel and U-wall sections for the Project configuration described by the 2004 Environmental Assessment. While the report was titled "...Geotechnical Baseline Report", it was prepared prior to preliminary engineering, and not in conjunction with the Procurement Engineering design effort. The report is made available by TxDOT as a reference document only, and the Developer should not rely on the information contained therein, but rather on his own assessment and interpretation of the geologic conditions in light of the Developer's planned construction means and methods.

Following the Phase 1 investigation, a Phase 2 geotechnical investigation was performed to address design requirements identified by the PcE during the preliminary engineering effort, and to fill in gaps in the geotechnical information gathered during the Phase 1 program. The Phase 2 investigation for Segment A included 41 additional borings (33 vertical borings and 8 angled borings). The angled borings were located at proposed portal locations and at locations where faulting was expected. In the 8 angled holes, downhole resistivity-based micro-imaging techniques were used to gather information on the number, location, type and orientation of discontinuities at those locations. Groundwater observations were recorded in seven standpipe piezometers and eight vibrating wire piezometers at the time the borings were drilled and for approximately 3 months following installation. Field permeability testing was performed by means of packer tests within selected rock strata in 5 of the Phase 2 borings.

In addition, the laboratory testing effort for the Phase 2 investigation was expanded from the Phase 1 testing program to include specially testing for rock properties that impact the ability to mechanically excavate tunnel openings in rock. These tests, performed by the Earth Mechanics Institute (EMI) at the Colorado School of Mines in Golden,
Colorado, included petrographic analysis, abrasion and hardness tests, punch penetration and Cerchar abrasivity. These data were further utilized by representatives of EMI to produce an estimate of performance for roadheader excavation and bit consumption. The field and laboratory test data from the Phase 2 investigation, including the specialty testing and roadheader performance prediction are presented in the Phase 2 Geotechnical Data Report (Phase 2 GDR).

For this GIR, the three previous reports prepared by Fugro for the Phase 1 and Phase 2 geotechnical investigations will be collectively referred to as the “Referenced Fugro Reports.”

1.3.2.3 Boring Nomenclature

The locations of the 148 total borings drilled to date for Segment A of the Reference Schematic are presented in Figures 2a-2k.

The first fourteen borings drilled by Terra-Mar, Inc. in 1998 are designated TUN-1 through TUN-14 and the boring designations increase numerically from east to west.

The borehole designation system established by TxDOT for the Phase 1 and Phase 2 geotechnical investigations and taken directly from the Referenced Fugro Reports is as follows: "Each boring number begins with BE, BW or T. The BE and BW borings are generally for “non-tunnel” borings on the east and west ends of the Project, respectively. The “tunnel” borings are designated with a T. Following the first letters is a numeral designation. The numbers increase from west to east as does the Project reference stationing. Thus, 10 is east of 5, etc. Borings with the same number are located at approximately the same station. The numbers are then followed by L, R, or C. These represent the boring position in reference to the Project centerline, looking east, (i.e., L is left and north of the centerline, R is right and south of the centerline, and C is approximately along the Project centerline. Several of the borings have the L, R, or C followed by the numeral 1. This number represents a 2nd boring left or right of centerline at that location. For example: BE1L1. This is the first boring east of the final tunnel boring “T”. It is located left of centerline, and left of BE1L, also left of centerline. There is also a BE1R right of centerline at this station" (Fugro 2004b). Finally, for borings drilled during the Phase 2 investigation, the nomenclature is the same except that all Phase 2 borings are assigned a prefix 2.

1.4 Limitations and Disclaimer

This report was prepared for TxDOT as a reference document for the IH-635 Managed Lanes Project. No other use of this document is authorized.

The provision of this GIR is not a guaranty, representation, warranty nor indication, expressed or implied, that the conditions described herein will actually be encountered. The subsurface conditions described in this report are based on the three geotechnical site investigations that were completed, reported on, and certified by other licensed professional engineers as described above. These investigations included interpretations of material descriptions, data, properties and behavior, and determination of boundaries between various strata by qualified professionals in the preparation of the
respective reports. We based our conclusions and inferences on the assumption that the data and interpretations presented and certified by the previous investigators was accurate. These interpretations made by the previous investigators have been deemed reasonable, but it is recognized that other interpretations are possible. Additionally, subsurface conditions at boring locations may differ from conditions at other locations, including between specific boring locations described in this report. The stratification lines designating the boundary between soil and/or rock types shown on the geologic profiles and other figures represent approximate boundaries developed through one of many possible geological interpretations. Actual stratigraphy may differ from the stratigraphy shown on the profile, and the transition between materials may be gradational and/or one or more strata may be absent at specific locations.

Furthermore, the Developer may not exclusively rely upon this report for the planning or performance of any aspect of its work, including without limitation the selection, design, or implementation of the means, methods, techniques, sequences and procedures of construction to be employed by the Developer. Rather, the Developer must procure the geotechnical data required for Project design and construction, and must evaluate design and construction related issues on the basis of the Developer's own knowledge and experience in the local area, and/or on the basis of similar projects, taking into account the Developer's own proposed construction methods and procedures.
2.0 PROJECT SETTING

2.1 General

The following sections summarize the regional geologic setting as it applies to the Project area. Stratigraphy and structural geology as they apply to the Project alignment are presented and discussed in context of the overburden soils and rock strata that will be encountered during Project construction. Groundwater conditions and seismicity are also discussed from a regional perspective.

2.2 Segment A Configuration

As presented in the Reference Schematic, Segment A of the IH-635 Managed Lanes Project is approximately 5.5 miles long and follows the alignment of the existing IH-635 (LBJ Freeway). As described by Fugro in the Phase 1 GBR: “All of the existing right-of-way is currently occupied by the existing freeway and its ancillary structures such as ramps, service roads, retaining walls, engineered slopes, storm water management facilities (including ditches, swales and drains), and several overpass and underpass bridge structures. The existing right of way also includes buried utilities such as sewer, water, gas, and electrical. The existing right-of-way is bordered by various sections of residential, retail, and commercial structures, including several high-occupancy buildings such as offices, hotels, and apartments” (Fugro 2004b).

Segment A construction extends from near Josey Lane at the west end to near Hillcrest Road at the east end. As detailed in the Reference Schematic, the Project includes twin mined tunnels of approximately 2.7 miles in length and approximately 2.8 miles of twin cut-and-cover tunnel and U-wall sections. The mined tunnel section is located in the central portion of the Project (from about Midway Road on the west to approximately 1,000 feet east of the Preston Road overpass on the east). It is accessed from surface roadway sections at either end through transitional U-wall and cut-and-cover tunnel sections. The section lengths and starting stationing information for each section are shown in Table 2-1.

Table 2-1 Roadway Configuration and Section Lengths for “Segment A” per the Reference Schematic

<table>
<thead>
<tr>
<th>Section Type</th>
<th>Eastbound Lanes</th>
<th>Westbound Lanes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Beginning Station, ft</td>
<td>Section Length, ft</td>
</tr>
<tr>
<td>U-Wall</td>
<td>67+00</td>
<td>800</td>
</tr>
<tr>
<td>Cut-and-Cover</td>
<td>75+00</td>
<td>11,600</td>
</tr>
<tr>
<td>Mined Tunnel</td>
<td>191+00</td>
<td>14,300</td>
</tr>
<tr>
<td>Cut-and-Cover</td>
<td>334+00</td>
<td>1,200</td>
</tr>
<tr>
<td>U-Wall</td>
<td>346+00</td>
<td>1,400</td>
</tr>
<tr>
<td>End of U-Wall</td>
<td>360+00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total (ft)</td>
<td>29,300</td>
</tr>
<tr>
<td></td>
<td>Total (mi)</td>
<td>5.5</td>
</tr>
</tbody>
</table>
2.3 Physiography and Regional Geology

The IH-635 Managed Lanes Project is located in North-Central Texas in northwest Dallas County. The area is underlain by rocks of the Upper Cretaceous System, with limestone of the Austin Group forming the uppermost rocks and clay shale of the Eagle Ford Group underlying the Austin. These strata dip generally to the southeast, with the overall topography also following this general slope. The regional geology and site topography are presented on Figure 3.

The bedrock geology of Dallas is a result of geologic processes which were active during the Cenozoic era. Following the withdrawal of the Cretaceous seas, all Cenozoic and some Cretaceous sediments were eroded by the ancestral Trinity River and its tributaries, which formed broad, flat, floodplains that are flanked by several Quaternary-aged terrace deposits. There are no existing major stream crossings within the area of the proposed Segment A alignment. However, based on early regional topographic maps (USGS 1959a-b), and available air photos (Tobin International 1938) taken prior to existing freeway construction, one portion of the alignment appears to be located over an unnamed tributary to Farmers Branch Creek that was likely channeled and drained during early highway construction. In addition, north-south draining culverts at the IH-635/Dallas North Tollway (DNT) interchange have been installed at locations that are visible on historic aerial photography as localized low topographic areas.

More detail regarding the regional geology is presented in the Referenced Fugro Reports.

2.4 Stratigraphy

In the Project area, the surficial geologic strata of relevance are the Eagle Ford Group, the Austin Group, the overlying residual soils derived from them, sandy and clayey alluvial soils and fill. Figures 1 and 3 present the general location of the Project in context of these geologic units. The strata are further depicted in the geologic profile following the Segment A alignment from west to east (Figure 4).

2.4.1 Overburden

Soil deposits along the Project alignment consist mainly of residual soils derived from the Austin and Eagle Ford Groups, and sandy and clayey alluvial soils and fill. In addition, due to the developed nature of the existing IH-635 corridor, a thin veneer of fill exists at the ground surface over much of the investigated area and was sampled in most borings.

2.4.1.1 Fill

Fill materials may be encountered throughout the Project alignment. These are typically very stiff to hard clays of medium to high plasticity, reflecting to some degree the local native soils which are presumed to have served as borrow sources. In many places the strength and plasticity of the fills are indistinguishable from the underlying native soils. The thickness of the fill is highly variable across the site, with the thickness recorded in
the borings ranging from several inches to as much as 15 ft. Due to this variability, the reported thickness of fill materials should be viewed as accurate only at each boring location. In areas where the thickness of fill materials may impact design, additional geotechnical information should be collected by the Developer on a site-specific basis as the design progresses.

2.4.1.2 Alluvial Soils

Two broad classes of alluvial soils exist in the Project area, described here as sandy and clayey alluvial soils. The sandy alluvial soils are attributable either to riverbed deposits or lower terraces of the ancestral Trinity River (Eubank 1965), while the clayey alluvial soils are attributable either to upper terrace deposits of the Trinity River (Eubank 1965), or are soils that are the product of erosion and nearby deposition of residual soils or intermittent streams. These erosional/depositional soil deposits will be referred to herein as colluvial soils.

Sandy alluvial deposits exist in the Trinity River floodplain and extend down to the top of bedrock, which may be as much as 60 feet below existing grade in the Project area, and potentially deeper in the actual floodplain. These soils tend to be composed of sands and gravels. Where these granular deposits exist along the Project, generally west of Webb Chapel Road, they tend to be found below a depth of 18 feet, and are typically of limited thickness (e.g., about 5 feet), suggesting their origin could be a buried channel or stream meander. A few borings indicated these granular deposits could be deeper, in some cases extending down to bedrock.

The clayey alluvial soils that are believed to be remnants of terraces from the ancestral Trinity River (Eubank 1965) are generally found from the ground surface down to a depth of approximately 15 feet, and are mainly high plasticity clays. Such clayey alluvial soils are encountered in borings at the west end of the Project along the gentle slope rising out of the Trinity River bottom up to the ridge formed in the more resistant Austin Group, and on the east end of the Project on the bluff dropping down into the White Rock Creek drainage.

The other class of clayey alluvial soils, described above as colluvial soils, which are higher up on the hillsides along the alignment, originated primarily as residual clays and are likely to exist on the eastern and western slopes at each end of Segment A.

2.4.1.3 Residual Soils

The residual soils to be encountered on this Project are generally high plasticity clayey soils developed by weathering of the parent Eagle Ford and Austin Groups. In the Project area, the residual soil of the Eagle Ford Group is described by the Soil Conservation Service (SCS) as the Houston Black-Urban Land Complex within the Houston Black-Heiden map unit (SCS 1980). The residual soil derived from the Austin Group is described in the same reference as the Austin-Urban Land complex within the Eddy-Stephen-Austin map unit.
2.4.2 Bedrock

As shown in Figures 1 and 3, bedrock units that outcrop in Dallas County from west to east include the Eagle Ford Group, the Austin Group, and the Taylor Group of the Gulfian Series, all of Cretaceous age. The Taylor Group outcrops in eastern Dallas County and will not be encountered on Segment A.

Based on the Project configuration presented in the Reference Schematic, the Eagle Ford Group will not be encountered in the mined tunnel portions of the Project, but will be encountered on the west end of the Project from approximately Station 90+00 to 158+00 in the U-wall and cut-and-cover tunnel sections. The Eagle Ford Group underlies the Austin Group and is frequently referred to as the “Eagle Ford Shale.” It consists of two primary members, the Arcadia Park and the Britton member, of which the upper Arcadia Park member will be encountered for a considerable distance on this Project. The Arcadia Park member consists primarily of a dark-gray to black, montmorillonitic, calcareous to non-calcareous, laminated clay-shale with high shrink-swell potential (Allen and Flanigan 1986). Surles (1987) described the Arcadia Park in the Dallas area as “consisting of gray to dark gray, fissile, calcareous mudstone or clay shale with thin laminae of siltstone, sandstone, and fragmental limestone”.

The lower member of the Eagle Ford Shale, known as the Britton member, is generally more calcareous than the Arcadia Park member. The Britton member may also be encountered, but only on the westernmost end of Segment A, underlying the alluvial terrace deposits of the Trinity River.

Along the Project alignment, the contact between the Eagle Ford Shale and the overlying Austin Group occurs just east of Marsh Lane, where it may be covered by about 20 feet of residual soil. The Austin Group unconformably overlies the Eagle Ford Shale and will be encountered by Project construction from approximately Station 150+00 to Station 360+00. The contact between these strata is an erosional unconformity, and sediments deposited on this surface at the base of the Austin Group are known as the “Fishbed Conglomerate”. The Fishbed Conglomerate typically varies from 1 to 12 feet in thickness in the Dallas area, and is described as arenaceous limestone with abundant marine fossil debris, phosphate nodules, pyrite and marcasite crystals, and reworked materials derived from the underlying Eagle Ford Shale and older sediments (Allen and Flanigan 1986).

The Austin Group is locally referred to as the Austin Chalk. The Austin Chalk is characteristically subdivided into upper, middle, and lower members. The upper and lower members are similar, consisting of limestone with interbedded marl and argillaceous limestones and shales. The middle member is primarily a marl or argillaceous limestone with interbedded limestone. There is a regionally persistent 9-12 inch thick bentonite layer, which is close to the separation of the lower and middle members, and is referred to locally as the Bentonite Marker Bed. Additional continuous to nearly continuous bentonite seams were identified in the borings and are discussed in more detail in the following chapter. It has also been interpreted that the lower member of the Austin Chalk thickens to the north, with the Bentonite Marker Bed having been observed in north central Texas at varying heights above the Eagle Ford Group, as presented in Table 2-2.
Table 2-2  Height of Bentonite Marker Bed above the Eagle Ford Shale

<table>
<thead>
<tr>
<th>Location</th>
<th>Project</th>
<th>Distance from Marker Bed to Eagle Ford Shale Contact (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Waco</td>
<td>Waco Dam</td>
<td>55</td>
</tr>
<tr>
<td>Waxahachie</td>
<td>Superconducting Super Collider (SSC)</td>
<td>70</td>
</tr>
<tr>
<td>Dallas</td>
<td>Dallas Area Rapid Transit Tunnels (DART)</td>
<td>90</td>
</tr>
<tr>
<td>Addison</td>
<td>Addison Airport Toll Tunnel</td>
<td>110</td>
</tr>
</tbody>
</table>

As the study area for this Project is between the Addison Airport Toll Tunnel and the tunnels for the DART project, it would be reasonable to expect the Bentonite Marker Bed to be located between 90 and 110 feet above the Eagle Ford Shale contact. In general this range fits with the geologic data gathered for this Project, which indicates that the Bentonite Marker Bed is on the order of 100 feet above the Eagle Ford Shale across the Project alignment.

### 2.5 Geologic Structure

Several regional tectonic events have created the gently sloping monoclinal structure of the Gulf Coastal plain. Structures resulting from these events include the Ouachita Fold Belt, the East Texas Embayment, the Sabine Uplift, the Llano Uplift, and the Northeast Texas Fault System, which includes the Balcones, Mexico-Talco, and Mount Enterprise Fault Zones. The city of Dallas is located near the northwestern margin of the East Texas Embayment, the eastern margin of the Ouachita Fold Belt, and is north to northwest of the currently mapped Balcones Fault Zone.

In the Dallas area, normal faults are typically mapped, with some reverse faults noted as well. Vertical displacements along the faults are typically less than 15 ft, but Ingels (1959) mapped a fault near the Dallas-Ellis County line, southeast of the city of Dallas with a calculated vertical displacement of 100 ft. Several displacements up to 20 ft were mapped during the DART project (DART 1991) as well. The major faults that are found in Dallas County are noted to strike N10°W (Blakemore 1939), and are believed to have been active during the early Cretaceous to Miocene time (Woodruff 1980).

Along the IH-635 Managed Lanes Project several minor faults oriented roughly north-south are suspected. Using the Bentonite Marker Bed as a datum for calculating vertical displacement, there appear to be at least two locations where several offsets of 10 feet or more occur. One such area is located approximately ½ mile west of the DNT, and another is just east of the DNT (Figure 4). In addition, there is a zone approximately 2,000 feet in length and centered approximately ½ mile east of the DNT in which vertical offsets of 5 to 7 feet appear between adjacent borings.

Specifically, the Austin Chalk is reported to have several dominant joint sets that are related to faulting. Blakemore (1939) reports a dominant joint set striking N65°E. In addition, geologic structural data, compiled and analyzed using stereonets, from the Addison, DART, Cole Park and SSC projects identified joints at N2°E and N55°E as two dominant joint set orientations within the vicinity of the Project. Based on the same stereonet analysis, two other minor joint set orientations were noted at N59°W and
Each of the above joint sets appeared to dip in both directions and at approximately 55-65°. Furthermore, many streams and tributaries of the Trinity River system in the Project vicinity are thought to be joint controlled in that the drainage patterns reflect the joint pattern documented for the bedrock units (Blakemore 1939, and Norton 1965).

The occurrence of high joint frequencies has been documented typically in close proximity to the mapped faults. In addition, some of the joints in the Austin Chalk have been described as non-linear, displaying a curved rather than planar surface (Blakemore 1939). This non-linear description was also observed in the cut slope failure at the Addison Airport Tunnel (see Section 4.3.2).

2.6 Groundwater Conditions from a Regional Geologic Perspective

The groundwater conditions across Segment A are found to exist in three relatively distinct regimes, and appear to be highly dependent upon both local topography and stratigraphy.

At the westernmost end of Segment A, from approximately Station 67+00 to Station 95+00, the stratigraphy is comprised of more permeable, sandy alluvial soils, and the groundwater level typically ranges from 15 to 20 feet below the ground surface. Due to the presence of these granular deposits, the groundwater levels can be expected to vary considerably over time, and may respond to precipitation events or seasonal fluctuation in water levels closer to the Trinity River floodplain.

Continuing to the east, from approximately Station 95+00 to 160+00, as the topography rises up from the Trinity River, and where the stratigraphy is comprised of the less permeable, clayey alluvial soils, the residual soils of the Eagle Ford Shale, and the weathered Eagle Ford Shale, the groundwater levels are generally within 2 to 8 feet of the ground surface. In some locations perched groundwater may also be present atop the relatively impervious materials.

Finally, from approximately Station 160+00 to the easternmost end of Segment A, at Station 360+00, the alignment of the Reference Schematic traverses a regional topographic high on a north-south trending ridge formed in the Austin Chalk. This ridge comprises an area that is drained by small tributary creeks to White Rock Creek to the east and to the Trinity River to the west. Variable thicknesses of weathered Austin Chalk, overlying fresh Austin Chalk are located throughout this portion of the Project, and are covered by a relatively thin soil veneer. Groundwater levels range from essentially at the ground surface to depths of 15 to 20 feet. However, indications of possible artesian conditions at some points along this section were identified in the vibrating wire piezometers installed in the Phase 1 and Phase 2 geotechnical investigations and are discussed in further detail in Chapter 3.

2.7 Regional Seismicity

Faulting processes in the Dallas area are considered inactive. Based on the National Earthquake Information Center (NEIC) earthquake catalog, 43 small magnitude events within a 100-mile radius of Dallas have been recorded in the past 150 years, with all but
one event being recorded as smaller than magnitude 4.0. The anticipated peak ground acceleration in the Dallas area is approximately 0.02g with a probability of exceedance of 10 percent in 50 years (Frankel et al. 2002, ICC 2003). Furthermore, the Dallas-Ft. Worth area is located in Seismic Zone 0 as identified in the Uniform Building Code (UBC 1997), which is the lowest earthquake hazard region in the United States. Seismic design considerations should therefore have no impact on the proposed facilities covered by this report.
3.0 GROUND CHARACTERIZATION AND SITE CONDITIONS

3.1 Engineering Properties of Soil and Rock

Tables summarizing the geotechnical data and parameters obtained in the Terra-Mar and Fugro geotechnical investigations are presented in the following sections. Determination of the appropriate values for use in the design of all facilities for Segment A is the responsibility of the Developer.

3.1.1 Overburden Soils

Soil deposits along Segment A consist primarily of highly plastic residual soils derived from the Austin Chalk and Eagle Ford Shale, cohesionless, non-plastic sandy alluvial soils associated with the Trinity River floodplain or with sandy terrace deposits of the ancestral Trinity River, moderately to highly plastic clayey alluvial soils associated with terrace deposits of the Trinity River and its tributaries, and finally, highly plastic colluvial soils derived from the host rock strata but transported short distances by erosional processes. In addition, due to the developed nature of the existing roadway alignment, some moderately to highly plastic fill soils are likely to be encountered.

For this Project, these overburden soils have been grouped into three classes based on their origin, and on the influence of origin on their characteristics and engineering properties. Based on these factors, the materials are identified as: 1) fill, 2) alluvial soils, or 3) residual soils. In the following paragraphs, the characteristics and pertinent engineering properties are of each described.

3.1.1.1 Fill

The fill soils encountered along Segment A varied considerably in terms of type, consistency and thickness. At some locations, medium stiff clayey fill soils were encountered. While at others, dense granular fills (clayey gravel) or non-plastic fine-grained soils (sилts) were encountered. This variation in soil consistency is likely due to variation in the native soils that were used as borrow sources for these fill materials. The thickness of fill at any specific location obviously depends on the original land configuration at that location, and on the final configuration or facility constructed. For example, at an existing bridge approach for the freeway, the fill could be upwards of 25 feet thick, while along frontage roads, fill thickness could be on the order of 2-3 feet.

Overall, the fill soil samples collected were generally very stiff to hard clays of high plasticity with an average unconfined compressive strength of about 4,900 psf. Penetration resistance as determined via TxDOT’s cone penetrometer test averaged 17 blows per foot, confirming the results of the unconfined compression tests and the conclusion that the clays are generally very stiff to hard (Table 3-1).
3.1.1.2 Alluvial Soils

The sandy alluvial soils are generally classified as sands, clayey sands and gravels (SP, SC, GC) using the Unified Soil Classification System (ASTM D2487). Excavations for below grade roadway sections near the western end of Segment A may encounter these soils, either as seams of limited thickness or as a deep granular deposit that extends down to bedrock. For example, in some locations these soils were found in relatively thin bands about 4 feet thick, surrounded above and below by stiff clay deposits. In contrast, in the westernmost borings, close to the Trinity River floodplain, a granular deposit up to 50 feet thick overlying the Eagle Ford Shale bedrock was encountered.

Based on blow counts, these soils were generally found to exist in a medium dense condition. Moisture contents in the sandy alluvial soils ranged from 16 percent to 21 percent (Table 3-1). Unit weights and strength parameters were not readily obtained, but can be inferred from the penetration resistance and from visual USCS classification (NAVFAC 1986). A single triaxial compressive strength test was performed on a sample of the granular alluvial deposits, from Boring BW3L, at 24.5 feet, and yielded an effective cohesion of 0 psf and an effective friction angle of 33 degrees.

The clayey alluvial deposits that are believed to be remnants of terraces or upland erosional processes are generally identified as moderately to highly plastic clays. Moisture contents of the clayey alluvial soil samples varied considerably, from 2 percent to 34 percent with an average about 21 percent. A limited number of borings encountered clays of medium plasticity (CL), but for the most part these soils are generally high plasticity clays (CH). The liquid limits of these soils varied from about 21 to as high as 79, and their plastic limits varied from 14 to 58 (Table 3-1). The activity indices for these materials averaged 0.74 and indicate very high swell potential (Figure 7). This description is supported by the results of seven swell test of various types performed on clayey alluvial soil samples collected along Segment A. These tests indicated widely varying but significant swell pressures ranging from 0.6 tsf to 4 tsf, and swell volume increases of as little as 0.1 percent to as much as 8 percent.

These clayey alluvial soils were found with consistencies ranging from stiff to hard, with unconfined strength averaging 4,444 psf. Three consolidated undrained triaxial strength tests with pore pressure measurements were performed on samples of this material, resulting in a wide range of effective friction angles (Figure 6). These tests resulted in an effective friction angles ranging from 11 to 19 degrees with effective cohesion ranging from 400 to 600 psf. The plasticity indices from tests near these samples indicated these soils would plot as high plasticity clays (CH), although close to the boundary with moderately plastic clays (Figure 5).

The previous geotechnical exploration programs did not distinguish between the clayey alluvial deposits and alluvial deposits that can also be described as colluvial. Colluvial deposits that originated as weathering products of the underlying shale and limestone bedrock, but which have been redeposited downhill by erosional processes or creep, are likely to retain much of the higher plasticity, including swelling potential, of their parent residual soils.
3.1.1.3 Residual Soils

The residual soils to be encountered on this Project are generally stiff to hard, high plasticity clayey soils developed by weathering of the host Eagle Ford and Austin Groups. In their publications, the SCS describes these soils as having very high to high shrink-swell potential, respectively.

Based on the Project configuration depicted in the Reference Schematic, the majority of the residual soils to be encountered on the Project will consist of residual soils of the Eagle Ford Shale. Specifically, these materials will be encountered west of the contact between the Eagle Ford Shale and the Austin Chalk at approximate Station 155+00.

The residual soils of the Eagle Ford Shale are highly plastic clays, generally stiff to hard, and are classified as CH soils (Table 3-1). Hydrometer grain size and Atterberg limits test data from the Terra-Mar, Inc. (1998) investigation and 1 sample from the Fugro investigation (2004a) were plotted on a diagram commonly used to assess swelling potential (Figure 7). It can be observed that according to this criterion the Eagle Ford Shale residual soil samples would classify as having "very high" swell potential. Relative to strength, triaxial strength tests in the residual soils of the Eagle Ford Shale yielded effective friction angles from 12 to 19 degrees and effective cohesion values of 170 to 710 psf (Figure 6).

Similarly, the residual soils derived from the Austin Chalk are typically very stiff to hard, moderately to highly plastic clays, classified as CL or CH. This description is based on a limited number of plasticity tests (9) and strength tests (3), due mainly to the limited occurrences and limited thickness of residual clay derived from Austin Chalk encountered in the test borings.
<table>
<thead>
<tr>
<th>MATERIAL PROPERTY</th>
<th>Units</th>
<th>Fill Materials</th>
<th>Alluvial Soils (a)</th>
<th>Residual Soils</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Cohesionless</td>
<td>Cohesive</td>
<td>Eagle Ford</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(Min, Max, Count)</td>
<td>(Min, Max, Count)</td>
<td>Shale</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(Min, Max, Count)</td>
<td>(Min, Max, Count)</td>
<td>Austin Chalk</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Mean, Std. Dev.</td>
<td>Mean, Std. Dev.</td>
<td>Mean, Std. Dev.</td>
</tr>
<tr>
<td>Index Tests</td>
<td></td>
<td></td>
<td>(Min, Max, Count)</td>
<td>(Min, Max, Count)</td>
</tr>
<tr>
<td>Natural Moisture Content</td>
<td>%</td>
<td>21, 7 (3, 38, 115)</td>
<td>18, 2 (16, 21, 7)</td>
<td>21, 5 (2, 34, 90)</td>
</tr>
<tr>
<td>Dry Unit Weight</td>
<td>pcf</td>
<td>103, 14 (87, 135, 18)</td>
<td>49, NA (49, 49, 1)</td>
<td>22, 8 (14, 58, 31)</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>%</td>
<td>21, 16 (16, 85, 47)</td>
<td>21, 5 (2, 34, 90)</td>
<td>54, 15 (21, 79, 31)</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>%</td>
<td>20, 5 (12, 32, 47)</td>
<td>17, NA (17, 17, 1)</td>
<td>22, 8 (14, 58, 31)</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>%</td>
<td>31, 12 (4, 56, 47)</td>
<td>32, NA (32, 32, 1)</td>
<td>34, 11 (15, 52, 31)</td>
</tr>
<tr>
<td>Percent Passing No. 200</td>
<td>%</td>
<td>49, 44 (20, 99, 3)</td>
<td>13, 8 (5, 23, 5)</td>
<td>56, 27 (24, 97, 8)</td>
</tr>
<tr>
<td>Activity Index</td>
<td></td>
<td>--</td>
<td>0.74, 0.01 (0.73, 0.74, 2)</td>
<td>0.79, 0.19 (0.59, 1.06, 5)</td>
</tr>
<tr>
<td>Predominant USCS material classification, in order of occurrence</td>
<td>--</td>
<td>CH, CL (rare)</td>
<td>GC, SP, SM, SW, ML</td>
<td>CH, CL</td>
</tr>
<tr>
<td>Strength Tests</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TX-CONE Penetration Resistance, ( N_{\text{tex}} ) (b)</td>
<td>17, 13 (4, 64, 45)</td>
<td>22, 8 (2, 32, 11)</td>
<td>29, 29 (6, 100, 65)</td>
<td>21, 22 (6, 100, 37)</td>
</tr>
<tr>
<td>Standard Penetration Resistance, ( N_{\text{spt}} )</td>
<td>20, 7 (10, 30, 5)</td>
<td>35, 24 (4, 76, 8)</td>
<td>20, 11 (10, 36, 5)</td>
<td>46, NA (46, 46, 1)</td>
</tr>
<tr>
<td>Unconfined Compressive Strength</td>
<td>psf</td>
<td>4873, 3252 (1584, 12384, 14)</td>
<td>--</td>
<td>4444, 2943 (403, 11664, 30)</td>
</tr>
<tr>
<td>UU Compressive Strength</td>
<td>psf</td>
<td>--</td>
<td>--</td>
<td>5733, 2261 (3150, 7352, 3)</td>
</tr>
<tr>
<td>Triaxial Strength, Effective Stress (peak)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \phi' ) (deg)</td>
<td></td>
<td>33, NA (33, 33, 1)</td>
<td>16, 5 (11, 19, 3)</td>
<td>14, 3 (12, 19, 5)</td>
</tr>
<tr>
<td>( c' ) (psf)</td>
<td></td>
<td>0.0, NA (0.0, 0.0, 1)</td>
<td>530, 113 (400, 600, 3)</td>
<td>342, 212 (170, 710, 5)</td>
</tr>
<tr>
<td>Swell Tests</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Simple Swell (% of ( H_0 ))</td>
<td>%</td>
<td>--</td>
<td>--</td>
<td>2.8, 4.5 (0.1, 8.0, 3)</td>
</tr>
<tr>
<td>ASTM D4546 1D-Swell Test, Method A</td>
<td>%</td>
<td>--</td>
<td>--</td>
<td>4.1, 4.2 (1.1, 7.0, 2)</td>
</tr>
<tr>
<td>ASTM D4546 Methods A and C, Swell Pressure</td>
<td>tsf</td>
<td>--</td>
<td>--</td>
<td>1.7, 1.6 (0.6, 4.0, 4)</td>
</tr>
</tbody>
</table>

---

(a) Includes colluvial soils.

(b) In the Texas Cone Penetrometer (TCP) test, the penetrometer is driven for 100 blows, recording the depth of penetration for the first and second 50 blows. In this table, tests in which the penetrometer was driven less than 12 inches were assigned a value of 100.
3.1.2 Bedrock

3.1.2.1 Austin Chalk

The cut-and-cover tunnels and U-wall sections at the east end of Segment A, as well as the mined tunnels and associated underground structures will be excavated entirely within the limestone of the Austin Chalk. The Austin Chalk is classified as a weak rock (ISRM 1981), and in terms of discontinuities, the Austin Chalk is considered a massive, moderately jointed rock (Proctor and White 1977). Discontinuities are typically spaced on the order of 10 to 300 feet apart, with an average spacing of 60 feet (Collins et al. 1992, DART 1991). Some of the jointing occurs in swarms and is delineated with a tight joint spacing that is visible over some distance. Reported spacing between these features varies from 500 to 21,000 feet apart (Mace et al. 1995).

Fugro Consultants LP collected site-specific data regarding the orientation of discontinuities in the Phase 2 geotechnical investigation. During the investigation, Schlumberger Wire Line Services was subcontracted to perform downhole surveys of the eight angled borings using resistivity-based micro-imaging methods. When the data were further analyzed using stereonet analysis, three reasonably well defined discontinuity orientations of N65°E, N24°E and N75°W were identified. These orientations are fairly consistent with the orientations identified through the analysis of historical reference data presented in Chapter 2. There are variations in the orientations of the data ranging from 10° to 22° from the historical data analysis, but overall, the data collected appear consistent. Further description of the downhole survey process and the discontinuity data gathered are presented in the Phase 2 GDR (Fugro 2005).

Joints and fractures in the Austin Chalk that were identified in the test borings and during the downhole logging, are also summarized graphically on Figure 10. It can be seen from the figure that the density of fractures can vary significantly over a short distance and from one boring to the next. In addition to these structural features, minor inclusions of several types were noted on many the borings in the chalk. Roughly one fourth of the borings in the chalk encountered pyrite, generally as nodules ranging in size from ¼ inch to one inch. These were not confined to joints and fractures or to proximity to a bentonite bed, but appeared instead to be dispersed throughout the rock mass. Less frequent, calcareous nodules and laminae up to 2 inches were noted in the Austin Chalk, and a large septarian concretion was identified in a shear zone.

The thickness of weathered Austin Chalk was found to vary along Segment A, with the thickness in the borings ranging from 1 foot to 24 feet, and averaging approximately 8 feet. Core recovery in weathered Austin Chalk from the geotechnical investigations conducted for this Project was generally excellent, ranging from 28 percent to 100 percent. The average core recovery was 89 percent, and approximately 68 percent of the individual core runs are reported to have core recovery of 90 percent or more. Rock Quality Designation (RQD) values ranged from 0 to 100 percent and averaged 69 percent. Approximately 38 percent of the individual core runs were reported to have an RQD of 90 percent or more. According to rock classification methods using these parameters, the quality of the weathered Austin Chalk would be classified as “fair” (Deere and Deere 1988).

Core recovery in fresh Austin Chalk from the geotechnical investigations conducted for this Project was generally excellent, ranging from 0 percent to 100 percent. The
average core recovery was 98 percent, and approximately 96 percent of the individual core runs are reported to have core recovery of 90 percent or more. RQD values ranged from 0 to 100 percent and averaged 95 percent. Approximately 87 percent of the individual core runs were reported to have an RQD of 90 percent or more. According to rock classification methods using these parameters, the Austin Chalk would be classified as “excellent”.

The results of laboratory tests performed on samples collected during the geotechnical investigations are presented in Table 3-2. The results indicate that the fresh Austin Chalk that was tested has an average dry unit weight of 125 pcf, and average moisture content of 11 percent. Unconfined compressive strength tests performed on the Austin Chalk indicated strengths ranging from 615 psi to 4,159 psi, with an average of 2,468 psi and a standard deviation of 713 psi. This average unconfined strength is within the range, but on the high end of average strength values determined in previous tunneling projects in the Austin Chalk.

Based on these strength values, an effort to investigate the range of unconfined compressive strength with depth and location along Segment A was conducted. In general, all samples obtained for testing were selected from the borings based on the depth of the tunnel envelope at the time the borings were drilled. In Figure 8, samples with strength values less than 2,400 psi were plotted with an open circle, and tests with strength values greater than 2,400 psi were plotted with a solid circle. The figure does not clearly indicate a relationship between unconfined compressive strength and depth or location along Segment A. Rather it suggests that there is little difference in unconfined strength between the lower and middle Austin Chalk members. In contrast, the differences in argillaceous content between the lower chalk member and the middle marl member can be seen in Figure 9. The figure is a graphical representation of data gathered in the laboratory, by separate visual examination and logging, of the argillaceous content of each foot of Austin Chalk core from the Phase I and Phase II borings. The figure confirms that the argillaceous content of the lower chalk member (excluding the Fishbed Conglomerate) is lower than in the middle marl member, and as expected, is significantly lower than the argillaceous content of the underlying Eagle Ford Shale.

During the Phase 2 geotechnical investigation, specific laboratory tests including physical property tests and petrographic analysis used in the determination of performance prediction estimates for mechanical excavation, and estimates of cutter wear were also performed. The results of these tests are summarized in Table 3-2.

3.1.2.1.1 Fishbed Conglomerate

The Fishbed Conglomerate lies at the base of the Austin Chalk, and based on the mined tunnel alignment described in the Reference Schematic it should not be encountered during mined tunnel construction. It will however be encountered at the west end of the Project in the cut-and-cover tunnel and/or U-wall sections.

As described in Chapter 2, the Fishbed is an arenaceous limestone with abundant marine fossil debris and reworked materials derived from the underlying Eagle Ford Shale. The boring logs indicate an increase in fracture frequency, and lower values of RQD through the Fishbed, compared with the balance of the Austin Chalk. The impacts
of the Fishbed on design and construction of the cut-and-cover tunnel and U-wall sections are described in Chapters 6 and 8, respectively.
Table 3-2  Austin Chalk Engineering Properties

<table>
<thead>
<tr>
<th>MATERIAL PROPERTY</th>
<th>Units</th>
<th>Weathered Rock</th>
<th>Fresh Rock</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean, Std. Dev.</td>
<td>Mean, Std. Dev.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(Min, Max, Count)</td>
<td>(Min, Max, Count)</td>
</tr>
<tr>
<td><strong>Index Tests</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Natural Moisture Content</td>
<td>%</td>
<td>18, 6 (9, 37, 38)</td>
<td>11, 4 (1, 22, 478)</td>
</tr>
<tr>
<td>Dry Unit Weight</td>
<td>lb/cu ft</td>
<td>117, 10 (98, 135, 14)</td>
<td>125, 6 (97, 143, 382)</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>%</td>
<td>44, 10 (31, 58, 11)</td>
<td>--</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>%</td>
<td>20, 3 (15, 24, 11)</td>
<td>--</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>%</td>
<td>24, 7 (15, 36, 11)</td>
<td>--</td>
</tr>
<tr>
<td>USCS Classification of disaggregated rock</td>
<td></td>
<td>CH</td>
<td>--</td>
</tr>
<tr>
<td><strong>Strength/Consistency Tests</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TX-CONE Penetration Resistance, $N_{tip}^{(a)}$</td>
<td>88, 29 (7, 100, 41)</td>
<td>100, NA (100, 100, 15)</td>
<td></td>
</tr>
<tr>
<td>Unconfined Compressive Strength</td>
<td>psi</td>
<td>1142, 1185 (13, 3253, 13)</td>
<td>2468, 713 (615, 4159, 216)</td>
</tr>
<tr>
<td>UU Compressive Strength</td>
<td>psi</td>
<td>--</td>
<td>4205, 896 (2126, 6048, 26)</td>
</tr>
<tr>
<td>Modulus of Elasticity, $E \times 10^6$</td>
<td>psi</td>
<td>0.2, 0.2 (0.06, 0.5, 6)</td>
<td>0.47, 0.29 (0.03, 1.46, 146)</td>
</tr>
<tr>
<td>Poisson’s Ratio, $\nu$</td>
<td>--</td>
<td>0.23, 0.09 (0.15, 0.40, 8)</td>
<td></td>
</tr>
<tr>
<td>Axial Point Load</td>
<td>psi</td>
<td>379, 133 (241, 505, 3)</td>
<td>462, 122 (67, 780, 268)</td>
</tr>
<tr>
<td>Diametric Point Load</td>
<td>psi</td>
<td>377, NA (377, 377, 1)</td>
<td>274, 121 (26, 656, 131)</td>
</tr>
<tr>
<td>Brazilian Tensile Strength</td>
<td>psi</td>
<td>--</td>
<td>234, 55 (94, 350, 58)</td>
</tr>
<tr>
<td><strong>Mechanical Excavation Performance</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cerchar Abrasivity Index</td>
<td>--</td>
<td>0.5, 0.1 (0.4, 0.6, 11)</td>
<td></td>
</tr>
<tr>
<td>Taber Abrasion, $A_r$</td>
<td>g$^{-1}$</td>
<td>--</td>
<td>55, 50 (6, 115, 6)</td>
</tr>
<tr>
<td>Taber Hardness, $H_a$</td>
<td>g$^{-1}$</td>
<td>--</td>
<td>0.6, 0.4 (0.2, 1.2, 5)</td>
</tr>
<tr>
<td>Shore Hardness, $H_s$</td>
<td>--</td>
<td>--</td>
<td>14, 2 (12, 20, 11)</td>
</tr>
<tr>
<td>Punch Penetration</td>
<td>ksi</td>
<td>--</td>
<td>30, 12 (15, 43, 7)</td>
</tr>
<tr>
<td><strong>Slaking</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Immersed cage$^{(b)}$</td>
<td>200 rev</td>
<td>--</td>
<td>high (med, very high, 10)</td>
</tr>
<tr>
<td><strong>Hydraulic Conductivity</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Field Packer Tests, $k$</td>
<td>cm/sec</td>
<td>--</td>
<td>$1.8 \times 10^6$, 6.2 $\times 10^6$ (0, 2.6 $\times 10^5$, 17)</td>
</tr>
</tbody>
</table>

---

(a) In the Texas Cone Penetrometer (TCP) test, the penetrometer is driven for 100 blows, recording the depth of penetration for the first and second 50 blows. In this table, tests in which the penetrometer was driven less than 12 inches were assigned a value of 100.

---

(b) In the Immersed cage test, the sample is subjected to 200 revolutions. The slaking classification is determined based on visual observation.
b) Slake durability tests were performed on ½"-thick rock discs of appropriate weight, in a mesh cylinder immersed in a water bath and rotated 200 revolutions, measuring retained weight, similar to ASTM D4644.

3.1.2.1.2 Bentonite Beds in the Austin Chalk

The 4 to 14 inch thick bentonite layer, regionally known as the Bentonite Marker Bed, near the contact between the middle and lower Austin members, differs significantly from the host Austin Chalk. The Bentonite Marker Bed is highly plastic, and is classified as a CH material. The average natural water content of this material was found to be 36 percent with an average dry unit weight of approximately 83 pcf (Table 3-3). This material is weaker than the surrounding limestone and can deteriorate or soften, and expand upon exposure to air or groundwater. Particle size and Atterberg limits tests performed on a single sample of the Bentonite Marker Bed indicates that approximately 36 percent of the sample had a particle size less than 2 microns, a liquid limit of 80 percent, a plasticity index of 53 and an activity index of 1.47 (Table 3-3 and Figure 7).

Triaxial shear strength testing of Bentonite Marker Bed samples indicated an effective cohesion of between 500 and 2,500 psf and a peak effective friction angle between 18 and 35 degrees. Drained direct shear testing indicated very low effective cohesion values of 1.1 to 1.8 psf, but peak effective friction angles of 35 to 48 degrees. In general, the results of shear strength testing of the Bentonite Marker Bed on this Project compared fairly well with data from the SSC, where an average peak effective cohesion of 720 psf and an average peak effective friction angle of 31 degrees were recorded (The PB/MK Team 1992). In all cases, the results presented above are based on a limited number of tests and may be misleading. Historically, peak effective strengths for bentonites are considerably lower (Font 1979), and the higher values that were measured may represent a coarser-grained portion of the material that could be sampled. Where the shear strength of the Bentonite Marker Bed or of other bentonite beds is a critical design or construction consideration, additional testing and characterization is warranted.

Five ASTM D-4546 Type C swell tests were performed on bentonite samples from the Phase 2 investigation. Laboratory tests to establish swell pressure are often problematic due to the inevitable sample disturbance associated with sampling via rock coring techniques. Although this material is generally viewed as having very high swell potential, the swell pressure values inferred from those tests and reported in Table 3-3 should be viewed as rough approximations. These data indicate this material can exert swell pressures of at least 8 tsf and possibly as high as 30 tsf.

In addition to the regionally defined Bentonite Marker Bed, eight additional continuous to nearly continuous bentonite beds were identified during analysis of the field investigation data (Table 3-4). To evaluate the continuity of these additional seams across the borings, a numeric bentonite seam designation system was developed in which the seam designation numbers are labeled with 1 being the deepest and 9 being the shallowest. The Bentonite Marker Bed is designated as 5 and is the thickest of the beds.

Each of these bentonitic beds has a relatively uniform thickness and vertical offset from the regionally defined Bentonite Marker Bed. These relative consistencies were also
helpful in evaluating the presence of discontinuities in the rock mass along the Segment A alignment.

Table 3-3  Bentonite Marker Bed Engineering Properties

<table>
<thead>
<tr>
<th>MATERIAL PROPERTY</th>
<th>Units</th>
<th>Bentonite Marker Bed</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean, Std. Dev. (Min, Max, Count)</td>
</tr>
<tr>
<td><strong>Index Tests</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Natural Moisture Content</td>
<td>%</td>
<td>36, 7 (22, 42, 14)</td>
</tr>
<tr>
<td>Dry Unit Weight</td>
<td>lb/cu ft</td>
<td>83, 7 (75, 94, 9)</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>%</td>
<td>80, NA (80, 80, 1)</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>%</td>
<td>27, NA (27, 27, 1)</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>%</td>
<td>53, NA (53, 53, 1)</td>
</tr>
<tr>
<td>Percent Passing No. 200</td>
<td>%</td>
<td>97, NA (97, 97, 1)</td>
</tr>
<tr>
<td>Percent finer than 2 micron</td>
<td>%</td>
<td>36, NA (36, 36, 1)</td>
</tr>
<tr>
<td>Activity Index</td>
<td></td>
<td>1.47, NA (1.47, 1.47, 1)</td>
</tr>
<tr>
<td><strong>USCS Classification of disaggregated rock</strong></td>
<td></td>
<td>CH</td>
</tr>
<tr>
<td><strong>Strength/Consistency Tests</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Triaxial Strength, Effective Stress (peak)</td>
<td>deg</td>
<td>24, 10 (18, 35, 3)</td>
</tr>
<tr>
<td>$\phi'$</td>
<td>deg</td>
<td>1467, 1002 (500, 2000, 3)</td>
</tr>
<tr>
<td>$c'$</td>
<td>psf</td>
<td></td>
</tr>
<tr>
<td>Direct Shear Strength, Effective Stress (peak)</td>
<td>deg</td>
<td>42, 10 (35, 48, 2)</td>
</tr>
<tr>
<td>$\phi'$</td>
<td>deg</td>
<td>1.5, 0.5 (1.1, 1.8, 2)</td>
</tr>
<tr>
<td>$c'$</td>
<td>psf</td>
<td></td>
</tr>
<tr>
<td><strong>Swell Tests</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ASTM D4546 Method C, Swell Pressure</td>
<td>tsf</td>
<td>14, 9 (8, 30, 5)</td>
</tr>
</tbody>
</table>

Table 3-4  Bentonite Beds Logged along Mined Tunnel Alignment

<table>
<thead>
<tr>
<th>Individual Bentonite Beds</th>
<th>Vertical Offset from Bentonite Marker Bed (ft)</th>
<th>Approximate Thickness (in)</th>
<th>Consistency</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-70</td>
<td>2-4</td>
<td>Soft to Very Soft</td>
</tr>
<tr>
<td>2</td>
<td>-49</td>
<td>4-7</td>
<td>Soft to Very Soft</td>
</tr>
<tr>
<td>3</td>
<td>-23</td>
<td>&lt;1</td>
<td>Soft to Very Soft</td>
</tr>
<tr>
<td>4</td>
<td>-18</td>
<td>0.5-1.5</td>
<td>Soft to Very Soft</td>
</tr>
<tr>
<td>5 (a)</td>
<td>--</td>
<td>4-14</td>
<td>Soft to Very Soft</td>
</tr>
<tr>
<td>6</td>
<td>+11.5</td>
<td>2-3</td>
<td>Soft to Very Soft</td>
</tr>
<tr>
<td>7</td>
<td>+28</td>
<td>2-3</td>
<td>Soft to Very Soft</td>
</tr>
<tr>
<td>8</td>
<td>+35</td>
<td>3-9.5</td>
<td>Soft to Very Soft</td>
</tr>
<tr>
<td>9 (b)</td>
<td>+50</td>
<td>3-7</td>
<td>Soft to Very Soft</td>
</tr>
</tbody>
</table>

a) Bentonite Marker Bed
b) Referred to as “Upper Bentonite Marker Bed” in the Phase 1 GBR (Fugro 2004b)
3.1.2.2 Eagle Ford Shale

Portions of the cut-and-cover tunnel and U-wall sections will be excavated within the Eagle Ford Shale. The Eagle Ford Shale is classified as a very weak rock (ISRM 1981) and upon wetting is known to swell, slake and lose strength rapidly. In terms of discontinuities, the Eagle Ford Shale is considered massive, moderately jointed rock (Proctor and White 1977), and except in isolated cases, the joints are generally widely spaced and tight. Of the joints and fractures in the Eagle Ford Shale that were identified in the test borings approximately half were classified as slickensided.

In addition to the discontinuities that were identified in the logs, several other types of minor features were noted as well. For example, near the western end of the Project, where the roadway will be stratigraphically lower in the Eagle Ford Shale, many borings encountered gypsum crystals in joints in the weathered shale. Further to the east, calcite nodules and concretions from 1/8 inch to 3 inches thick were encountered, along with limestone interbeds of less than an inch to as much as one foot in thickness. Occasional pyrite nodules were observed, and in one instance a large (approximately 6 inch) septarian concretion was identified. Similar concretions up to 3 feet in diameter and ranging in shape from plate-like to elongated cigar-like structures were identified in excavations at the SSC, and were found to be present in zones or horizons within the shale (Shannon & Wilson 1993).

Core recovery in the Eagle Ford Shale during the Phase 1 and Phase 2 geotechnical investigations was generally excellent, ranging from 0 percent to 100 percent. The average core recovery was equal to 93 percent. Approximately 82 percent of the individual core runs are reported to have core recovery of 90 percent or more. Rock Quality Designation (RQD) values ranged from 0 to 100 percent and averaged 90 percent. Approximately 71 percent of the individual core runs are reported to have an RQD of 90 or more. According to rock classification methods using these parameters, the Eagle Ford Shale would be classified as “good to excellent”.

The results of laboratory tests performed on samples collected during the geotechnical investigations are presented in Table 3-5. The results indicate that the weathered Eagle Ford Shale that was tested has an average dry unit weight of 105 pcf, and average moisture content of 23 percent. The liquid limits of the weathered shale ranged from 45 to 96, and the plasticity indices ranged from 26 to 63.

For the fresh Eagle Ford Shale samples tested for this Project, an average dry unit weight of 117 pcf and an average moisture content of 17 percent were determined. Four tests were run to determine liquid limit and plasticity index in the fresh Eagle Ford Shale. In these tests, the liquid limit of the shale ranged from 58 to 59, and the plasticity index ranged from 34 to 36. These values appear to be on the low end of reported literature values which suggest that the liquid limit for the Eagle Ford Shale can typically range from 60 to 80 and the plasticity index can typically range from 34 to 48 (Font 1979). Unconfined compressive strength tests performed on the fresh Eagle Ford Shale indicated strengths ranging from 29 psi to 700 psi, with an average of 190 psi. Young’s modulus and Poisson’s ratio was determined in three of the unconfined compressive strength tests and average values of 40,000 psi and 0.25 were determined, respectively.
For this Project, a total of four swell tests of various types were performed on samples of weathered Eagle Ford Shale obtained from undisturbed (Shelby tube) samples. These indicated swell pressures of 3 to 5 times the field effective stress, and significant volume increases up to 13 percent. No swell tests were performed on samples of fresh Eagle Ford Shale for this Project due to the practical difficulties associated with trimming the fresh shale from NX cores to fit the testing apparatus, with consequent damage to the sample that can render the results questionable. However, swell test data from the SSC project indicated that while swell pressures in the range of the field effective stress were often measured, swell pressures of two to three times the field effective stress were also found to develop (Lai 1997). The swelling characteristics of the Eagle Ford Shale are due in large part to the high montmorillonite content of the shale. Data from the SSC indicated that the fines content of the Eagle Ford ranges from 65 to 100 percent, with an average percent passing the No. 200 sieve of 97 percent (Lai 1997). Of this percentage, 14 to 74 percent of the material is montmorillonite (average 49 percent).

Based on the roadway geometry presented in the Reference Schematic, essentially all excavation for the cut-and-cover tunnel and U-wall sections located in the fresh Eagle Ford Shale will extend at least 10 feet into the shale below the Austin Chalk/Eagle Ford Shale contact. As presented earlier, the Eagle Ford Shale underlies the Austin Chalk along an unconformable contact, with erosional periods occurring in the Eagle Ford Shale prior to deposition of the Austin Chalk. Results of laboratory and field tests performed for the design of the SSC project indicate this period of erosion and exposure may have had a significant impact on the properties of the Eagle Ford Shale within 10 ft of the contact (Hasek 1993). The data from the SSC indicated that the upper 10 ft is generally weaker and has a lower Young's modulus than the formation as a whole, it is also less cemented and has a lower slake durability index. The upper 10 ft generally has a higher moisture content, higher clay percentage; higher liquid limit and higher plasticity index as well (Teetes 2001).
Table 3-5  Eagle Ford Shale Engineering Properties

<table>
<thead>
<tr>
<th>MATERIAL PROPERTY</th>
<th>Units</th>
<th>Weathered Rock</th>
<th>Fresh Rock</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean, Std. Dev. (Min, Max, Count)</td>
<td>Mean, Std. Dev. (Min, Max, Count)</td>
</tr>
<tr>
<td>Index Tests</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Natural Moisture Content</td>
<td>%</td>
<td>23, 6 (11, 36, 70)</td>
<td>17, 3 (8, 32, 55)</td>
</tr>
<tr>
<td>Dry Unit Weight</td>
<td>pcf</td>
<td>105, 10 (89, 127, 40)</td>
<td>117, 7 (94, 137, 48)</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>%</td>
<td>67, 12 (45, 96, 21)</td>
<td>58, 1 (58, 59, 4)</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>%</td>
<td>24, 3 (18, 33, 21)</td>
<td>23, 1 (22, 24, 4)</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>%</td>
<td>43, 9 (26, 63, 21)</td>
<td>35, 1 (34, 36, 4)</td>
</tr>
<tr>
<td>USCS Classification of disaggregated rock</td>
<td></td>
<td>CH</td>
<td>--</td>
</tr>
<tr>
<td>Strength/Consistency Tests</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TX-CONE Penetration Resistance, $N_{tex}$ (a)</td>
<td>69, 35 (11, 100, 79)</td>
<td>100, NA (100, 100, 19)</td>
<td></td>
</tr>
<tr>
<td>Unconfined Compressive Strength</td>
<td>psi</td>
<td>90, 115 (14, 594, 32)</td>
<td>190, 107 (29, 700, 34)</td>
</tr>
<tr>
<td>UU Compressive Strength</td>
<td>psi</td>
<td>75, 54 (31, 135, 3)</td>
<td>203, 56 (128, 321, 12)</td>
</tr>
<tr>
<td>Modulus of Elasticity, $E \times 10^6$</td>
<td>psi</td>
<td>0.03, 0.01 (0.02, 0.04, 4)</td>
<td>0.04, 0.02 (0.01, 0.08, 21)</td>
</tr>
<tr>
<td>Poisson’s Ratio, $\nu$</td>
<td></td>
<td>--</td>
<td>0.25, 0.06 (0.19, 0.30, 3)</td>
</tr>
<tr>
<td>Axial Point Load</td>
<td>psi</td>
<td>--</td>
<td>59, 9 (38, 64, 7)</td>
</tr>
<tr>
<td>Triaxial Strength, Effective Stress (peak)</td>
<td></td>
<td>37, NA (37, 37, 1)</td>
<td>--</td>
</tr>
<tr>
<td>$\phi'$</td>
<td>deg</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$c'$</td>
<td>psf</td>
<td>200, NA (200, 200, 1)</td>
<td>--</td>
</tr>
<tr>
<td>Swell Tests</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Simple Swell (% of $H_0$)</td>
<td>%</td>
<td>9.5, 4.9 (6.0, 13.0, 2)</td>
<td>--</td>
</tr>
<tr>
<td>ASTM D4546 1D-Swell Test, Method A</td>
<td>%</td>
<td>7.8, NA (7.8, 7.8, 1)</td>
<td>--</td>
</tr>
<tr>
<td>ASTM D4546 Methods A and C, Swell Pressure</td>
<td>tsf</td>
<td>2.7, 1.0 (2.0, 3.4, 2)</td>
<td>--</td>
</tr>
<tr>
<td>Slaking</td>
<td></td>
<td></td>
<td>moderate</td>
</tr>
<tr>
<td>Jar Slaking (USBR 1998)</td>
<td></td>
<td></td>
<td>(none, moderate, 11)</td>
</tr>
<tr>
<td>Hydraulic Conductivity</td>
<td>cm/sec</td>
<td>--</td>
<td>$2.2 \times 10^{-7}$, NA (2.2x10^{-7}, 2.2x10^{-7}, 1)</td>
</tr>
</tbody>
</table>

(a) In the Texas Cone Penetrometer (TCP) test, the penetrometer is driven for 100 blows, recording the depth of penetration for the first and second 50 blows. In this table, tests in which the penetrometer was driven less than 12 inches were assigned a $N_{tex}$ value of 100.
3.2 Hydrogeologic Setting

3.2.1 Hydraulic Conductivity

Field hydraulic conductivity testing was performed during the Phase 1 and Phase 2 geotechnical investigations in both the fresh Austin Chalk and fresh Eagle Ford Shale (Table 3-6). No field or laboratory hydraulic conductivity tests were performed for the overburden soils or weathered rock. Therefore, values of hydraulic conductivity for these materials, reported in Table 3-6, were obtained from the literature.

The packer tests in the Austin Chalk where flow was measured indicate a fairly narrow range in hydraulic conductivities of $1.4 \times 10^{-7}$ to $1.1 \times 10^{-6}$ cm/sec, with an average hydraulic conductivity of $4.1 \times 10^{-7}$ cm/sec. The single packer test performed in the Eagle Ford Shale indicated a hydraulic conductivity of $2.2 \times 10^{-7}$. These low hydraulic conductivity values are consistent with other packer testing performed for the SSC and for the Cole Park Detention Vault project.

The values presented in Table 3-6 can be used to estimate steady state flow rates based on the overall rock mass permeability and including steady state flow through fracture systems. However, these values cannot be used to estimate flush flows from previously undrained or newly recharged fracture zones. Flush flow from fracture systems, as discussed in Chapters 5 and 6, must also be addressed by the Developer in the design.

Table 3-6  Summary of Field Hydraulic Conductivity Testing

<table>
<thead>
<tr>
<th>Geologic Stratum</th>
<th>No. Tests (a)</th>
<th>(k_{\text{min}}) (cm/sec)</th>
<th>(k_{\text{mean}}) (cm/sec)</th>
<th>(k_{\text{max}}) (cm/sec)</th>
<th>Range of Hydraulic Conductivity Values, (k), from Literature (cm/sec)</th>
<th>Literature Reference (c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clayey Soils</td>
<td>--</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>$1.0 \times 10^{-9}$ to $1.0 \times 10^{-3}$</td>
<td>1, 2</td>
</tr>
<tr>
<td>Sandy Soils</td>
<td>--</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>$1.0 \times 10^{-6}$ to $1.0 \times 10^{-1}$</td>
<td>1, 2</td>
</tr>
<tr>
<td>Weathered Austin Chalk</td>
<td>--</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>$1.6 \times 10^{-6}$ to $3.4 \times 10^{-3}$</td>
<td>3, 4</td>
</tr>
<tr>
<td>Fresh Austin Chalk (b)</td>
<td>13</td>
<td>$1.4 \times 10^{-7}$</td>
<td>$4.1 \times 10^{-7}$</td>
<td>$1.1 \times 10^{-6}$</td>
<td>$5.0 \times 10^{-9}$ to $7.8 \times 10^{-5}$</td>
<td>5, 6</td>
</tr>
<tr>
<td>Weathered Eagle Ford Shale</td>
<td>--</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>$1.0 \times 10^{-11}$ to $1.0 \times 10^{-3}$</td>
<td>1</td>
</tr>
<tr>
<td>Fresh Eagle Ford Shale</td>
<td>1</td>
<td>-</td>
<td>$2.2 \times 10^{-7}$</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

a) Packer tests presented in the Phase 1 and Phase 2 Geotechnical Data Reports.
b) Only tests with measurable flow were included in this summary.
3.2.2 Groundwater Elevations

Piezometers in the Austin Chalk, installed during the Phase 1 and Phase 2 geotechnical investigations were observed over an extended period of time (up to 18 months). The data obtained from the most recent monitoring event, on April 1, 2005, is presented in Table 3-7. Based on these observations, the groundwater elevation in the 34 open standpipe and vibrating wire piezometers was generally found to exist from 2 to 20 feet below existing grade. The groundwater level on one day represents only a snapshot record of the groundwater levels on that date, and the Developer should consider the complete record of piezometric readings presented in the Phase 1 and Phase 2 GDRs (Fugro 2004a and 2005) in determining groundwater elevations for design.

While some of the instruments revealed fairly consistent readings, some indicated seasonal fluctuations in piezometric level as much as 25 ft. In one instance, the open standpipe installed in Boring BE1L1 indicated a drop of 18 feet over a 4-month period, encompassing March through early July 2004. It then recovered in the seven months from August 2004 through March 2005, and appeared to peak once again in March 2005. Similar behavior was observed in Boring BE7L, approximately one-half mile to the east of Boring BE1L1, where the fluctuation in the standpipe was confirmed by simultaneous fluctuation in a vibrating wire piezometer also installed in that boring. The magnitude of the fluctuation described by the vibrating wire piezometer was approximately double that observed in the standpipe, but the dates of the peaks and valleys compared reasonably well. These fluctuations are likely related, at least in part, to seasonal changes in precipitation, but may also be an indication of areas where the hydraulic conductivity of the Austin Chalk is generally higher and suggesting that there may be zones in fresh Austin Chalk of locally higher fracture density and correspondingly higher permeability, perhaps connected to localized recharge zones.

In addition, as of April 2005, 4 of the 19 vibrating wire piezometers were indicating groundwater elevations above existing grade. Groundwater levels, varying from 5 feet to as much as 37 feet above grade were identified at those locations. All four of the apparent artesian conditions occur in vibrating wire piezometers in the Austin Chalk, and all four occur below relatively mild topographic highs along Segment A where the topography drops off either westward to the Trinity River Valley (Boring 2T10L) or eastward to the White Rock Creek Valley (Borings T30L, 2T31R and 2T34L). The maximum surface elevations through these topographic highs are barely above, and in one case actually below, the groundwater elevations indicated in the piezometers. Consequently, these topographic conditions do not appear to provide the recharge capacity capable of explaining the readings in these instruments.

An attempt was also made to explain these artesian pressures in the context of fracture systems and/or the presence of bentonite beds which could possibly act as aquitards or aquicludes in the Austin Chalk. Three of the piezometers were completed at depths between the two thickest and most continuous bentonite beds, and one other instrument exhibiting apparent artesian pressure (Boring 2T10L) was finished below the regionally defined Bentonite Marker Bed, approximately halfway between the marker bed and the underlying Eagle Ford Shale. However, once again, in each of these locations the surrounding terrain does not exhibit enough relief to induce such artesian pressures.
A subsequent theory involving the swelling of bentonitic layers within the Austin Chalk inducing an overstated pore pressure response was also investigated. In the final analysis, however, insufficient data was available to strongly support this premise.

In view of the unusual nature (considering the topography) of the above findings, Fugro Consultants LP was asked to review the collected data and the calibration of all instruments that were exhibiting pressures higher than existing grade. In response, they conducted a comprehensive review of the data and reported finding no errors in the instrument calibrations, data collection or reporting.

The Developer is encouraged to continue reading the vibrating wire piezometers and standpipe piezometers to better define the existing phreatic conditions, including the apparent seasonal fluctuations and the suspect artesian condition described above.
Table 3-7. Standpipe and Vibrating Wire Piezometer Data (April 1, 2005)

<table>
<thead>
<tr>
<th>Boring</th>
<th>Station, Offset (ft)</th>
<th>Ground Surface Elevation (ft)</th>
<th>Type</th>
<th>Interval (SP) or Tip Elev. (VW) (ft)</th>
<th>Stratum</th>
<th>Groundwater Data</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Elevation</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Depth</td>
</tr>
</tbody>
</table>

a) Negative (-) Depth indicates height above grade.

Stratum Legend: ALLUV – Alluvial soil, WEFS – Weathered Eagle Ford Shale  
EFS – Eagle Ford Shale, AC – Austin Chalk

Station/Offset: 80+99(E), 110 R - Station (EB HOV/Toll Baseline), Offset Right or Left  
208+16(C), 139 R - Station (Existing IH-635 Centerline), Offset Right or Left  
84+87(W), 39 L - Station (WB HOV/Toll Baseline), Offset Right or Left
3.3 In Situ Stress Conditions

The vertical stresses at locations along Segment A can be determined based on the unit weights of the respective soil and rock stratigraphic layers at those locations and knowledge of the groundwater regimes at those locations. The corresponding horizontal stresses are more difficult to determine, with the magnitude and orientation of these locked-in stresses being the result of several potential processes including historic deposition and erosion cycles (i.e., loading and unloading of sediments), tectonic forces, and other processes such as cementation.

No testing was performed during the feasibility study or the Phase 1 and Phase 2 geotechnical investigations for this Project. Thus at this stage of Project development, the Developer must rely on data from previous projects. Horizontal stresses determined by hydraulic fracturing for the underground excavations at the SSC were found to be on the order of 2.1 times the vertical stress for the Austin Chalk and 1.5 for the Eagle Ford Shale (Kim 1991).

3.4 Environmental Considerations

In July 2002 TxDOT conducted a preliminary assessment to determine the potential for environmental degradation in the Project corridor due to the presence of regulated facilities that may generate, transport, store, and/or dispose of hazardous materials and substances. This investigation was conducted pursuant to the Comprehensive Environmental Response, Compensation and Liability Act (CERCLA) and the Resource Conservation and Recovery Act (RCRA). It was documented in a TxDOT report entitled, “Environmental Assessment – Interstate Highway (IH) 635 From: Luna Road to US 75 Dallas County” (TxDOT 2004).

The report concluded that the potential for environmental degradation to shallow groundwater within the Project limits may be present. Various businesses within the Project area use or have used hazardous materials and substances as part of their operations. Some of these businesses own or have owned underground storage tanks and/or generate some quantity of hazardous wastes, primarily related to fueling and servicing automobiles. While the report mentioned that no stressed vegetation or other surficial signs of environmental contamination were observed within the Project limits, further assessment and investigation if needed would be completed later in Project development. Provisions for addressing potential or confirmed contaminated soils or groundwater must be developed consistent with the requirements of the Contract Documents and applicable laws.

3.5 Geologic Conditions along Segment A

To facilitate description of subsurface conditions, Segment A has been divided into sections of generally similar existing geologic conditions to be encountered during construction (Table 3-8). Typically the transition from one section of similar conditions to the next is gradual and may span several hundred feet. The soil units encountered within each of the sections are generally consistent, but the thicknesses of each unit within a section may vary by more than 20 feet. In sections where there is evidence of a
gradual transition, the center of the transition was selected as the limit of a particular section.

The station locations identifying the limits of each section must be considered as approximate. Furthermore, the defined limits are expressed by stationing along the Project alignment, which includes both the eastbound and westbound HOV/Toll baselines of the Reference Schematic and the existing IH-635 centerline (see Figure 2). Along the alignment, the eastbound and westbound lanes are separated by significant distances, at times over 200 feet, and subsurface conditions could change considerably from north to south through these sections as well. A general discussion of each of the sections is provided in the following paragraphs. More detailed descriptions of the existing subsurface conditions are presented in the test boring logs included in the Terra-Mar and Referenced Fugro Reports. In all cases, the Developer should evaluate the similarities of the described geologic conditions relative to his Project configuration.

Table 3-8  Sections of Similar Geologic Conditions Along Segment A

<table>
<thead>
<tr>
<th>Stationing</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Begin</td>
<td>End</td>
</tr>
<tr>
<td>67+00</td>
<td>92+00</td>
</tr>
<tr>
<td>92+00</td>
<td>130+00</td>
</tr>
<tr>
<td>130+00</td>
<td>163+00</td>
</tr>
<tr>
<td>163+00</td>
<td>191/195+00</td>
</tr>
<tr>
<td>191/195+00</td>
<td>264+00</td>
</tr>
<tr>
<td>264+00</td>
<td>312+00</td>
</tr>
<tr>
<td>312+00</td>
<td>334+00</td>
</tr>
<tr>
<td>334+00</td>
<td>356+00</td>
</tr>
<tr>
<td>356+00</td>
<td>360+00</td>
</tr>
</tbody>
</table>

3.5.1  Station 67+00 to 92+00 (Josey Lane to Webb Chapel Road)

Based on the roadway geometry presented in the Reference Schematic, U-wall and cut-and-cover tunnel construction is expected for this section of roadway.

The typical soil profile along this section of the Segment A alignment includes 0 to 8 feet of fill overlying stiff to hard clay soil of alluvial origin. The stiff to hard alluvial soil overlies weathered Eagle Ford Shale found at a depth of 30 to 70 feet. In most of the test borings in this section, a zone of non-cohesive alluvial soil was encountered at a depth of 15 to 20 feet below grade. This zone varied in thickness from about 5 feet, to as much as 20 feet, and in some cases was continuous down to bedrock. In all cases where these granular deposits were encountered, they were saturated and open graded enough to produce water observations on the drilling equipment during drilling. Cuts in this section will likely encounter these granular deposits, and drilled piers to bedrock, if used as deep foundations in this section, will also encounter these deposits. Based on readings from the standpipe and vibrating wire piezometers installed in this section, and groundwater observations on the drilling tools in some borings, groundwater is expected to be encountered approximately 10 to 15 feet below existing grade.
3.5.2 Station 92+00 to 130+00 (Webb Chapel to 1,800 ft West of Marsh Lane)

Base on the roadway geometry presented in the Reference Schematic, cut-and-cover tunnel construction is expected for this section of roadway.

A typical soil profile along this section of Segment A includes 0 to 10 feet of fill overlying residual clay soil, which overlies weathered Eagle Ford Shale. In some locations, clayey alluvial soils are also present and overly the residual clay soils. Where present however, the clayey alluvial soil layer is generally less than 20 feet in thickness. The thickness of the residual clay soil layer overlying the weathered Eagle Ford Shale averages about 12 feet, but it varies in thickness from 5 feet to as much as 20 feet, and in at least one location was observed to pinch out to zero adjacent to a thicker alluvial lens (Boring 2BW14R). In addition, the previously described tributary of Farmers Branch Creek parallels and at times is directly under the roadway alignment through the western end of this section, from Station 115+00 to Station 130+00, and potentially extending into the next section, to approximately Station 140+00 (Figure 3).

An examination of borings potentially affected by this tributary failed to disclose any clear evidence of buried channel deposits. In general, the clays described as alluvial in the borings were high plasticity clays (CH), and virtually indistinguishable from the residual clays. In one location a relatively weaker zone in the alluvial clay was observed (BW14L), although even in that case, the material still classified as a stiff clay, and its plasticity was close to that of the underlying residual soil. In addition, based on blow counts, pocket penetrometer measurements, and available unconfined compression tests, the strength and consistency of soils in the borings was only slightly lower than that of the underlying residual clays, and did not suggest the presence of a "softer" buried channel. Though not identified in the borings to date, the potential for the presence of this feature does exist, and may significantly impact the stratigraphy to be encountered. Based on readings from the standpipe and vibrating wire piezometers installed in this section, and groundwater observations on the drilling tools in some borings, groundwater is expected to be encountered approximately 0 to 10 feet below existing grade.

Based on anticipated excavation depths of less than 40 feet, the bottom of the excavations for the cut-and-cover tunnels in this section will be located in weathered or fresh Eagle Ford Shale, and the walls will be located in weathered Eagle Ford Shale, its residual clay soil, clayey alluvial soils, and shallow surface fill soils. The upper 5 to 15 feet of soil can be expected to include cracks related to the shrink/swell characteristics of the residual clay and cohesive alluvial materials. The weathered and underlying fresh Eagle Ford Shale can be expected to contain existing structural discontinuities including low strength slickensided joints, which will contribute to instability of cut slopes in this material.

3.5.3 Station 130+00 to 163+00 (1,800 ft West of Marsh Lane to 1,500 ft East of Marsh Lane)

Based on the roadway geometry presented in the Reference Schematic, cut-and-cover tunnel construction is expected for this section of roadway.
A typical soil profile along this section of the Segment A alignment includes 0 to 8 feet of fill overlying residual clay or weathered Eagle Ford Shale. Where it does exist, the residual clay soil varies between 5 and 15 feet in thickness, although along the westbound baseline profile the residual soil cover is often thin or missing, with shallow clay fill only a few feet thick and resting directly on weathered Eagle Ford Shale. Fresh Eagle Ford Shale was generally encountered at approximately 20 to 30 feet below grade at the boring locations, but at one location along the westbound baseline profile (i.e., Boring 2BW22L) the fresh Eagle Ford Shale was encountered at a depth of only 3 feet due to its location in a roadway cut for the existing IH-635 freeway. The eastern 1,200 feet of this section will also extend up through the Austin Chalk/Eagle Ford Shale contact and into the lower member of the Austin Chalk. The Fishbed Conglomerate, lying at the base of the lower chalk member will be encountered throughout these 1,200 feet. Based on readings from the standpipe and vibrating wire piezometers, and groundwater observations on the drilling tools in some borings, groundwater is expected to be encountered in this section approximately 10 to 20 feet below existing grade.

Based on anticipated excavation depths of less than 40 feet, the base of the cut-and-cover tunnel excavation will likely be in fresh Eagle Ford Shale. The walls will be excavated in the weathered and fresh Austin Chalk, the bottom 10 to 12 feet of which will be Fishbed Conglomerate, fresh and weathered Eagle Ford Shale, its residual clay soil, clayey alluvial soils, and shallow surface fill soils. This section of Segment A will also continue directly through the unnamed branch tributary of Farmers Branch Creek. Although a thin seam (i.e., less than 3 1/2 feet thick) of “gravelly clay with sand” was noted in boring BW17L at Station 134+21, the tributary is not believed to have been readily identified in the borings. The upper 5 to 15 feet of soil may include cracks related to the shrink/swell characteristics of the residual clay and shallow alluvial soils, and the weathered and underlying fresh Eagle Ford Shale can be expected to contain existing structural discontinuities, which will contribute to instability of cut slopes in this material.

3.5.4 Station 163+00 to 191+00/195+00 (1,500 ft East of Marsh Lane to West Mined Tunnel Portals)

Based on the roadway geometry presented in the Reference Schematic, cut-and-cover tunnel construction transitioning to mined tunnel at the west mined tunnel portals is expected for this section of roadway.

Along this section, the Segment A alignment climbs up onto the local plateau formed by the more resistant Austin Chalk. A typical soil profile through this section includes 0 to 8 feet of fill overlying approximately 5 to 10 feet of weathered Austin Chalk overlying fresh Austin Chalk. Portions of this section will also encounter the Fishbed Conglomerate located at the base of the lower member of the Austin Chalk. Based on the readings from a single standpipe piezometer (i.e., Boring BW30L), groundwater through this section and at the west portals is expected to be encountered at approximately 15 to 20 feet below existing grade.

Based on anticipated excavation depths ranging from 40 feet on the west to 60 feet on the east near the mined tunnel portals, the Eagle Ford Shale may be encountered only at the far western end of the section. In that vicinity, the Eagle Ford Shale may be in the bottom of the excavation or just beneath a thin layer of Fishbed Conglomerate and fresh Austin Chalk at the bottom of the excavation. The depth to the top of the Eagle Ford
Shale will then increase toward the east as the roadway grade rises and the Eagle Ford Shale/Austin Chalk contact dips to the southeast. In both the eastbound and westbound roadways, the Fishbed Conglomerate will be at or near the base of the wall for the westernmost 1,000 to 1,500 feet of these sections.

The west portal for the eastbound mined tunnel is located at Station 191+00. A typical soil profile at that location includes a nominal fill layer of 2 feet, underlain by 15 feet of weathered Austin Chalk overlying fresh Austin Chalk. Approximately 18 feet of fresh Austin Chalk is located above the crown of the tunnel.

The west portal for the westbound mined tunnel is located at Station 195+00. Just to the east of the portal location at approximate Station 195+65, boring 2T3L drilled during the Phase 2 geotechnical investigation indicates that an increase in the thickness of weathered Austin Chalk, and consequent increase in depth to fresh Austin Chalk occurs at that location. The boring indicates fill overlying approximately 8 feet of residual soil, overlying approximately 16 feet of weathered Austin Chalk, and leaving only 13 feet of fresh rock over the crown of the tunnel at the portal. The borings along the westbound profile (Figure 4), both to the east and west suggest this is a localized feature, with a thickness of weathered rock at boring T2L (to the west) of 9 feet, and a thickness of 8 feet at boring T4L (to the east).

Although the west portals of the mined tunnels are well below the elevation of the Bentonite Marker Bed, which has been eroded away at this location, there are several thinner bentonite seams that have been described in the test borings. Some of these may be encountered in the walls of the mined tunnel at the portals and in the tunnel portal face. In contrast, both the Fishbed Conglomerate and the Austin Chalk/Eagle Ford Shale contact are at minimums of 10 feet and 24 feet below the invert of the eastbound and westbound mined tunnels, respectively at the west mined tunnel portals and should not impact construction.

3.5.5 West Mined Tunnel Portals to Station 264+00 (Mined Tunnel)

Based on the roadway geometry presented in the Reference Schematic, the sections of mined tunnel from the west portals to Station 264+00 will be excavated in the lower member of the Austin Chalk. The lower chalk member is typically characterized as massive, light to medium gray limestone with thin calcareous shale and argillaceous limestone interbeds. Based on readings from the standpipe and vibrating wire piezometers installed in this section, groundwater is expected to be encountered at or within a few feet of the ground surface.

Previous local tunneling projects have noted little or no variation in strength properties of the Austin Chalk with depth. The findings presented in Figure 8 are consistent with the data from previous projects, and do not clearly indicate a relationship between unconfined compressive strength and depth or location along Segment A. However, additional data collected by Fugro which summarizes the argillaceous content of the Austin Chalk, indicates as expected that this section in the lower member has less frequent argillaceous zones or seams than the middle member to the east (Figure 9).
3.5.6 Station 264+00 to 305+00 (Mined Tunnel)

Based on the roadway geometry presented in the Reference Schematic, this section of mined tunnel along Segment A will be excavated within the lower member of the Austin Chalk. However, materially this section will differ from the previous section. First, the regionally persistent Bentonite Marker Bed will be located within 15 feet of the crown of the tunnel from approximately Station 264+00 to 305+00. Second, as shown on Figure 9, the extent of the increased frequency of argillaceous rock in the middle member of the Austin Chalk extends to a depth of approximately 10 feet below the Bentonite Marker Bed, and may also be a factor in the ground response of the crown of underground excavations through this section. Thirdly, several minor shears or faults are suspected through this section of Segment A. The main area of faulting is expected to occur just to the east of the North Dallas Tollway (Figure 4), and using the Bentonite Marker Bed as a datum, the boring log data suggests in that area there are several locations where vertical displacements of 10 feet or more may occur. Similarly, Figure 10 tends to suggest a higher overall incidence of fractures in the vicinity of the tollway. The impacts of these factors on the design and construction of the mined tunnel will be presented in Chapters 5 and 7, respectively. Based on readings from the standpipe and vibrating wire piezometers installed in this section, groundwater is expected to be encountered at or within a few feet of the ground surface.

3.5.7 Station 305+00 to 334+00 (Mined Tunnels to East Mined Tunnel Portals)

Based on the roadway geometry presented in the Reference Schematic, this section of mined tunnel along Segment A will be excavated in both the middle and lower members of the Austin Chalk. From west to east, excavation will progress with the crown of the tunnel located in the more argillaceous middle marl member of the Austin Chalk, and the rock below the crown being excavated in the lower chalk member. The middle marl member is generally characterized as light to medium gray limestone with frequent calcareous shale and argillaceous limestone interbeds; the bentonite layers are generally 1 to 2 in thick and more frequent than in the lower and upper chalk members. From west to east, the portion of the excavation at which the Bentonite Marker Bed will be encountered will progress from the crown of the excavation, through the sides of the tunnels and eventually through the invert of the mined tunnel and below. At this point, the tunnels will be excavated entirely within the middle marl member of the Austin Chalk. As presented earlier, the middle marl member is considerably more argillaceous than the lower chalk member (Figure 9) and is generally considered to have more continuous to non-continuous bentonite seams. In addition, Figure 10 suggests a higher incidence of fractures here compared to further west along Segment A. The impacts of these fractures and the increased argillaceous material and bentonite seams on the ground response to excavation, and on ground support and trafficability will be presented in Chapters 5 and 7.

The east portals for both the westbound and eastbound mined tunnels are located at Station 334+00. In test borings performed near the proposed portal locations, soil cover was only 1 to 2 feet thick. Immediately below the soil was weathered Austin Chalk, varying between 5 feet and 12 feet in thickness, and overlying the fresh Austin Chalk. Approximately 12 to 15 feet of fresh Austin Chalk are located above the crown of the mined tunnels at the east mined tunnel portals. Based on readings from the standpipe and vibrating wire piezometers installed in this section, groundwater is expected to be
encountered through this section and at the east mined tunnel portals at or within a few feet of the ground surface.

3.5.8 Station 334+00 to 356+00 (East Mined Tunnel Portals to 700 ft West of Hillcrest Road)

Based on the roadway geometry presented in the Reference Schematic, cut-and-cover tunnel construction transitioning to U-wall construction is expected for this section of roadway.

In this section of Segment A, the ground surface gradually descends toward the White Rock Creek valley. A typical soil profile includes 2 feet of fill overlying 5 feet of weathered Austin Chalk, in turn overlying fresh Austin Chalk. As for the previous section, Figure 10 suggests a higher incidence of fractures here compared to further west along Segment A. In addition, at the ground surface, some surface cracks in the fill were observed along this section, and are likely related to seasonal changes in moisture content in soils with moderate to high shrink-swell potential. Based on the observations in several piezometers in this section, groundwater is expected to be encountered at or within a few feet of the ground surface.

3.5.9 Station 356+00 to 360+00 (End Below Grade Sections to West of Hillcrest Road)

Based on the roadway geometry presented in the Reference Schematic, U-wall construction transitioning to short retaining wall construction as the below-grade segments rise to meet the existing grade are expected for this section of the roadway.

The topography through this section of Segment A slopes down to the east. A typical soil profile in the center of this section consists of approximately 12 feet of fill overlying stiff to hard plastic alluvial clay soil overlying fresh Austin Chalk. Based on anticipated excavation depths, the excavation here will start in weathered or fresh Austin Chalk near the western end of this section, but will transition into cuts within stiff clay soils (fill or alluvial) as the roadway rises to grade. Groundwater is expected to be encountered at or within a few feet of the ground surface in this section.
4.0 PREVIOUS CONSTRUCTION EXPERIENCE

4.1 General

Previous construction experiences in the Dallas area in both mined tunnel and open cut excavations are described in the following sections. Locations of these case histories relative to the IH-635 Managed Lanes Project are presented in Figure 11.

4.2 Mined Tunnels in Austin Chalk

To date there has been considerable experience in the construction of underground facilities in the Austin Chalk. Five of the most notable local projects include: the Superconducting Super Collider, the tunnels for Dallas Area Rapid Transit, the Addison Airport Toll Tunnel, the Cole Park Detention Vault project, and various drainage tunnels constructed for the I-75 North Central Expressway.

A thorough description of each of these projects, along with a summary of relevant geotechnical information, means of excavation and muck handling, production rates, initial and final ground support, waterproofing methods, etc., is provided in the Phase 1 GBR (Fugro 2004b), and will not be repeated here.

4.3 Vertical Cuts in Austin Chalk

The Austin Chalk is generally considered strong enough to support itself vertically for the long-term design condition. It is nevertheless customary to install rock reinforcement to control large and small rock wedges originating from localized fractures and joints that have orientations conducive to failure once the excavation is opened.

4.3.1 DART Mockingbird Station

The Mockingbird Station of DART was constructed using open cut excavation methods, and is approximately 37 feet deep. The upper 9 feet of the excavation was in clayey soil, either fill or residual clay derived from the Austin Chalk. The lower 28 feet of the excavation was in sound Austin Chalk limestone. The walls for the station were ultimately constructed with pre-cast fascia panels fixed to tiebacks that had been installed in the rock to control possible slides along pre-existing discontinuities. The overlying soil was stabilized for both the short-term (i.e., during construction) and for the long-term utilizing soil nailing and shotcrete prior to installation of the fascia panels.

The Geotechnical Interpretive Report for the project (SWL 1988) included loading diagrams for yielding walls based on an active earth pressure with the coefficient determined according to the backfill type. This resulted in an equivalent fluid pressure triangular loading distribution based on a soil unit weight ranging from 20 to 55 pcf. That value assumes that either the poorest grade of backfill might be used (native clay soils) or that the distance between the back of the wall and the cut slope face might only be a few feet, in which case the earth pressure associated with the native undisturbed clay would be exerted on the wall. Added to this triangular earth pressure distribution was a
uniform loading of 100 psf to account for surcharge loads at the ground surface. In addition, the design loading included a triangular hydrostatic loading associated with the observed groundwater table near the ground surface.

The report pointed out that the residual clays on site had moderate to high swell potential, and a test on a sample of the residual clay indicated a swell pressure of almost 2,000 psf. However, no additional wall loading associated with swelling clay was included in the above described design envelope. The report did not elaborate on why swell pressures were not included in the design loading.

For the walls in rock, the report stated that the Austin Chalk, if stabilized via rock reinforcement to control localized wedges and raveling, was considered capable of supporting itself, wall pressures exerted by the rock would be small, on the order of 20 psf. In addition, a triangular hydrostatic loading based on the associated groundwater table near the ground surface was added to this nominal pressure exerted by the intact Austin Chalk.

4.3.2 Addison Airport Toll Tunnel

Both the east and west approaches to the 1,650-foot long tunnel below Addison Airport in Addison, Texas were constructed as vertical cuts up to 45 feet deep. In general, the cuts were in sound Austin Chalk overlain by weathered Austin Chalk with a thin veneer (i.e., 1-3 feet) of residual soil, although both soil thickness and weathered rock thickness varied somewhat along each excavation.

At the approach on west end of the tunnel, the cut in the vicinity of the mined tunnel portal was intersected by a buried channel feature filled with alluvium to a depth of approximately 20 feet. Consequently, wall designs providing substantial horizontal support were developed for the west end, employing closely spaced cantilevered drilled shafts (i.e., 9 to 16 inches between shafts) with diameters ranging from 36 to 60 inches. A fascia wall system was attached to the drilled shafts.

At the approach on the east end of the tunnel, the weathered Austin Chalk was removed at the top of the excavation to permit founding of a retaining wall in sound Austin Chalk, behind which granular backfill was placed. Along the north (i.e., south-facing) retaining wall, no permanent rock support was installed. Rock dowels penetrating the rock 6.5 feet were installed in a 7.5-foot square grid pattern to provide anchorage for cast-in-place closure/support elements at junctions between panels of the precast fascia wall system. Along the south retaining wall, where the possibility of an additional tunnel tube in the future was provided for in the design, the cut was sloped back and supported using 10-foot long rock bolts on a 7-foot square grid in conjunction with 4 inches of shotcrete reinforced with welded wire fabric. Subsequently, an aesthetic fascia constructed of precast concrete modular block was installed and anchored to the rock bolt reinforcing system.

During construction, excavation of the cut at the east end of the project near the mined tunnel portal exposed two prominent fractures in the north wall of the excavation. In addition, portions of a drainage culvert that had conveyed storm water runoff flow below one of the taxiways of the Addison Airport had been removed such that it drained into the excavation along the north wall. Heavy rains resulted in flooding of the excavation to
significant depths (at least 10 feet) for a substantial period of time, submerging at least part of the prominent fractures. A slide involving perhaps 500 cubic yards of rock occurred in January 1998, apparently triggered by vibration from a hoe-ram dressing the cut wall surface relatively soon after the excavation was pumped out after the flooding. Subsequent analysis of the failure indicated the slide extended a distance of approximately 20 feet behind the face of the excavation and involved approximately 30 feet of wall height. It occurred along a curved failure surface, consistent with the description of discontinuities presented by Blakemore (1939) and with other anecdotal information from Dallas-area geotechnical professionals. It is believed that exposure of the existing curvilinear fractures, coupled with the flooding of the excavation, created conditions that were conducive to slope instability. The equipment vibration appeared to trigger the subsequent movement. Construction at the site was allowed to proceed only after supplemental rock bolting and design of remedial treatment of the rock surface left by the failure was completed.

4.4 Vertical Cuts in Eagle Ford Shale

The Eagle Ford Shale is well recognized in the Dallas area as a “bad actor”, with the majority of construction problems related to its high swell potential, its low residual shear strength, or both. These problems exist not only for the intact Eagle Ford Shale, but also for the weathered Eagle Ford Shale and the residual clays derived from it.

4.4.1 President George Bush Turnpike, Carrollton Section

Along the President George Bush Turnpike in Carrollton, several wall sections up to 4,200 linear feet long, with wall heights up to 34 feet were designed as pre-cast fascia panels affixed to 30-inch and 36-inch diameter tied-back drilled shafts on 5 to 10 foot centers. The walls were to retain residual clays overlying weathered Eagle Ford Shale. In some cases the lower portions of the walls were within intact Eagle Ford Shale. The design loading was based on an equivalent fluid pressure approach, using a unit weight of 87.5 pcf for the retained material, and no additional hydrostatic pressure was included. A surcharge load equivalent to 2 feet of earth was added (Mas-Tek 1997).

Selection of the required anchorage length for the anchors grouted into rock for final design was determined on the basis of an extensive test program conducted by NTTA in 1999 using full scale grouted anchors tested to failure. The test program revealed that anchor shear strength in the Eagle Ford Shale was considerably lower than had been anticipated during preliminary design. Peak failure stresses at pullout failure were found to be on the order of 10 to 30 psi (1,500 to 4,300 psf). Although the failure patterns exhibited the typical brittle mechanism of rock anchor pullout, and the ultimate yields were larger than pullout loads in strong soil, the strengths were well below what would otherwise have been expected for intact rock prior to the testing program.

Prior to construction, a value-engineering (VE) alternative was offered by the contractor and accepted by the owner, replacing the tied-back design with mechanically stabilized earth (MSE) walls for all sections except the bridge abutments. In the end, the VE alternative was selected and only the bridge abutments were constructed per the tied-back drilled shaft design. The rest of the walls were constructed using a MSE and backfill design (Bouma 2005).
4.4.2 President George Bush Turnpike, Las Colinas Section

In December 2004, several sections of wall along the President George Bush Turnpike near Las Colinas Boulevard in Irving, moved sufficiently to prompt the North Texas Tollway Authority (NTTA) to close the outer turnpike lanes and install stabilizing MSE buttresses on either side of the bridge abutment in the vicinity of the movement. The wall was originally constructed by stabilizing the Eagle Ford Shale with vertical soil cement columns that were mixed in situ and drilled into the Eagle Ford Shale in a horizontal grid pattern. The section that failed was approximately 20 foot high and retained a sloping backfill cut in the Eagle Ford Shale.

The walls had been observed to be moving shortly after the roadway was opened to traffic in 2001, and were being monitored. However, acceleration in the rate of movement in December 2004 led NTTA to take action. The cause of the movement is currently under investigation by NTTA, and information is also being gathered to develop a design for the permanent repairs (Bouma 2005).

4.4.3 One Main Place, Downtown Dallas

A 70-foot deep vertical cut for the foundation of the One Main Place building in Dallas, was excavated in the Austin Chalk and bottomed in the Fishbed Conglomerate. Over a period of several hours in January 1967, a large section of wall failed as an intact block and translated horizontally approximately 20 feet into the excavation. An entire block of Elm Street pavement subsequently subsided into the cavity that formed behind the translated block. Later investigations attributed the movement to an existing near vertical joint in the Austin Chalk and the presence of relatively weaker material at the contact between the Austin Chalk and the Eagle Ford Shale. Pertinent technical details are not available due to litigation following the incident and the sealing of proceedings associated with the case. The information provided here was obtained from Dallas Morning News articles written throughout the year following the date of the incident.

4.4.4 I-30 Widening, Dallas, Loop 12 to Sylvan

Similar to the Carrollton Section of the President George Bush Turnpike, this 4.75 mile widening project constructed in 2003, included wall sections over 30 feet in height. One of the walls, a 1,083 foot long segment (Wall 4A) was constructed using pre-cast fascia panels affixed to 30-inch diameter tied-back drilled shafts that were placed on 6.5-foot centers. The drilled shafts at the east end of Wall 4A were founded in the Eagle Ford Shale and they retained a stiff to very stiff clay.

The design required that all tiebacks grouted into sound shale and stiff clay were to be proof tested to at least 120 percent of the design load, and 5 percent were to be tested to 133 percent of the design load (TxDOT 2002).
4.4.5 Landfill Case History, Slope Failure

At the Hunter-Ferrell Landfill in Irving, Texas, an 80-foot high slope in the Eagle Ford Shale constructed at approximately a 1V:3H (18 degree) angle, failed overnight in March 1999. Forensic geotechnical investigations were performed, ultimately concluding that the presence and impact of water along a bedding plane, which was at or near its residual strength, was the probable cause of the failure (Haney 2005).

4.4.6 DFW Airport, Slope Failure During Construction of Utility Corridor

In 1971, during the construction of the Dallas/Fort Worth (DFW) airport, a slope failure occurred in excavations that were being made for a cut-and-cover utility tunnel along the alignment of the main north-south access road into the airport, commonly referred to as “Spine” Road. The excavation in the Eagle Ford Shale was approximately 40 feet deep with near vertical cuts in the shale and side slopes in the overlying soil laid back at close to 1:1. The failure resulted in three construction worker fatalities (Dallas Times Herald May 11, 1971).
5.0 DESIGN CONSIDERATIONS FOR MINED TUNNELS

5.1 General

This chapter presents design considerations for the mined tunnels and associated underground structures as depicted in the Reference Schematic. The Developer is responsible for performing any necessary geotechnical exploration and testing, for determining appropriate design values, and for applying those values in the appropriate analytical models to assess design loadings and design the facilities, consistent with the planned drainage systems, and the Developer's intended construction means and methods.

5.2 Ground Response

The ground response and stability of underground excavations in the massive, relatively low-strength Austin Chalk will be controlled largely by one or a combination of the following factors:

- in situ stresses and construction induced stresses that will develop in response to excavation;
- the location, thickness and strength of bentonite seams and argillaceous zones above, within and below the tunnel envelope; and the tendency of these seams to deteriorate or swell in response to exposure and/or water; and
- the frequency and orientation of discontinuities in the rock mass and by the characteristics and shear strength of these discontinuities.

In the latter case, rock wedges of varying sizes that require stabilization should be expected based on the overall variability in joint spacing, orientation and characteristics of the discontinuities in the Austin Chalk.

While no faults were directly intersected in the borings, including the angled borings drilled specifically for identifying these features, correlations of the offsets of the Bentonite Marker Bed in the borings, suggest that zones of faulting and shearing will be encountered during tunnel excavation. The underground engineering significance of these features is noted by zones of highly fractured rock, and often by the presence of low-strength fault gouge. In the Austin Chalk, these features are often healed and may not significantly impact excavation progress. In some cases however, more intensely sheared zones may be significantly weaker that the host limestone mass resulting in an increased tendency for blocky ground conditions, shorter stand-up times and increased potential for significant groundwater fracture flow. The groundwater inflows can lead to further stability problems or squeezing ground requiring heavier than normal support.

Potential instabilities in the Austin Chalk may also result from excavation spans too large to provide sufficient stand-up times necessary for installation of initial ground support. In areas of thin rock cover (i.e., near the portal locations) or in areas where unfavorable geologic conditions exist, such as the presence of the Bentonite Marker Bed in or above the tunnel crown, excavation spans must be designed to anticipate and prevent such instabilities. Safe and feasible span dimensions, along with the resulting stand-up times,
will increase as the thickness of unweathered rock over the tunnel crown increases and/or as the distance below a feature such as the Bentonite Marker Bed increases. The actual width of the unsupported span will, therefore, be a function of the depth of the tunnel and the thickness and characteristics of the overlying fresh Austin Chalk. Restrictions on the width of the unsupported span should be considered by the Developer and included in the design.

5.3 Initial Ground Support

5.3.1 Mined Tunnels and Associated Underground Structures

It is anticipated that the initial ground support designs for the mined tunnels and associated underground structures will be comprised primarily of rock dowels and fiber reinforced shotcrete. If highly fractured ground is encountered, lattice girders and/or spiling may also be required. Based on ground conditions, lengths and spacing of rock reinforcement will vary, as will the required thicknesses of shotcrete. In addition, in the mined tunnels it is expected that an observational approach to initial support installation will be used. This approach will provide designs for various classes of ground support that will be used in conjunction with excavation of multiple drift openings with spans and lengths that can be adjusted depending on encountered ground conditions. This approach can be used to ensure proper support of the rock arch and safe working conditions in the tunnel. Observation of rock conditions and behavior in the tunnel by an experienced practitioner should be used to validate or adjust design support schemes and to ensure an application of the correct support and excavation sequence for the encountered conditions.

For the vertical shafts depicted in the Reference Schematic, shaft collars should be installed through the overburden soils and weathered rock, and keyed or socketed into fresh rock. The design of these elements should consider at a minimum earth loads including potential swell pressures, hydrostatic pressures, equipment and surcharge loadings, and the impacts of construction sequencing.

In the fresh Austin Chalk below the shaft collar, the primary role of initial ground support will be to prevent loose wedges or blocks of rock from sliding into the excavation. It is anticipated that the initial ground support will be comprised primarily of light pattern dowels and shotcrete.

5.3.2 Portal Design

The mined tunnel portals will serve as the key points of access to the heading of each underground excavation during construction. As such, the structural stability of these features must be ensured. The portal locations were originally chosen to provide for a minimum of 15 feet of fresh Austin Chalk above the crown of the tunnel at each location. However, as presented in Chapter 3, findings in one of the Phase 2 borings taken in the vicinity of the westbound mined tunnel portal at Station 195+00 indicated an increase in thickness of weathered rock in the vicinity of the portal, and suggesting that the thickness of fresh rock at that location may be less than the minimum 15 feet. During
final design, the Developer should further investigate the geotechnical conditions at the mined tunnel portals relative to his project configuration and tunnel geometry.

It is anticipated that the primary ground support for the highwall above each portal will be comprised primarily of patterned rock dowels and shotcrete. The rock reinforcement and shotcrete should be installed as the excavation for the portal face is deepened, in a sequenced approach, in order to reduce the height of unsupported highwall. Based on the geotechnical conditions at each portal location and the Developer's planned means and methods, consideration should also be given to installing additional horizontal rock reinforcement or spiling at each portal, just above the crown of the tunnel to further fortify the structural integrity of the opening.

5.4 Final Lining Design

It is anticipated that the final surface of all underground facilities including the mined tunnels and associated underground structures will be constructed of cast-in-place reinforced concrete. The design of the final concrete lining at a minimum should account for long-term earth pressures, long-term hydrostatic pressures based on detailed design of the tunnel waterproofing and drainage system, and on potential pressures that could be induced by swelling of bentonite seams encountered during excavation.

5.5 Argillaceous Zones

Portions of the tunnel excavation, primarily but not exclusively in the middle member of the Austin Chalk, will encounter significant zones of argillaceous or marly limestone that may be subject to slaking and deterioration upon exposure to the atmosphere and moisture. These materials are likely to produce trafficability problems in the invert of the underground excavations, and the deterioration of these materials in the crown and sidewalls of the excavations will likely induce slabbing and separation of material along bedding planes. These slabs may vary in thickness from less than one inch to several inches.

In previous local projects, sealants have been used to prevent deterioration of these zones prior to application of shotcrete or final cast-in-place lining. When applied shortly after excavation, shotcrete that is required for the initial support can also serve to prevent the deterioration of these zones.

5.6 Bentonite Seams

As presented in Chapter 3, the numerous bentonite seams, including the Bentonite Marker Bed, that are present along the Segment A alignment are subject to swelling and slaking upon exposure to the atmosphere or moisture. These seams, upon exposure, may be capable of producing pressures of sufficient magnitude to induce slabbing and separation of material along bedding planes. Wedges that are created by structural discontinuities intersecting the bedding plane surfaces of these bentonite seams and daylighting within the tunnel excavation can vary in size from less than one cubic foot to blocks or slabs up to several cubic yards. These wedges may fall out during actual mechanical excavation or may dislodge over time in response to deterioration, changes
in moisture content, swelling or changes in stress in the bentonite seams. Additionally, due to the shallow dip of these bentonite seams and the flat angle at which these seams will be intersecting the underground excavations, the bentonite seams and associated slabbing and separation that are encountered may have to be dealt with for hundreds and possibly thousands of feet depending on the thickness and persistence of these features.

Design work should include development of procedures for dealing with bentonite seams of variable thickness and located at different portions within the zone of excavation. These procedures must consider bentonite seams located above, within and below the mined openings and should be integrated with the planned support schemes and observational method approach to effectively control the size of these potential slabs by limiting the length and width of exposed unsupported span.

Furthermore, as presented in the Reference Schematic, and as employed in past tunnel projects in the Austin Chalk, consideration should be given during design to preventing the development of undue long-term pressures on the tunnel liner, by requiring over-excavation and sealing of these exposed bentonite layers.

5.7 Groundwater and Seepage Control

The measured values of rock mass permeability indicate that groundwater infiltration should not present major construction difficulties in underground excavations beneath the weathered Austin Chalk. The groundwater inflows at localized shear zones found along the alignment may be significantly higher (an order of magnitude or more) than would be estimated from the steady state values of the host material. This is especially relevant in areas of thin fresh limestone cover above the crown of the running tunnels (i.e., at the ends of the tunnels at or near the portal locations).

Prediction of groundwater flow quantities for construction and long-term conditions requires consideration of flow based on both the porosity of intact rock, as well as flow along discontinuities and through fractured zones in the rock mass. The prediction process can be a complicated analytical task, but is one that must be considered and performed. The final waterproofing and drainage systems for the mined tunnels must accommodate the predicted steady state flow and any periodic transient flows through fractures that will be experienced at the tunnel-to-rock interface.

During construction, measurable inflows are most likely to occur at point sources, and the Developer should address identification and collection of inflows in the design in order to prevent the deterioration of the working invert, and to reduce the adverse effects of swelling materials on the initial and final ground support systems. For the vertical shafts, shaft collars should be designed to key into fresh rock to reduce problems associated with groundwater inflows and seepage into shaft excavations.

In the long-term, the mined tunnels and associated underground structures will in essence become large drains that will result in significant lowering of the groundwater table along the Segment A alignment. Construction processes must be designed to accommodate and handle this water in such a way that potential detrimental effects are mitigated.
In addition, due to the potential for leaching of calcium carbonate into the permanent drainage systems and for long-term calcification of these systems, the Developer should provide sufficient spare flow capacity to account for this process, and include means for maintaining flow capacity through the system over time.

### 5.8 Combustible and Toxic Gases

While no specific evidence of combustible or toxic gases (i.e., either natural or manmade) were found during the geotechnical investigations for this Project, these substances have been encountered in other local projects in the Austin Chalk (e.g., DART, Contract NC-1B, (Henn and Rogstad 1999)). In addition, one documented example of the potential for encountering similar substances in the mined tunnels is the October 1998 gasoline release/spill that occurred at a Mobil service station located on the southeast corner of the intersection of Preston Road and IH-635. Reports indicate that over a five-day period, approximately 9,700 gallons of gasoline were released from an underground storage tank at that location (EPA 1998). The Developer should consult TxDOT's Environmental Assessment for the corridor and investigate any potential areas where existing contamination could negatively impact construction.

In all cases, care must be taken by the Developer to ensure that the possibility of encountering explosive or toxic gases is considered during both design and construction. Furthermore, the Developer should classify the excavations for the mined tunnels and associated underground structures as outlined in 29 CFR 1926.800, and require equipment of suitable condition to meet his classification.

### 5.9 Corrosion Potential

Chemical testing of groundwater samples for sulfate and chloride content was performed during the Phase 2 geotechnical investigation and the results are presented in the Phase 2 GDR (Fugro 2005). These tests may be used in part for corrosion analysis, which may be further warranted due to the presence of pyrite nodules in the Austin Chalk, identified in the borings, and which can upon weathering and exposure to air and water react to produce sulfuric acid. During final design, an analysis should be performed by the Developer to assess the potential for corrosion to affect the long-term structural performance of the mined tunnels and associated underground structures.

### 5.10 Recommended Additional Subsurface Investigations

The subsurface investigations that have been completed to date have been used in progressing the development of Segment A through the preliminary engineering effort. In all cases however, the Developer is required to perform all additional geotechnical investigations needed to advance his design to the final design level and construction.
6.0 DESIGN CONSIDERATIONS FOR CUT-AND-COVER TUNNEL AND U-WALL SECTIONS

6.1 General

This chapter presents design considerations for the cut-and-cover tunnels and U-wall sections as depicted in the Reference Schematic. The Developer is responsible for performing any necessary geotechnical exploration and testing, for selecting appropriate design values, and for applying those values in the appropriate analytical models to assess design loadings and design the facilities consistent with his planned drainage systems and construction means and methods.

6.2 Long-term Earth Pressure Loading Basis

6.2.1 At-rest Earth Pressures ($K_o$) in Clay and Shale

The primary loading condition for the U-wall and cut-and-cover tunnel sections is based on the assumption of an at-rest earth pressure against an essentially rigid wall. Due to the location of the walls, at or near the right-of-way limits, the use of a yielding wall that could potentially reduce earth pressures to "active" values would potentially result in unacceptable values of surface settlement.

The applied lateral pressure under these conditions is normally separated into two components, one due to the effective weight of the soil and the other due to the hydrostatic head at the particular section. The at-rest earth pressure coefficient for the effective weight of soil is calculated as $K_o = 1 - \sin \Phi'$, where $\Phi'$ is the effective (drained) friction angle (Jaky 1944). These earth pressures should not be confused with the existing in situ stresses, as described in Chapter 3. These existing in situ horizontal stresses, if not already relieved over time due to exposure and the shallow depth, will be relieved in the area adjacent to the wall upon excavation of wall components.

The soil strata along Segment A, with the exception of the sandy alluvial soils, are generally moderate to high plasticity clayey soils (Figure 4). The appropriate effective friction angle to assume for these soils and the weathered and fresh Eagle Ford Shale was selected by combining published values and experience, correlations to index tests, and the results of triaxial strength tests performed on samples from the Phase 1 and Phase 2 geotechnical investigations. With the plasticity index as high as 52 for the clay soils, an effective friction angle ranging from 17 to 23 degrees can typically be assigned for these materials (Lambe & Whitman 1969). In addition, the results of triaxial tests performed on residual and alluvial soils from this Project indicate effective friction angles ranging from 11 to 19 degrees. Finally, from the literature, peak effective friction angles ranging from 13.5 to 24 degrees were reported for the Eagle Ford Shale (Font 1979, Lai 1997).

Based on this information, a peak effective friction angle of 13 degrees was deemed a reasonable value for use in preliminary design for the clay soils and the weathered and fresh Eagle Ford Shale along Segment A. Using this representative peak effective friction angle of 13 degrees, a $K_o$ value of 0.78 was determined.
In the case of a structural wall retaining a saturated backfill with a sustained free water surface, as is expected to be the case for much of this Project’s alignment, the above described earth pressure coefficient is applied to the effective (or “buoyant”) unit weight of the soil. The hydrostatic component of pressure due to the water surface must then be added to the above described earth pressure component to arrive at the total pressure on the wall. Given that the unit weight of soil materials on this Project is approximately 125 lb/cu ft, the result is an equivalent fluid unit weight of approximately 110 lb/cu ft. Though there are a range of methods for constructing earth retaining structures, the large lateral pressures imparted by this equivalent weight make some of the options more or less viable than others. For example, cantilever drilled shafts become impractical for all but the shortest walls (e.g., walls less than about 10 feet in height). By contrast, the majority of retaining structures in clay and shale on this Project will be significantly higher than 10 feet. For these higher walls it is anticipated that the Developer will utilize retaining structures such as tied back drilled shafts, mechanically stabilized earth (MSE), or conventional cast-in-place structural cantilever walls. For the cast-in-place concrete walls however, the large pressures described above will impart large bending moments and correspondingly require massive concrete sections. Thus cast-in-place cantilever or counterfort walls may be uneconomical compared to MSE or tieback wall alternatives.

6.2.2 Special Considerations Regarding Lateral Swell Pressures due to Swelling Clays and Clay Shales

In addition to the primary loading condition described above, this section presents design considerations regarding lateral swell pressures that should also be addressed by the Developer. Additional background discussion regarding swelling phenomenon and tests commonly used to evaluate swelling potential are presented in Appendix A.

As described by the SCS, the primary problem associated with the clay soils found along Segment A is their potential for large volume changes (shrink or swell) in conjunction with changes in moisture content. If swelling is restrained, such as by a rigid retaining wall, these materials are capable of producing lateral swell pressures several times greater than the at-rest or active earth pressures normally used in the design of retaining structures (Richards 1977). Special foundation design practices have been developed in the Dallas area to accommodate or prevent structural damage due to vertical swell and settlement associated with these materials. However, there is little reported local experience reported dealing with excessive lateral pressures on retaining structures attributable to the same phenomenon.

In view of the above, the issue of lateral swell potential must be evaluated in greater detail during detailed design. The Developer may address the potential for lateral swell pressure in a number of ways, including but not limited to the following:

1) **Further characterize the lateral swelling potential and/or pressure of the swelling clays and clay shales.** Instrumented field tests using drilled shafts fitted with horizontal earth pressure cells, such as were conducted by the Corps of Engineers at Lackland A.F.B in 1968 (USACOE 1968), may likely produce the most reliable data for design. However, in view of Project time constraints, this approach may not be practical. Indications of swelling potential can be obtained for soils using simple parameters such as the Atterberg limits, the shrinkage limit,
the percent finer than 2 microns, and the Activity Index (Holtz & Gibbs 1956). For rock, correlations to swelling potential using similar indices can be utilized if sample preparation for index testing rigorously adheres to the testing protocols specific to clay shales (USACOE 1986). For both soil and rock, measurements of lateral swell pressures can also be determined in the laboratory, although once again special procedures must be adhered to for clay shales (USACOE 1986).

2) Accommodate the maximum swell pressure in the design. Based on measurements made on similar materials in comparable applications, this pressure can be as much as 10,000 to 12,000 psf (Richards 1977). Preliminary analysis has shown that a tangent drilled shaft wall acting as a cantilever under these pressures would not be practical. However, a strutted design with a limited vertical height (25 feet or less) which could be adapted to the cut-and-cover tunnel sections on this Project may be feasible if large diameter drilled shafts (i.e., 42-60 inches) are used. Walls constructed using tied-back drilled shafts of smaller diameter might also be feasible if lateral swell pressures are moderate and if sufficient right-of-way is available to permit tieback installation. Finally, thick cast-in-place reinforced concrete walls may also be feasible.

3) Construct the walls for the cut-and-cover tunnel and U-wall sections such that the lateral swell pressures exerted by the expansive materials do not reach the wall or any wall components. This approach could include the use of frangible material placed between the expansive soils and the retaining structure, or if lateral space is available, placement of a large wedge (e.g., width equal to the passive wedge) of non-swelling backfill soil behind the wall (USACOE 1981). If MSE walls are to be considered, this non-swelling backfill zone must be wide enough to prevent the passive wedge from impacting the wall anchorage.

4) Make provisions in the design to insure that expansive clays and clay shales are protected against exposure to water. Procedures implemented in India in the 1980’s, which included placement of zones of non-swelling soils between the wall and the host expansive material, and also overlying the expansive soils at the ground surface, were successful in preventing these materials from having access to moisture (Katti 1983). Similarly, the Corps of Engineers successfully waterproofed an excavation in expansive shale in Hawaii using a spray applied bitumen-based sealant (Stroman 2005).

6.2.3 Methods of Stabilizing Vertical Cuts in Weathered and Fresh Austin Chalk

Stability of vertical excavations in the Austin Chalk, both in the short-term and the long-term conditions, is dictated not by the strength of the intact rock, but by the presence, orientation and shear strength of discontinuities present in the rock mass. Mechanisms of instability may include both simple wedge failures with blocks sliding on the lines of intersection of existing discontinuities, or a planar type failure with a large block sliding along a horizontal or sub-horizontal weak seam such as a bentonite seam or the Fishbed Conglomerate that daylights in the wall of the excavation.

It is common in the Dallas area to stabilize vertical excavations by installation of rock reinforcement including rock dowels, rock bolts or stressed tiebacks. Using this approach, particular attention must be given to identifying the presence, orientation and
persistence of existing discontinuities. Designed reinforcement must be sufficiently close in spacing and where possible extend significantly through these features to provide the necessary anchorage to prevent block displacement.

An alternative design approach is to utilize a design pressure envelope based on the active earth pressure, $K_a$, where $K_a = \tan^2 (45° - \Phi/2)$ and $\Phi$ is the friction angle along the discontinuities within the rock mass. For preliminary design this approach was taken and a friction angle of 40 degrees was assumed. Using this 40-degree friction angle, a $K_a$ value of 0.22 was determined. The utilization of $K_a$ in the equivalent fluid pressure approach is presented in Section 6.4.

6.2.4 Groundwater Level at the Ground Surface

For preliminary design, the design loading for rigid walls was developed assuming an at-rest or active earth pressure plus the full hydrostatic head. If during final design, the Developer can demonstrate that his proposed drainage system can effectively control and lower the long-term hydrostatic pressures against the wall, the portion of the above-described load that is attributable to the groundwater table could be reduced.

6.2.5 Surcharge Loads

For surcharge loads on cut-and-cover tunnel walls, the Tunnel Engineering Handbook depicts a surcharge of 600 psf at the ground surface, which is then distributed as a lateral pressure on the wall of 240 psf down to a depth of 10 feet, followed by 100 psf for the next 10 feet (Bickel et al. 1996). This is a nominal surcharge to account for normal construction activities and traffic. Surcharge loads due to construction activities involving major equipment or stockpiling materials adjacent to excavations must be added depending on the proposed construction logistics. In addition, depending on the configuration of the final roadway, there may be routine sustained live loads due to traffic that may need to be considered as a long-term loading. For this condition, it is recommended that the standard AASHTO surcharge loading be applied.

6.3 Vertical Swell Design Considerations

Although discussion of the customary impacts of vertical loads from swelling clays and clay shales on spread footings, pile or drilled shaft foundations, and pavement are specifically excluded from the scope of this report, there is a potential impact of vertical swell that will be mentioned here, and should be considered in the design of the U-wall and cut-and-cover tunnel sections along Segment A. In these sections, the Developer may be relying on the pavement slab or grade beam to provide lateral support at the base of the structural walls. If this slab or “beam” experiences vertical swell pressures due to underlying swelling soils or rock, that uplift pressure should be included in the design of the pavement slab acting as a lateral strut.

Recommendations and considerations regarding expansive soils, and the impacts of uplift on foundations, pavement, etc. due to vertical swell are presented in the Phase 1 GBR (Fugro 2004b).
6.4 Long-term Loading on Walls (by Station)

Structural designers often prefer to express earth loads on walls via an “equivalent fluid pressure”, similar to the triangular pressure distribution of water on a wall. This equivalent fluid pressure is arrived at by multiplying the effective unit weight of the soil (e.g., 125 pcf - 62.4 pcf = 62.6 pcf) by the earth pressure coefficient. Using a $K_o$ of 0.78 based on an effective friction angle of 13 degrees, the resulting equivalent fluid unit weight from the soil would be 48.8 pcf. The equivalent fluid pressure from the soil at any depth can then be determined by multiplying the equivalent fluid unit weight by the depth. The groundwater hydrostatic pressure must then be added to this value to arrive at the total pressure for structural design. This equivalent fluid pressure approach can also be taken using the active earth pressure coefficient, $K_a$, in cases where active earth pressure behavior is likely.

In spite of its simplicity and widespread use in preliminary design, there are shortcomings with the equivalent fluid pressure approach. The approach does have difficulty in dealing with variable groundwater levels, and it does not take into account additional lateral pressures that may be imparted long-term to the wall by swelling clays or clay shales. In those sections along Segment A where the lateral swell potential is judged to be significant based on the subsurface information currently available, additional lateral pressures must be determined by performance of additional specialized laboratory tests and considered in the design to account for this long-term swell potential.

Pressures on the passive, or "resisting" side, should be determined using a conventional $K_p$ coefficient based on an effective friction angle of 13 degrees for those sections that will be founded essentially in the alluvial and/or residual clays. If a particular section can be demonstrated to be entirely founded in sandy alluvial soils, the passive earth pressure can be based on a higher friction angle such as 30 degrees. For the sections founded in weathered or fresh rock, it is assumed that the determination of the resisting forces will be based on an elastic analysis. For these sections, the elastic properties can be selected from the data presented in Tables 3-2 and 3-5. A summary, by station, of the preliminary design pressures and the assumptions implicit in those pressures for each section along Segment A are presented in Table 6-1.

6.4.1 Station 67+00 to Station 92+00 (Josey Lane to Webb Chapel Road)

Critical design earth pressures in this section of Segment A for long-term loading are likely to be caused by the high plasticity alluvial clay. Using a $K_o$ of 0.78 based on an effective friction angle of 13 degrees, the resulting equivalent fluid unit weight from the soil would be 51.2 pcf. The equivalent fluid pressure from the soil at any depth can then be determined by multiplying the equivalent unit weight by the depth. If considerable depths of granular deposits, more representative of the sandy alluvial soils, are found to exist within this section, their effective friction angle would be greater than 13 degrees and the above described earth pressure would be considered conservative. For design purposes, the groundwater level along this section should be considered at or near existing grade and the appropriate hydrostatic pressure should be included in the design pressures.
In addition, due to the highly plastic nature of these clayey alluvial soils, the impact of swelling on lateral loadings must be evaluated for this section in light of the Developer's planned means and methods, and the appropriate additional lateral swell pressure should be included in the design. Furthermore, if the pavement slab is to be counted on to provide lateral restraint to the base of walls in this section, the potential for uplift due to swelling soils to cause flexural loading on the slab acting as a strut should be considered.

6.4.2 Station 92+00 to Station 130+00 (Webb Chapel to 1,800 ft West of Marsh Lane)

Critical design earth pressures in this section of Segment A for long-term loading are likely to be caused by the high plasticity residual clay of the Eagle Ford Shale, and the weathered Eagle Ford Shale. Using a $K_o$ of 0.78 based on an effective friction angle of 13 degrees, the resulting equivalent fluid unit weight from the soil would be 48.8 pcf. The equivalent fluid pressure from the soil at any depth can then be determined by multiplying the equivalent unit weight by the depth. For design purposes, the groundwater level along this section should be considered at or near existing grade and the appropriate hydrostatic pressure should be included in the design loading.

The possibility of encountering soft alluvial deposits associated with an unnamed tributary to Farmers Branch Creek, and the potential impact of these deposits on the design lateral earth pressure were evaluated for this section. Although no soft zones were identified in the borings, a pessimistic case assuming a full height of soft to medium stiff, $c = 500$ psf clay overlying bedrock was evaluated. The analysis indicated that the soft to medium stiff clay would produce total lateral earth pressures less than those produced by the above described equivalent fluid pressure based on $K_o$ with full hydrostatic head. While the presence of a thick softer deposit such as this may impact temporary wall stability during construction and require special treatment to control deformations, this analysis indicates it would not adversely impact the long-term loadings if the above described $K_o$ approach is used for design.

Due to the highly plastic nature of the residual clay and the weathered Eagle Ford Shale, the impact of swelling on lateral loadings must also be evaluated for this section in light of the Developer's planned means and methods, and the appropriate additional lateral swell pressure should be included in the design. Furthermore, if the pavement slab is to be counted on to provide lateral restraint to the base of walls in this section, the potential for uplift due to swelling soils to cause flexural loading on the slab acting as a strut should be considered.

6.4.3 Station 130+00 to Station 163+00 (1,800 ft West of Marsh Lane to 1,500 ft East of Marsh Lane)

Critical design earth pressures in this section of Segment A for long-term loading are likely to be caused by the weathered and fresh Eagle Ford Shale. Using a $K_o$ of 0.78 based on an effective friction angle of 13 degrees, the resulting equivalent fluid unit weight from these materials would be 51.9 pcf. The equivalent fluid pressure from the soil at any depth can then be determined by multiplying the equivalent unit weight by the depth. For design purposes, the groundwater level along this section should be
considered at or near existing grade and the appropriate hydrostatic pressure should be included in the design loading.

The potential for encountering soft alluvial deposits associated with an unnamed tributary to Farmers Branch Creek was also evaluated for this section. For the reasons discussed above in Section 6.4.2, the presence of a thick soft deposit should not adversely impact the long-term loadings provided the above described equivalent fluid pressure based on $K_o$ plus full hydrostatic head, is used for design.

Due to the highly plastic nature of the residual clay and the Eagle Ford Shale, the impact of swelling on lateral loadings must also be evaluated for this section in light of the Developer's planned means and methods, and the appropriate additional lateral swell pressure should be included in the design.

In addition, on the eastern end of this section, the bottom of the excavation will be at or near the Austin Chalk/Eagle Ford Shale contact, and may be within the Fishbed Conglomerate, which is, on average, weaker than the rest of the Austin Chalk and may have a higher incidence of fractures. If a near vertical discontinuity extends to depth in this area, the potential exists for a translational type block failure with sliding along the top of the Austin Chalk/Eagle Ford Shale contract or through the Fishbed. However, the use of the $K_a$ approach in the vicinity of this contact may not be conservative in light of the possibility of a slightly inclined plane in the weaker material at this contact, and the Developer may wish to revaluate the design loading on the wall through this transitional area consistent with proposed design details, means and methods, and construction sequence.

6.4.4 Station 163+00 to Station 191+00/195+00 (1,500 ft East of Marsh Lane to West Tunnel Portals)

Critical design earth pressures in the majority of this section of Segment A for long-term loading are likely to be caused by the weathered and fresh Austin Chalk. These materials can potentially stand vertically for much of their exposure, but also have the potential for developing unstable rock wedges at intersecting discontinuities. Accordingly, it is recommended that a design relying either on a rock reinforcement program based on more detailed discontinuity data, or an envelope of wall pressure based on $K_a$ be adopted. Using a $K_a$ of 0.22 based on a friction angle of 40 degrees, the resulting equivalent fluid unit weight from the soil would be 16.9 pcf. The equivalent fluid pressure from the rock at any depth can then be determined by multiplying the equivalent unit weight by the depth. For design purposes, the groundwater level along this section should be considered at or near existing grade and the appropriate hydrostatic pressure should be included in the design loading.

In addition, on the western end of this section, the bottom of the excavation will be at or near the Austin Chalk/Eagle Ford Shale contact, and may be within the Fishbed Conglomerate, which is on average weaker than the rest of the Austin Chalk and may have a higher incidence of fractures. If a near vertical discontinuity extends to depth in this area, the potential exists for a translational type block failure with sliding along the top of the Austin Chalk/Eagle Ford Shale contact or through the Fishbed. The use of the $K_a$ approach in the vicinity of this contact may not be conservative in light of the possibility of a slightly inclined plane in the weaker material at this contact, and the Developer may wish to revaluate the design loading on the wall through this transitional area.
Furthermore, where the base of the excavation is at or near the Austin Chalk/Eagle Ford Shale contact, the potential for uplift forces to cause flexural loading on the pavement slab needs to be evaluated if that slab is being used as a strut to provide lateral restraint for the base of the walls.

6.4.5 Station 334+00 to Station 356+00 (East Portals to 700 ft West of Hillcrest Road)

Critical design earth pressures in this section of Segment A for long-term loading are likely to be caused by the weathered and fresh Austin Chalk. These materials can potentially stand vertically for much of their exposure, but also have the potential for developing unstable rock wedges at intersecting discontinuities. Accordingly it is recommended that a design relying either on a rock reinforcement program based on more detailed discontinuity data, or an envelope of wall pressure based on $K_o$ be adopted. Using a $K_o$ of 0.22 based on a friction angle of 40 degrees, the resulting equivalent fluid unit weight from the soil would be 16.9 pcf. The equivalent fluid pressure from the rock at any depth can then be determined by multiplying the equivalent unit weight by the depth. In addition, for design purposes, the groundwater level along this section should be considered at or near existing grade and the appropriate hydrostatic pressure should also be included in the design loading.

6.4.6 Station 356+00 to End Below Grade Sections Station 360+00

With respect to earth pressures, the material in this section of Segment A with the potential to produce the largest long-term loading is the high plasticity alluvial clay. Using a $K_o$ of 0.78 based on an effective friction angle of 13 degrees, the resulting equivalent fluid unit weight from the soil would be 51.2 pcf. The equivalent fluid pressure from the soil at any depth can then be determined by multiplying the equivalent unit weight by the depth. For design purposes, the groundwater level along this section should be considered at existing grade and the appropriate hydrostatic pressure should also be included in the design loading.

In addition, due to the highly plastic nature of the alluvial soils, the impact of swelling on lateral loadings should be evaluated for this section in light of the Developer's design configuration and planned means and methods, and the appropriate additional lateral swell pressure should be included in the design.
<table>
<thead>
<tr>
<th>Station</th>
<th>Typical Description</th>
<th>Borings (Example Boring)</th>
<th>Generalized Subsurface Conditions</th>
<th>Total Unit Weight</th>
<th>Buoyant Unit Weight</th>
<th>Earth Pressure Coefficient for Preliminary Design</th>
<th>Rationale</th>
<th>Equivalent Fluid Pressure</th>
<th>Water Height</th>
<th>Surcharge</th>
<th>Lateral Pressure due to Surcharge</th>
<th>Passive Earth Pressure Coefficient, ( K_p )</th>
<th>Rationale</th>
</tr>
</thead>
<tbody>
<tr>
<td>67+00 to 92+00</td>
<td>Josey Lane to Webb Chapel Road</td>
<td>TUN-14 thru BW9R (BW8R)</td>
<td>Fill down to 6 ft, then medium stiff to very stiff clayey alluvium (CH), with occasional sand and/or gravel zones up to 2 ft thick, down to top of weathered EFS at approximately 60 ft</td>
<td>128pcf</td>
<td>65.6pcf</td>
<td>( K_n = 0.78 )</td>
<td>Based on ( \theta' = 13^\circ ) for fill, alluvium and residual soils equal to 13(^\circ) (Chapter 6.2.1)</td>
<td>( (K \times \gamma' \times H) )</td>
<td>Full wall height</td>
<td>600psf</td>
<td>240 psf down to 10 ft, then 100 psf for next 10 ft</td>
<td>( K_n = 1.58 )</td>
<td>Based on ( \theta' = 13^\circ ) for fill, alluvium or residual soil from EFS</td>
</tr>
<tr>
<td>92+00 to 130+00</td>
<td>Webb Chapel Road to 1,800 ft West of Marsh Lane</td>
<td>BW7L thru BW21R (BW13L)</td>
<td>Fill down to 7 ft, then stiff to very stiff clay, residual soil derived from EFS (CH), down to approximately 25 ft, then weathered EFS down to fresh EFS at approximately 50 ft</td>
<td>125pcf</td>
<td>62.6pcf</td>
<td>( K_n = 0.78 )</td>
<td>See discussion of AC to right.</td>
<td>( 51.2 \times H ) psi</td>
<td>Full wall height</td>
<td>600psf</td>
<td>240 psf down to 10 ft, then 100 psf for next 10 ft</td>
<td>( K_n = 1.58 )</td>
<td>Based on ( \theta' = 13^\circ ) for fill, alluvium or residual soil from EFS</td>
</tr>
<tr>
<td>130+00 to 163+00</td>
<td>1,800 ft West of Marsh Lane to East of Marsh Lane</td>
<td>BW17L thru BW24R (BW19L)</td>
<td>Fill down to 3 ft, then 24 ft of weathered EFS above fresh EFS</td>
<td>129pcf</td>
<td>66.6pcf</td>
<td>( K_n = 0.78 )</td>
<td>See discussion of EFS to left.</td>
<td>( 48.8 \times H ) psi</td>
<td>Full wall height</td>
<td>600psf</td>
<td>240 psf down to 10 ft, then 100 psf for next 10 ft</td>
<td>( K_n = 1.58 )</td>
<td>Based on ( \theta' = 13^\circ ) for fill, alluvium or residual soil from EFS</td>
</tr>
<tr>
<td>163+00 to 191+00/195+00</td>
<td>East Mined Tunnel Portals to 700 ft West of Hillcrest Road</td>
<td>BW25L thru T3L (BW23L)</td>
<td>Fill down to 5 ft, then 5 to 15 feet of weathered AC above fresh AC. Fresh AC down to top of fresh EFS at approximately 30 ft</td>
<td>139pcf</td>
<td>76.6pcf</td>
<td>( K_n = 0.022 ) in AC</td>
<td>AC can stand vertically unsupported. ( \theta' ) for the AC is assumed to be equal to 40(^\circ)</td>
<td>( 51.9 \times H ) psi</td>
<td>Full wall height</td>
<td>600psf</td>
<td>240 psf down to 10 ft, then 100 psf for next 10 ft</td>
<td>( K_n = 1.58 )</td>
<td>Based on ( \theta' = 13^\circ ) for fill, alluvium or residual soil from EFS</td>
</tr>
<tr>
<td>334+00 to 356+00</td>
<td>334+00 to 360+00</td>
<td>BE4R thru BE9C (BE6R)</td>
<td>Fill to 2 ft, then 3 ft of weathered AC over fresh AC down to a minimum of 50 ft</td>
<td>139pcf</td>
<td>76.6pcf</td>
<td>( K_n = 0.22 ) in AC</td>
<td>See discussion of AC to right.</td>
<td>( 16.9 \times H ) psi in AC</td>
<td>Full wall height</td>
<td>600psf</td>
<td>240 psf down to 10 ft, then 100 psf for next 10 ft</td>
<td>( K_n = 1.58 )</td>
<td>Based on ( \theta' = 13^\circ ) for fill, alluvium or residual soil from EFS</td>
</tr>
<tr>
<td>356+00 to 360+00</td>
<td>End of Below Grade Sections</td>
<td>BE10C (BE16C)</td>
<td>Existing roadway over clay fill to a depth of 12 ft, then very stiff to hard clayey alluvium (CH) down to top of fresh AC at approximately 33 ft</td>
<td>128pcf</td>
<td>65.6pcf</td>
<td>( K_n = 0.78 )</td>
<td>Based on ( \theta' = 13^\circ ) for fill, alluvium or residual soil from EFS</td>
<td>( 51.2 \times H ) psi</td>
<td>Full wall height</td>
<td>600psf</td>
<td>240 psf down to 10 ft, then 100 psf for next 10 ft</td>
<td>( K_n = 1.58 )</td>
<td>Based on ( \theta' = 13^\circ ) for fill, alluvium or residual soil from EFS</td>
</tr>
</tbody>
</table>

Notes: 1) Above-described earth pressures do not account for additional lateral loading due to expansive clays or clay shales 2) Compare to appropriate AASHTO surcharge depending on final project configuration
7.0 CONSTRUCTION CONSIDERATIONS FOR MINED TUNNELS

7.1 Mechanical Excavation

It is anticipated that the mined tunnels and associated underground structures will be excavated using mechanical means. The massive, relatively soft Austin Chalk lends itself well to mechanical excavation, specifically by roadheaders equipped with either an axial or transverse type cutterhead and carbide bits, or by a tunnel boring machine (TBM) equipped with disc cutters. Mined tunnel excavation by drill and blast methods will not be permitted.

Based on the mined tunnel configuration depicted in the Reference Schematic, it is anticipated that the underground openings will be excavated primarily by roadheader, and will be conducted sequentially using multiple headings and drifts. With this approach, the excavation must be coordinated with observations of ground response, geologic mapping and instrumentation monitoring to ensure that excessive deformations that could ultimately lead to drift collapse or potential surface settlement are prevented.

An estimate of instantaneous cutting rates for roadheaders operating in the Austin Chalk was performed by the Earth Mechanics Institute in Golden, Colorado, and is presented in the Phase 2 GDR (Fugro 2005). The estimate does not take into account machine availability or utilization, and thus the Developer should estimate his overall production in light of his planned means and methods at a minimum considering the sequencing of excavation, ground support requirements, muck removal and handling, etc.

7.2 Muck Handling

For roadheader-excavated openings, muck handling operations may vary throughout construction and will depend largely on the Developers approach and sequencing. For smaller drifts, muck removal may be performed using LHDs (load-haul-dump underground mining vehicles), which are very efficient in tight spaces and when relatively short haulage lengths are required (i.e., less than 1,500 feet). For larger drifts, muck removal may be performed via a short conveyor on the roadheader dumping directly into typical 10-wheel highway end dump trucks, or may be by rubber tired or tracked front-end loaders loading into standard highway rock haulage belly dump or end dump trucks modified for underground use.

During muck removal, the Developer must consider the effects of loader and haulage traffic on argillaceous and bentonitic material exposed in the invert of underground excavations. If these materials become wet due to groundwater inflows or from water used in construction processes, they will have a tendency to become slick and/or sticky and hamper trafficability in the invert.

In contrast, muck handling for TBM excavated tunnels will likely be by conveyor. The conveyor will be affixed to the wall of the tunnel, and muck handling operations will not be impacted by or impact trafficability issues in the tunnel invert. Current technology for long conveyance projects includes the use of appropriately placed booster drives that eliminate transfer points within the tunnel. Rail facilities however, may still be required.
for personnel transport and for servicing the heading with materials, supplies, etc., and invert trafficability concerns are warranted.

7.3 Groundwater and Seepage Control (Mined Tunnel Construction)

Based on previous experience with underground excavations in the Austin Chalk, and due to the low hydraulic conductivity of the intact limestone, it is anticipated that much of the mined tunnels will be essentially dry during construction. Where groundwater inflows are present they may generally occur as flush flows from fracture zones. The inflow rates at these locations may be significant for short periods of time (e.g., 100 gpm per 1,000 linear feet of underground excavation) but will tend to dissipate to very small inflows or no flow within 12 to 24 hours. Depending on the connectivity of these fracture zones to potential recharge from surficial recharge zones, it is possible that renewed inflows may be experienced during or following periods of precipitation. Similarly, where the tunnel profiles rise at each portal location and the fresh limestone cover is thinner, recharge of fractured zones resulting in periodic or steady long-term inflows is more likely.

In addition, over time the mined tunnels and associated underground structures will in essence become large drains that will result in significant lowering of the groundwater table along the Segment A alignment. Construction processes must be developed to accommodate and handle this water so as to mitigate potential detrimental effects both to the ongoing construction effort and to the structures and facilities adjacent to Segment A.

7.4 Geotechnical Instrumentation and Monitoring

Successful projects utilizing the likely construction methods in urban areas typically incorporate geotechnical instrumentation and monitoring programs into the construction effort. These programs are necessary to monitor ground response and tunneling performance, to protect existing structural facilities and utilities, to evaluate loads on the initial ground support and final liner, and to detect potential problems arising from underground construction operations. Effective instrumentation programs include proper instrument selection, identification of the locations and depths of instruments to be installed, and a monitoring and response schedule tied to the progress of tunnel excavations. In addition, for the observational method of excavation anticipated for the mined tunnels and associated underground structures along Segment A, excavation configuration and sequence and initial ground support type are often modified to suite the observed ground conditions and response. As such, it is vital that monitoring efforts be conducted in a disciplined and expedient manner, and that information be recorded and digested quickly for timely dissemination to tunnel construction personnel.
8.0 CONSTRUCTION CONSIDERATIONS FOR CUT-AND-COVER TUNNEL AND U-WALL SECTIONS

8.1 Temporary Excavation Support

Based on historic construction experiences in the Dallas metro area, the principal construction challenges relating to temporary excavations along Segment A are:

- Potential for a sudden cut slope failure in the weathered or fresh Eagle Ford Shale or its residual soil;
- Potential for large slabs of Austin Chalk breaking loose and sliding out along pre-existing discontinuities, also capable of occurring with little warning; and
- Potential for a block translational slide on a horizontal or shallow dipping zone in the Eagle Ford Shale, in one of the many bentonite seams in the Austin Chalk, or in the Fishbed Conglomerate at the base of the Austin Chalk.

Depending upon the Developer’s construction sequencing, temporary excavation support may be required at various locations along Segment A. Since the construction sequence, methods, wall types, etc. are not known at this time, no attempt has been made to develop specific design pressure envelopes for the wide range of possible wall types and construction methods. There may however be circumstances or locations where the small amount of lateral movement needed to mobilize the active earth pressure on walls can be tolerated during construction. In those locations, design of temporary excavation support can be based on customary trapezoidal pressure envelopes (Terzaghi and Peck 1967) or on active earth pressures in conjunction with the hydrostatic pressure expected during construction.

This approach could be used throughout Segment A, but should be tempered with caution when utilized for designing temporary excavation support in the Eagle Ford Shale, its residual soil, or colluvial deposits derived from the Eagle Ford Shale. For these materials, the high cohesion values indicated by unconfined compression tests or the Texas Cone Penetrometer and SPT blow counts have not been a good indicator of stability or of the freestanding height assessment, and sudden failures in these materials have occurred. Rather, as a general guideline, excavations in these materials should be analyzed on a case basis, as these can fail slowly by creep or quickly in a rotational slide (Allen and Flanigan 1986).

Other approaches are available to the Developer for stabilizing and supporting temporary excavations. For overburden soils, these may include flattening the excavation, benching the excavation back at shallower heights or using soil nailing techniques with shotcrete. These techniques should also be limited to walls of low to moderate height, typically less than 15 feet. For soil cuts of greater depth, conventional bracing or temporary MSE walls should also be considered.

In addition, for vertical excavations in the weathered and fresh Austin Chalk the customary approach of using rock reinforcement to stabilize the excavation for the long-term condition is equally applicable to the temporary or short-term construction loading condition. In that case, rock reinforcement length and spacing may be set during design for an assumed spacing of discontinuities. Then during construction, the actual
conditions should be examined once the rock surfaces are exposed, and additional or longer rock reinforcement may be installed as needed.

8.2 Temporary Excavation Support; Offsite Impacts

The overall construction logistics and methodology planned by the Developer must be considered in design of temporary excavation support systems. This includes protection against damaging movements of existing facilities within or adjacent to the Project right-of-way, such as adjacent roadways, frontage roads, bridges, ramps, utilities, buildings, or other structures. Also required is appropriate consideration of surcharge loadings from typical construction activities as well as any severe loading conditions that could be caused by stockpiling of materials adjacent to excavations or due to loadings from construction equipment working near walls or excavation side slopes. Temporary support or underpinning may be required to adequately protect existing facilities, depending on the proposed facilities design and associated construction sequence and means and methods.

8.3 Excavation Equipment and Methodology

The Developer should consider the soil and rock properties summarized in this report when selecting the equipment and methodology for excavation of the considerable volumes of earth materials to be encountered on this Project.

Based on previous projects, it is anticipated that excavation in the alluvial soils and the residual clay soils derived from Eagle Ford Shale and Austin Chalk will be carried out using conventional methods including hydraulic excavators, dozers, scrapers and front-end loaders. Excavation in the weathered Eagle Ford Shale and weathered Austin Chalk may also be done using hydraulic excavators or dozers with ripper attachments. For excavations in the weathered Austin Chalk, hydraulic excavators may require buckets fitted with special rock teeth, and dozers with pneumatic rippers may be required in the more competent zones of the weathered Austin Chalk.

Excavations in the fresh Eagle Ford Shale will probably require the use of larger dozers with ripper attachments and possibly equipped with pneumatic rippers. Hoe rams (i.e., hydraulic or pneumatic hammers) may also be required for stronger zones. These techniques may be feasible in the fresh Austin Chalk also; however, there may be significant zones that are not rippable and that may require alternative mechanical excavation methods to efficiently excavate. Consequently, due to the massive nature of the fresh Austin Chalk, mechanical excavators such as pavement milling machines have also been utilized to excavate fresh Austin Chalk. Though milling machines could also be considered for use in the Eagle Ford Shale, attention should be given to the concretions and thin limestone beds that are present in the Eagle Ford Shale, which can be harder, stronger and more cemented than the Austin Chalk. In addition, due to the high clay content of the shale, the potential for clogging or trafficability problems with these machines may exist depending on the amount of water present in the bottom of the excavation. Finally, due to the tendency for the Eagle Ford Shale to degrade quickly when exposed to variable moisture conditions, the scheduling and planning of excavation work, including how long excavations are left open, and the attention paid to
grading and directing groundwater inflows and rainwater runoff to sumps for removal, become important considerations for excavations in this material.

8.4 Compaction Control

While recommendations for embankment fill compaction or roadway subgrade compaction are not within the scope of this GIR, there may be instances along Segment A where compaction is required adjacent to or behind the U-walls or retaining structures that are discussed herein. In these cases, where compaction is required near or adjacent to a wall, the equipment selection and compaction procedures must ensure achievement of the required degree of compaction, but must also avoid application of excessive lateral pressures to existing structures due to over-compaction.

8.5 Right-of-Way Issues

For the roadway geometry depicted in the Reference Schematic, the ultimate roadway section consumes the entire right-of-way in most locations along Segment A. Staging of materials and construction activities will need to take these limitations into account. During preliminary design, wall designs utilizing tiebacks that would have to extend outside the right-of-way, or MSE walls systems that required overexcavation were not evaluated due to these limitations. However, the flexibility the Developer now has in determining the final Project configuration may allow for these or other wall construction methods to be utilized.

8.6 Construction Access

It is anticipated that there may be requirements to minimize impacts on existing roadway capacity along Segment A during construction. Consequently, construction access issues will be significant and will require detailed planning during final design.

8.7 Groundwater Inflow

Groundwater inflow and effects during construction of cut-and-cover tunnel and U-wall sections will be consistent with the mechanics of inflow as described in Chapter 7 for mined tunnels. However, the flow contributions from the weathered rock zones at the top of the rock column and from the permeable soil layers above due to the increased hydraulic conductivity may be considerable. In these areas, the excavations are likely to intercept water-bearing permeable soils that could direct large quantities of water into the excavations during and after precipitation events. This may necessitate the application of additional groundwater control systems to intercept this flow and/or to handle and remove it if it enters the work area.

The Developer's selected construction sequence and site drainage control can also significantly influence the amount of surface water that can flow into excavations, potentially causing deleterious effects as well as increasing quantities of water to be pumped and disposed of in an environmentally acceptable manner (ensuring sediment removal).
8.7.1 Flow From Soils in the Cut-and-Cover Tunnel and U-wall Sections

Prediction of groundwater inflow rates from soils depends on the thickness of the permeable soil layers, the respective hydraulic conductivity values, the groundwater head differences, and on the specific geologic and topographic conditions that will control the ultimate supply of groundwater. Variability in the thickness, consistency and persistence of the different soil strata along Segment A complicate this prediction. Similarly, the potential for encountering buried streams and their impacts on inflows must be considered. During final design, the Developer will have to perform groundwater inflow assessments for soils based on the specific facilities to be constructed, their position within the Project geologic environment, and the planned construction sequence and methodology.

8.7.2 Fracture Flow in the Cut-and-Cover Tunnel and U-wall Sections

For the cut-and-cover tunnel and U-wall sections, potential groundwater inflows in the weathered Austin Chalk may be several orders of magnitude higher than the inflows anticipated through fractures in the fresh Austin Chalk as described in Chapter 7. Furthermore, it is possible that these inflows may persist for substantial periods of time due to their shallow location and potential direct connection to the overlying soil strata.

In the Eagle Ford Shale, fracture flow is also likely to exist. However, due to the nature of the shale and its ability to heal itself by swelling, fractures may be less open and flows correspondingly less than in the Austin Chalk.
APPENDIX A - Overview of Swelling Soils and Clay Shales

A.1 Theoretical Considerations and Laboratory Indicators of Swell Potential

In general, the potential for the residual and clayey alluvial soils and clay shales along Segment A to exhibit swelling behavior is dependent on two considerations. The first relates to the physical and mineralogical makeup of the deposit, in particular the amount and type of clay minerals in the soil. For example, higher percentages of clay, and specifically higher percentages of montmorillonite clay, generally suggest higher swelling potential. These parameters are indirectly reflected in the relatively inexpensive Plasticity Index and the Activity Index tests. As such, these tests can be used to assess the potential for swelling, setting aside for the moment the soil’s initial or actual moisture content (Van Der Merwe 1964, and Holtz and Gibbs 1954).

The second consideration is the existing in situ moisture content, and the likelihood that the soil will be exposed to changes in moisture as a result of the proposed construction. If the residual soil or clay shale has high swelling potential and currently exists at a moisture content well below the liquid limit, and excavation or construction creates a situation where the material is exposed to water that it can then absorb, the potential for large swell or large swelling pressures exists. Alternatively, if the clay has high swelling potential but is already at a relatively high moisture content (essentially, has already swelled), and excavation or construction presents little possibility for the material to become exposed to water, then the swelling volumes and pressures will be light and probably not a serious design concern. However, to confirm that these mitigating conditions actually exist at a particular location in the field could require site specific field data, either before or during construction.

Several laboratory tests are used to define the problem, both qualitatively, in the sense of identifying soils with high swell potential, and quantitatively, in regard to predicting the actual heave in unrestrained situations or the swell pressure developed in restrained situations.

Simple and inexpensive laboratory tests commonly cited in the literature to identify the potential for swelling include the Atterberg limits tests (i.e., liquid limit and plastic limit), the activity index and the liquidity index. The activity index is defined as follows:

\[ A_c = \frac{PI}{\% \text{ clay fraction } (\sim 2\mu)} \]

Where:  
\( A_c \) = Activity index  
\( PI \) = Plasticity index (liquid limit – plastic limit)

The liquidity index is defined below and is simply an expression of the in situ moisture content of a soil in comparison to the range from it’s relatively dry condition (plastic limit) and it’s very wet condition (liquid limit). If a soil or clay shale exists at a moisture content well below the plastic limit, such as at the shrinkage limit, which is quite common in natural clay deposits in the Dallas area, the potential for swell upon exposure to water is the greatest.
Where: \( LI = \frac{w - PL}{PI} \)

- \( LI \) = Liquidity index
- \( w \) = Moisture content in percent (dry weight basis)
- \( PL \) = Plastic limit (%)
- \( PI \) = Plasticity index (%)

It should be noted that if the index tests described above are intended to be used to classify swelling potential of clay shales, the tests need to be conducted in accordance with procedures that adequately disaggregate the clay particles. The Corps of Engineers requires clay shale samples for liquid limit tests be thoroughly mixed in a high-speed blender before proceeding with the test. (This step is referred to as “blenderizing” in the USACOE Laboratory Soils Testing Manual, EM 1110-2-1906). It is an additional step not commonly practiced when determining the liquid limit of clay soils, but it has the potential of producing higher liquid limit results, which implies higher plasticity indices, and a higher activity index. For the indices to have meaning in the context of observations concerning swelling soils, they need to be based on liquid limit samples prepared in this manner.

A more direct measure of swell is the “simple swell” or “free swell” test, conducted on an undisturbed sample loaded into an oedometer (also referred to as a consolidometer, the same device customarily used to measure compressibility of soils). After loading the soil or clay shale sample into the device, a light seating-load is applied, the sample is then submerged and allowed to absorb water and swell. The volume change when the swelling has stopped is then recorded and is typically expressed as a percent of original sample height.

Another “free swell” test developed in the 1950s is mentioned here only for reference and to ensure clarity. This version of the “free swell” test involved simply dropping 10 grams of finely ground clay sample into water in a graduated cylinder and observing the volume change in the clay sediment that had fallen to the bottom of the flask, typically over a 24 hour periods. This test gave a qualitative indicator of swell potential. Although it was a very inexpensive test, it was recognized as early as 1957 to be unreliable for residual soils and clay shales and in Texas (Krynine and Judd 1957). In this GIR, the term “free-swell” test will be limited to the test performed in a consolidometer.

Estimating swell pressure in confined conditions requires a more complex and time consuming variation of the free swell test that is conducted in a laboratory consolidometer or oedometer. In this test, referred to in ASTM D 4546 as “Method A”, the sample is placed in a loading frame, seated with a nominal load, and then inundated to allow the sample to absorb water. The vertical swell under this nominal seating load is monitored, and when swelling has ceased, the total change in height is measured and reported as percent heave. The sample is then consolidated under incremental loading until it returns to its original void ratio. From the curve of void ratio versus log of pressure, the pressure at this point is deemed the swell pressure.

Another variation of the swell test carried out in a consolidometer begins as described above, but rather than allow the sample to swell, the expansion is restrained by applying enough vertical pressure to hold the strain close to zero, continuously adding load
increments as necessary to prevent swell. This method is described in ASTM D 4546 as a “Method C” swell test, and produces an estimate of the swell pressure that the soil can produce.

For both tests, quantitative prediction of swell remains a difficult exercise, because it requires reliable knowledge of the initial state of stress, the changes to it, and the initial and final moisture contents. To be most effective, testing needs to be customized to the stress states likely to be imposed during and following construction, including the effects of construction sequence, drainage conditions, etc.

The Corps of Engineers has a similar procedure for determining swelling pressure in clay shales, as described in EM 1110-2-1906 (1986).

Most laboratory tests and most design protocols deal with vertical swell. Lateral swell is of more interest with respect to the design of vertical retaining walls or walls in cut-and-cover sections. There is some justification for the intuitive belief that lateral swell will be a fraction of vertical swell, but the limited discussion of this topic in the literature is inconclusive. A USACOE Waterways Experiment Station (WES) investigator (Banks 1971) appeared to confirm intuition, and found that for certain clay shales, the swell pressure measured parallel to the bedding was less than that measured normal to the bedding. Other tests performed by the USACOE WES concluded that while elastic deformation of soil may be anisotropic, the swell pressures under a “no volume change” condition appear to be isotropic (Johnson 1987).

Swell tests that measure horizontal pressures are often expensive and still remain somewhat experimental (Ofer 1981). Therefore, at this point it may be safer to assume that horizontal swell pressures can be the same as vertical swell pressures under the same field conditions.

A.2 Practical Considerations

The potential for swelling clays to exert additional lateral pressures on retaining walls is generally mentioned in the text of geotechnical reports for highways and subways constructed in the Dallas area. However, the earth pressures used for design taken from the same reports generally do not include calculation of any additional pressures that may develop due to swelling clays being exposed to moisture changes. Typically, earth pressures for design are based on $K_a$ or $K_o$, depending on whether the wall is perceived to be yielding or rigid. The higher pressures are associated with the $K_o$ assumption, and typically use an “equivalent fluid pressure” approach with the “equivalent fluid” weight taken as about 75 pcf. For a wall 20 feet in height, this would produce a lateral pressure at the base of about 1,500 psf.

For comparison, in 1968 the Corps of Engineers used buried load cells to measure lateral swell pressures in an expansive clay shale of the Midway Group, in San Antonio, Texas. The clay shale was deliberately wetted and a sustained program of flooding the material was maintained. Over a one-year period, the lateral swell pressure increased to 9,500 psf at a 15 feet of depth. Similarly, measured swell pressures against a rigid basement wall in an expansive clay shale in Australia (Richards 1977) were found to be over 10,000 psf at a depth of 20 feet. These pressures developed over a period of 4
years. In general, these examples suggest that if swelling pressures do develop, they may greatly exceed the customary design pressures and may take years to develop.

A body of practice has developed over the years in the Dallas area to take into account and control the potential for vertical foundation or floor heave in those areas of the Dallas metroplex that are known to include soils of high swell potential. Similarly, the highway construction industry has developed a body of practice to deal with swelling clays in pavement subgrade, for example including lime treatment in some cases, and excavation/replacement for a limited depth in others, or both. It appears however, that the issue of lateral loads on retaining walls due to swelling soils has yet to be formalized into a set of generally accepted practices. Case histories of wall failures attributable to swelling clays have not been well documented, even in the Dallas-Fort Worth metroplex where swelling soils are known to be a concern at least with regard to building foundations and pavement heave. This lack of data may indicate that there are not enough high walls (greater than 20 ft) in expansive clays to create an experience record, or that swelling soils behind these walls are not being exposed to water, or that other factors in the designs of these walls may be accommodating these higher pressures.
APPENDIX B - REFERENCES


Fugro South, Inc. (2004a). “Phase 1 Geotechnical Data Report, IH-635 (LBJ Freeway) Corridor, Section 4-West.”
Fugro Consultants LP (2004b). “Phase 1 Geotechnical Baseline Report, IH-635 (LBJ Freeway) Corridor, Section 4-West.”

Fugro Consultants LP (2005). “Phase 2 Geotechnical Data Report, IH-635 (LBJ Freeway) Corridor, Section 4-West.”


LBJMP (2005). "Reference Schematic, IH-635, From: West of IH-635 To: West of Coit Road", Texas Department of Transportation, Dallas County, Dallas, Texas.


TxDOT (2002). Plans of Proposed State Highway Improvement, I.H. 30, Dallas, County, From: East of Loop 12, To: East of Sylvan Avenue, Texas Department of Transportation, Dallas County, Dallas, Texas.

TxDOT (2004). “Environmental Assessment, Interstate Highway (IH) 635 From: Luna Road, To: US 75, Dallas County”, Texas Department of Transportation.


USGS (1959b). "Carrollton 7.5 Minute Topographic Quadrangle", United States Geological Survey, 1:24,000 scale, 1 sheet.


APPENDIX C - FIGURES