PHASE 1 GEOTECHNICAL BASELINE REPORT

IH-635 (LBJ FREEWAY) CORRIDOR, SECTION 4-WEST
CONTRACT 18-2XXP0004
DALLAS, TEXAS

PROJECT NO. 0703-8100

Report to

TEXAS DEPARTMENT OF TRANSPORTATION
DALLAS, TEXAS

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Dear Mr. Aramoon:

Submitted herewith is the Phase 1 Geotechnical Baseline Report for the proposed improvements to LBJ Freeway. In brief, the report includes a description of the project, a description of the site conditions, a characterization of the groundwater conditions for the proposed Tunnel and Managed Box sections, and recommendations for additional investigations.

Fugro Consultants LP and Brierley Associates, LLC appreciate the opportunity to provide these engineering services to TxDOT. In the meantime, please feel free to contact us if you have any questions about information in this report. We look forward to working with you on future project phases.

Very truly yours,

FUGRO CONSULTANTS LP
Hugh T. Kelly, P.E.
Senior Geological Engineer

Saad M. Hineidi, P.E.
Senior Vice President

HTK/SMH/kp
Copies Submitted. (7)
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1.0 INTRODUCTION

1.1 Project Overview

The Texas Department of Transportation (TxDOT) is planning a number of significant improvements to the existing IH-635 (LBJ) Freeway, in the northern metropolitan Dallas area, as part of a major expansion to increase traffic capacity and to mitigate numerous concerns over safety, noise, air quality, operation, and maintenance of this congested freeway. Since April of 1993, TxDOT has coordinated study efforts for the corridor improvements with the cooperation and involvement of the North Central Texas Council of Governments (NCTCOG), the Dallas Area Rapid Transit (DART), the North Texas Tollway Authority (NTTA), and all local communities. The LBJ Freeway is located north and east of Dallas extending approximately 40 miles from SH-121 to IH-20. The LBJ Corridor portion of the project has been broken into four sections:

US 75 • Interchange (Dallas High Five) – Construct a 5-level interchange with frontage roads and HOV facilities.

IH-30 to US 80 • Mesquite – Add ramps to Towne Centre Drive, replace bridges, and add auxiliary lanes (Phase I is underway).

US 75 to IH-30 • East – Improve with 10 main lanes, HOV/Toll lanes, and continuous frontage roads. This includes the IH-30 Interchange.

IH-35 to US 75 • West – Improve with 8 main lanes, 6 HOV/Toll lanes, and continuous frontage roads. This includes the IH-35E Interchange.

This Phase 1 Geotechnical Baseline Report (GBR) includes discussions for the eastbound and westbound mined tunnels, the adjacent cut-and-cover tunnels, and the east and west managed box sections within the LBJ Section 4-West. To help mitigate community concerns with regard to right-of-way issues, noise, and traffic disruption, TxDOT is proposing to construct two mined tunnels (1.5 and 1.9 miles long) between Midway Road and Preston Road. The tunnels will connect the Dallas High Five Interchange in the east with box sections and new frontage roads in the west. Design of the Section 4-West improvements will be performed under the direction of a Procurement Engineering Team (PcE) and implemented by a Comprehensive Development Agreement (CDA) team. Geotechnical investigations were performed as a collaborative effort between Fugro Consultants LP (Fugro) of Dallas, Texas, and Brierley Associates, LLC (Brierley) of Littleton, Colorado.

1.2 Purpose and Scope

The purpose of this Phase 1 GBR is to provide a description of anticipated ground conditions for design and construction of surface and subsurface facilities associated with the proposed Section 4-West. The project is located in the northern Dallas metropolitan area as shown on Figure 1-1, in the Illustrations. Discussions contained herein represent a geotechnical basis of design for mined tunnels, portals, box
structures, depressed roadways, and other project components. This report and the Phase I Geotechnical Data Report (GDR), taken together, are intended to assist the PcE in planning the Tunnel Project and for evaluating the requirements for excavation, shoring, dewatering, and tunneling activities needed to accomplish the work.

Site conditions are discussed in Section 2 with respect to physiography and land use, regional geologic setting, subsurface conditions, and seismic considerations. Subsurface conditions are described in terms of the soil, bedrock, and groundwater conditions that occur above, within, and just below the proposed construction zone. Following a discussion of site conditions, Sections 3 and 4 provide a review of anticipated ground conditions. Sections 5 and 6 discuss design considerations for the tunnel and managed box sections, respectively. Sections 7 and 8 provide discussions of the construction considerations for the tunnel and managed box sections. Construction considerations include excavation, ground stabilization, groundwater control, and construction monitoring and instrumentation, as appropriate. Section 9 provides a brief summary of recommendations for additional geotechnical investigations for subsequent phases of this project.

1.3 Available Information

Three site-wide documents have been prepared for the Section 4-West Project. The first document is entitled “Geotechnical Data Report, LBJ Corridor Study Project, Dallas, Texas,” dated December 29, 1998, prepared by Terra-Mar, Inc. This data report summarized boring log data, rock descriptions, and laboratory test results accumulated from fourteen (14) planning level borings carried out along the entire project alignment. Of those fourteen (14) test borings, six (6) borings were drilled in the vicinity of the proposed tunnels.

The second document is a set of schematic plans entitled “Project Plan and Profile Drawings, TxDOT, Design Schematic,” dated March 2002, prepared by HNTB in association with Turner Collie & Braden, Inc. This design schematic was supported by a report prepared by Lachel & Associates entitled “Preliminary Geotechnical Design Basis and Tunnel Design, Cost and Constructability Report,” dated March 2002. Those two documents, taken together, provided the design details utilized as the basis for development of this Phase 1 GDR.

The third site-wide document is the “Phase I-Geotechnical Data Report, IH-635 (LBJ Freeway) – West Section Managed HOV Lanes Tunnel Project, Dallas, Texas,” dated April 2004, prepared by Fugro Consultants LP and Brierley Associates, LLC. From July 14 to October 10, 2003, Phase I of a site-wide subsurface exploration program consisting of 93 test borings was conducted for the Section 4-West Project; the results of which were described in Phase I-GDR.

Test borings for the Phase I investigations were located within the TxDOT IH-635 right-of-way along the existing freeway route approximately between Josey Lane on the west to Hillcrest Road on the east, at a typical spacing of approximately 400 ft. Borings drilled along the project alignment were situated on the shoulders of the eastbound and westbound lanes, the center high-speed travel lanes, and adjacent to service roads and
ramps, to minimize interference with traffic. The Phase I borings were drilled to depths of 45 to 226 ft. below the ground surface using truck-mounted CME-55 and CME-75 drilling rigs.

Additional details relating to results of subsurface investigations are presented in Section 2.0 of this report and in the Phase I-GDR. It is assumed, therefore, that anyone reading this document is also familiar with the subsurface information provided in the Phase I-GDR.

1.4 Limitations

All discussions provided herein are based on subsurface information as contained in the Phase I-GDR. When additional site-specific subsurface information becomes available, it is anticipated that supplements to the GDR will be published and it will be necessary to review those supplements in order to obtain a complete understanding of geotechnical considerations for this project. Hence, this document will be replaced as the final layout for the proposed tunnels is developed and as additional subsurface information is obtained in the field. Future editions of the GDR for the LBJ Section 4-West are anticipated as future design documents. It is also noted that this report contains no reference to any environmental issues, such as potential groundwater contamination issues, associated with this project.

2.0 SITE CONDITIONS

2.1 General

This Section of the Phase 1 GBR provides a summary of geotechnical investigations performed for the project followed by discussions of the physiography, regional geology, and anticipated soil, rock, and groundwater conditions. A more detailed discussion of geotechnical data collected during the subsurface investigations is presented in the Phase I-GDR and reference should be made to the Glossary of Terms in Appendix A of the GDR for definitions of some of the technical terms used in the discussions that follow. A preliminary characterization of the ground as it relates to tunneling is presented in Section 3 of this report.

From July 14 to October 10, 2003, 93 test borings were drilled during the Phase I subsurface exploration program. The location of these borings with respect to the entire Section 4-West project area is shown in Figure 2-1, in the Illustrations, Plan of Borings. Details of the subsurface explorations, in situ and laboratory testing, and field instrumentation associated with this program were presented in a report entitled "Phase 1 - Geotechnical Data Report, IH-635 (LBJ Freeway), Section 4-West, Dallas, Texas" (Phase 1-GDR) dated April 6, 2004.

The borehole designation system established by TxDOT consists of alpha and numeric characters for each boring. Each boring number begins with BE, BW, or T. The BE and BW borings are generally for "non-tunnel" (managed box) borings (B) on the east (E)
and west (W) of the tunnel borings, respectively. The “tunnel” borings are designated with a T. Following these first letters is a numeral designation. The numbers increase from west to east, i.e., 10 is east of 5, etc (Up station). Borings with the same number are located at approximately the same station. The numbers are then followed by L (left-north), R (right-south), or C (center). These represent the boring position in reference to the project centerline, looking up station. Finally, a few 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.

An example is “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. There is no BE1C currently proposed.

The ground characterization for the tunnel section provided herein is based on field and laboratory test data available from the 45 test borings drilled in the immediate vicinity of the proposed tunnels. Table 2-1 lists the specific borings for the tunnel section considered during preparation of this report. The “BW” and “BE” borings were considered for the west and east Managed Box sections.

| Table 2-1: List of Borings Included in the Section 4-West Tunnel Project |
|-------------------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|
| T1R                    | T5R            | T9L            | T13L           | T18C           | T21C           | T25C           | T30C           | T34C           |
| T2L                    | T6L            | T10R           | T14C           | T19C           | T22C           | T26C           | T30L           | BE1L1          |
| T3R                    | T6R            | T11L           | T15C           | T19L           | T23C           | T27C           | T31C           | BE1R           |
| T4L                    | T7L            | T12R           | T16C           | T19R1          | T23R           | T28C           | T32C           | BE2L           |
| T4R                    | T8R            | T13C           | T17C           | T20C           | T24C           | T29C           | T33C           | BE2R           |

2.2 Physiography and Regional Geology

The entire Section 4-West of the LBJ Corridor Study area is approximately 8.3 miles long and follows the existing IH-635 (LBJ Freeway) from Luna Road on the west (located west of the IH-635/IH-35E intersection) to a point approximately 720 ft. west of the Coit Road overpass on the east. Within the central portion of the Section 4-West study area, the Tunnel Project area comprises approximately a two-mile-long section of the LBJ Corridor, situated approximately between Midway Road on the west and a point about 1000 ft. east of the Preston Road overpass on the east. All of this land is presently used for the existing freeway, and ancillary structures such as ramps, service roads, retaining walls, several overpass and underpass bridge structures, and includes buried utilities such as sewer, water, gas, and electrical. The entire length of the proposed alignment is also immediately adjacent to residential, retail, and commercial structures, including several high-occupancy buildings such as offices, hotels, and apartments.

Based on available topographic information, the land surface is generally flat to gently rolling. The highest elevation is approximately El 644 ft in the vicinity of the proposed tunnels between Montfort Drive and Preston Road. From this point, the ground surface
slopes downward toward the east to El 545± near Hillcrest and toward the west to El 500± near Josey Lane. Isolated low areas along the tunnel alignment include the underpasses at Josey Lane, Webb Chapel, Midway Road, the Dallas North Tollway (DNT), Montfort Drive, and Preston Road. There are no existing stream crossings within the area of the roadway alignment. There are large culverts at the DNT crossing which are localized low areas.

The City of Dallas (including the project area) is situated on the up-dip edge of the Gulf Coastal Plain near the northwestern limit of the East Texas Embayment of the Blackland Prairie Physiographic Province of Central Texas, and on the western limit of the Ouachita Folded Belt. From this vicinity, a wedge of Cretaceous age sediments unconformably overlie rocks of Paleozoic age which extend from the surface and gently dip and thicken to the east and southeast. These Cretaceous rocks comprise a sequence of lagoonal, shallow marine, and transgressive/regressive marine sediments. Portions of younger overlying sediments were eroded by the ancestral Trinity River exposing the underlying Austin Chalk and Eagle Ford Shale. Due to the erosional truncation of the southeasterly dipping rock outcrops, the Eagle Ford Shale is encountered in borings for the West Manage Box and in a few of the deeper borings drilled through the overlying Austin Chalk. The Eagle Ford Shale is not present within the tunnel envelope that is currently represented by the design schematics (see Ref 12).

In North Central Texas, the surficial geologic structure of relevance is also of Cretaceous age. The contacts between the formations are largely parallel and oriented approximately northsouth. Because these are horizontally deposited sedimentary rocks, primary structure such as bedding and volcanic ash layering (bentonite) formed before lithification. The formations lie in order of deposition, and dip gently southeast at an angle of about 1 to 3 degrees. Geologic reports cited by Allen et al (1986) also identify an east plunging anticlinal nose which is situated in northern Dallas in the general vicinity of the tunnel project area and which can be attributed to a remnant Paleozoic structure.

The tunnel project area is also situated approximately 25 miles east of the Balcones Fault Zone. This fault zone extends west of Uvalde, Texas to about 25 miles north of Mount Pleasant, turning due east near Greenville. East of the project vicinity, just east of Rockwall, Texas, the fault strikes at an angle of approximately 20 degrees east of north. The age of Balcones faulting is more than 5 million years and faulting in the Dallas area is generally the result of a combination of various regional tectonic events and fault systems. Both normal and reverse faulting, with displacements typically less than 15 ft. but as much as 70 ft., has been observed at the Superconducting Super Collider (SSC) project and the Addison Airport tunnel project.

Jointing is common throughout the region, although generally infrequent, with high joint frequencies usually limited to the areas of faulting. Joints are frequently found to be non-linear displaying a curved rather than planar surface. The geologic conditions that contribute to the observed jointing include regional uplift, faulting, erosional stress relief, and seismic activity.
2.3 Soil Conditions

2.3.1 General

The generalized soil stratigraphy along the project alignment includes three units: fill, alluvium, and residual bedrock soil. Due to natural deposition/erosion processes and/or previous cut-and-fill activities prior to and during construction of the LBJ freeway, one or more of these soil units may be absent at any given location. The reader is referred to the individual test boring logs in Appendix B of the Phase I GDR for specific details of the soil stratigraphy at each boring location and to the Generalized Subsurface Profile, Figure 9 of the Phase I GDR for graphical representation.

Anticipated soil conditions along the tunnel section alignment are shown on the geotechnical profile (See Figure 2-2, in the Illustrations). It should be noted that the fill, alluvium, and residual soil, have been combined into one overburden layer on the profile. As shown on Figure 2-2, the total overburden thickness for the tunnel section alignment varies from 0.5 ft (BE1L) to 34 ft. (T21C).

The greatest variation in soil conditions is along the alignment of the West Managed Box. The overburden soils are largely alluvium from BW1L to BW6L and to BW8R. Relatively thin alluvium strata are also found in isolated areas along other portions of the alignment. Borings BW1L, BW2L, and BW4R were not advanced completely through the alluvium, so the total thickness is not known. The borings bounding BW4R on the east (BW3L) and west (BW5L&R) were advanced to the Eagle Ford shale. These borings show the overburden (alluvium and fill) to be 60 to 62 ft. in thickness. The greatest thickness of alluvium was found in boring BW6R. The overburden consists of 4 ft. of fill over 61 ft. of alluvium. Continuing east, the overburden consists of relatively thin layers of fill over residual soil. Immediately east of the alluvium area described above, the overburden is residual Eagle Ford shale. As the contact with the Austin Chalk is approached, the overburden becomes residual Austin Chalk soil. Along the West Managed Box alignment, the minimum overburden thickness is 1 ft., the maximum 65 ft., and averages 20 ft.

Along the alignment of the East Managed Box, where present, overburden consists primarily of a thin stratum of residual Austin Chalk. For the most part, the only overburden along this part of the alignment are thin fill strata. The farthest boring to the east, BE10C, shows a 21 ft. thick stratum of alluvium, the total overburden thickness is 31 ft. In the remaining borings, the overburden thickness ranges from 1-1/2 ft to 3 ft.

One or more of these soil units may be absent at any given location. Although residual soil comprises of weathered bedrock, it is considered soil for the purpose of this Phase 1 GBR presentation, primarily due to its anticipated soil-like behavior during construction.
2.3.2 Fill

The fill varies in density from soft/very loose to hard/very dense, and consists of dark brown calcareous clay mixed with variable percentages of sand and gravel-sized materials. Fill materials also included crushed limestone (road-base materials), asphalt and concrete fragments, and occasional cobble to boulder-sized pieces of limestone. Although encountered in only one of the Phase I borings (T9L), obstructions should be anticipated wherever fill occurs. Obstructions may hinder excavation and installation of excavation support and might include abandoned utilities or foundation elements, construction/pavement debris, wood, boulders, and other miscellaneous debris.

Geotechnical laboratory tests were conducted on fill samples to obtain classification information and, to a lesser extent compressive strength characteristics of intact soil samples. The results of all laboratory testing are presented in the Phase I-GDR. Table 2-2 reports a summary of the maximum, minimum, and mean values of several laboratory tests conducted on fill samples collected along the entire project alignment.

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<td>Liquid Limit</td>
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N = Number of Tests

2.3.3 Alluvium

Because of their very similar nature, both alluvial flood-plain deposits and fluviatile terrace deposits are described as alluvium. These Elm Fork Trinity River deposits can include gravel, sand, silt, silty clay, and organic matter. Alluvium was encountered in three borings along the tunnel alignment (T6R, T23C and T23R) and varied in thickness from about 5 to 16 ft. Only one boring along the East Managed Box (BE10C) showed alluvium. Alluvium, however, has a significant presence along the West Managed Box alignment. Alluvium is present from boring BW1L to BW8R and in borings BW11L/R, BW14L, BW15L, BW17L, and BW20R.

The alluvium ranged in density or consistency from soft/loose to very stiff. The alluvium largely consists of clay with various percentages of fine to coarse sand and fine gravel. Frequently the clays were interbedded with clayey fine gravels,
and lenses or layers of silt to fine sand. Soil classifications of the alluvium include CH, MH, ML, SC, SW-SC, SP-SM, SP, GC, and GP-GC.

There is a distinct granular layer within the overall clayey alluvium. The top of this layer varies from El 481 to El 489 and is generally 4 to 6 ft. in thickness in the borings from BW1L to BW5R. This layer is largely sand and gravel with various percentages of clay. It then tapers to the west, is 2 to 3 ft. thick in the BW6 borings and was not clearly identified in the BW7 borings. The relevant borings generally show groundwater seepage to be correlated to this layer. This granular layer within the alluvium is expected to be water bearing. A similar distinct 4 ft. thick granular layer was noted in boring BW6R at El 471±.

In general, a largely fine sand layer is present near the bottom of the alluvium close to the top of the weathered or fresh Eagle Ford Shale. Because of the erosional environment prior to deposition, the elevation of this layer will vary by several feet. The top of this layer was found to range from El 453± to El 434±. This layer is present in borings from BW1L to BW6L excluding BW6R. This layer within the alluvium is also expected to be water bearing.

Geotechnical laboratory tests were conducted on alluvium to obtain classification information and strength characteristics of intact soil samples. Field-testing was completed on the alluvium using the SPT and TCP tests. The results of all laboratory testing are presented in the Phase I-GDR. Table 2-3 reports a summary of the maximum, minimum, and mean values of several laboratory tests conducted on alluvium samples collected along the entire project alignment.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Min</th>
<th>Mean</th>
<th>Max</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content</td>
<td>%</td>
<td>2</td>
<td>20</td>
<td>31</td>
<td>58</td>
</tr>
<tr>
<td>Dry Density (y_d)</td>
<td>pcf</td>
<td>89</td>
<td>105</td>
<td>116</td>
<td>26</td>
</tr>
<tr>
<td>Liquid Limit*</td>
<td>%</td>
<td>21</td>
<td>60</td>
<td>74</td>
<td>22</td>
</tr>
<tr>
<td>Plastic Limit*</td>
<td>%</td>
<td>14</td>
<td>22</td>
<td>58</td>
<td>22</td>
</tr>
<tr>
<td>Plasticity Index*</td>
<td>%</td>
<td>15</td>
<td>31</td>
<td>48</td>
<td>22</td>
</tr>
<tr>
<td>Unconfined Compressive Strength (q_u)</td>
<td>psi</td>
<td>6</td>
<td>34</td>
<td>81</td>
<td>24</td>
</tr>
<tr>
<td>UU Triaxial, Failure Stress</td>
<td>psi</td>
<td>22</td>
<td>51</td>
<td>40</td>
<td>3</td>
</tr>
<tr>
<td>Free Swell</td>
<td>%</td>
<td>0.1</td>
<td>3</td>
<td>8</td>
<td>3</td>
</tr>
</tbody>
</table>

N = Number of Tests
* = Excludes non-plastic soils

2.3.4 Residual

Residual soils are present along the West Managed Box and Tunnel Section alignment developed on both the Austin Chalk and the Eagle Ford. Residual soil was encountered along the Tunnel Section in only five borings (T18C, T19C,
Residual soils were present in all but a few borings from BW7L/R to BW20R along the West Managed Box. The Eagle Ford region of the project is limited to the West Managed Box alignment. Residual soils ranged in thickness from 5 to 9 ft. along the tunnel alignment and from 6 to 24 ft. along the west box alignment. The residual soils generally ranged in consistency from medium stiff to hard.

The Austin Chalk residual soil material generally consists of dark brown or yellow-brown (tan), calcareous clay with traces of sand-size calcareous nodules, and shell fragments and represents bedrock that has weathered entirely to soil. For the most part, these residual soils are classified as CH materials.

Geotechnical laboratory tests were conducted on the residual Austin Chalk soils to obtain classification information, strength, and the expansive characteristics of intact soil samples. Field-testing was completed on the residual soils using the SPT and TCP tests. The results of all laboratory testing are presented in the Phase I-GDR. Table 2-4 reports a summary of the maximum, minimum, and mean values of several laboratory tests conducted on residual Austin Chalk samples collected along the entire project alignment.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Min</th>
<th>Mean</th>
<th>Max</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content</td>
<td>%</td>
<td>15</td>
<td>26</td>
<td>38</td>
<td>10</td>
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<tr>
<td>Dry Density ($\gamma_d$)</td>
<td>pcf</td>
<td>94</td>
<td>98</td>
<td>104</td>
<td>3</td>
</tr>
<tr>
<td>Liquid Limit*</td>
<td>%</td>
<td>30</td>
<td>51</td>
<td>72</td>
<td>5</td>
</tr>
<tr>
<td>Plastic Limit*</td>
<td>%</td>
<td>15</td>
<td>20</td>
<td>25</td>
<td>5</td>
</tr>
<tr>
<td>Plasticity Index*</td>
<td>%</td>
<td>15</td>
<td>31</td>
<td>47</td>
<td>5</td>
</tr>
<tr>
<td>Unconfined Compressive Strength ($q_u$)</td>
<td>psi</td>
<td>17</td>
<td>19</td>
<td>22</td>
<td>3</td>
</tr>
</tbody>
</table>

N = Number of Tests
* = Excludes non-plastic soils

The Eagle Ford residual soils are uniformly highly plastic (CH), expansive clays. These soils are mottled with color variations including gray, yellow-brown, light brown, light gray, tan, and olive-brown. They can contain trace to some quantities of iron nodules, iron oxide staining, calcite, gypsum, or anhydrite nodules, and iron or calcite concretions. They are found to range from non-calcareous to slightly calcareous to calcareous. Occasionally, there will be evidence of fissility in the materials classified as residual soil. The distinction between residual soil and completely weathered shale is very subtle and transitional. The differentiation is based on a geologic interpretation of the presence of relic shale (rock) structure or lack thereof. Locally these materials are often referred to as "Shaly Clay."
As with the Austin Chalk residual soils, geotechnical laboratory tests were conducted on the residual Eagle Ford soils. The results of all laboratory testing are presented in the Phase I-GDR Field-testing was completed on the residual soils using the SPT and TCP tests. Table 2-5 reports a summary of the maximum, minimum, and mean values of several laboratory tests conducted on residual Eagle Ford samples collected along the entire project alignment.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Min</th>
<th>Mean</th>
<th>Max</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content</td>
<td>%</td>
<td>19</td>
<td>26</td>
<td>35</td>
<td>43</td>
</tr>
<tr>
<td>Dry Density ((r_d))</td>
<td>pcf</td>
<td>87</td>
<td>97</td>
<td>110</td>
<td>24</td>
</tr>
<tr>
<td>Liquid Limit*</td>
<td>%</td>
<td>54</td>
<td>66</td>
<td>79</td>
<td>19</td>
</tr>
<tr>
<td>Plastic Limit*</td>
<td>%</td>
<td>20</td>
<td>24</td>
<td>28</td>
<td>19</td>
</tr>
<tr>
<td>Plasticity Index*</td>
<td>%</td>
<td>34</td>
<td>42</td>
<td>51</td>
<td>19</td>
</tr>
<tr>
<td>Unconfined Compressive Strength ((q_u))</td>
<td>psi</td>
<td>7</td>
<td>19</td>
<td>69</td>
<td>19</td>
</tr>
<tr>
<td>UU Triaxial, Failure Stress</td>
<td>psi</td>
<td>26</td>
<td>32</td>
<td>35</td>
<td>3</td>
</tr>
<tr>
<td>Free Swell</td>
<td>%</td>
<td>1</td>
<td>3</td>
<td>4</td>
<td>4</td>
</tr>
</tbody>
</table>

N = Number of Tests

* = Excludes non-plastic soils

2.4 Austin Chalk

2.4.1 Lithology

The principal bedrock unit to be encountered along the project alignment is the Austin Formation that is informally known as the Austin Chalk. The near surface contact between the Austin Chalk and the Eagle Ford is between the BW21 and the BW22 boring lines. Therefore, the Austin Chalk is present from the contact east to the end of the alignment. The weathered Austin Chalk includes the uppermost portions of the formation that are observed below the overburden described above. This portion of the Austin Chalk is characterized as being completely weathered to moderately weathered rock and was encountered in 47 borings drilled along alignment. The weathered Austin Chalk ranges in thickness from about 1 to 20 ft. with an average thickness of approximately 7 ft.

Completely weathered Austin Chalk was observed in nine of the 45 borings along the tunnel alignment. This material ranges from about 1 to 10 ft. in thickness, and has a clay-like consistency that is very stiff to hard. The completely weathered Austin Chalk is typically tan to yellow-brown with occasional iron oxide staining on relic features. This material represents bedrock that has completely weathered to a soil-like consistency, but which still demonstrates relic bedrock fabric (i.e. bedding and fractures) and transitions to highly to moderately weathered limestone.
Slightly weathered to fresh Austin Chalk was encountered in all 45 of the Phase I borings drilled along the tunnel alignment ranging in drilled thickness from approximately 80 ft at the west end of the Tunnel Project area to approximately 210 ft at the east end. West of the tunnel alignment, the Austin Chalk thins as the contact with the Eagle Ford is approached. The fresh limestone was observed to be light to medium gray with interbeds and zones of dark gray, argillaceous limestone. Bedding is subhorizontal and varies from laminar to thick. Due to the depositional history of this formation, the Austin Chalk has been divided into three members by local geologists: the upper chalk, the middle marl (argillaceous limestone), and the lower chalk. The upper and lower chalk units are similar in lithologies consisting of 2 to 5 ft thick beds of limestone, interbedded with 2 in. to 2 ft thick beds of argillaceous limestone and calcareous shale. The middle member consists of up to 5 ft thick beds of moderately to highly argillaceous limestone and/or calcareous shale interbedded with 1 to 2 ft thick beds of limestone. The division from the middle to the lower member is generally discernable in the project borings, being from a few to several feet below the primary bentonitic marker bed. In general, the engineering properties for each of these members of the Austin Chalk are quite similar.

Several thin layers of bentonite or bentonitic shale and weak shale partings are noted throughout the Austin Chalk varying in thickness from 1/8 inch to 3 inches. One particularly thick bentonitic shale layer referred to as the primary “bentonite marker bed” is observed as dark gray to blue gray bentonitic shale, very soft, ranging in thickness from 5 in. to 10 in., and is quite persistent throughout the project alignment (having been observed in 28 borings between test borings T9L and BE1L1). The contact between the Austin Chalk and the underlying Eagle Ford Shale was found to be 95 to 100 ft below the primary bentonite marker bed. Another similar marker bed was observed at 11 boring locations between test borings T26C and BE1L1 at a higher elevation in the Chalk. This upper bentonite marker bed also consists of very soft, dark gray to blue gray bentonitic shale, ranging in thickness from 3 to 5 in. and situated approximately 49 to 52 ft above the primary bentonite marker bed. These bentonite marker beds and other laminar to thin beds of bentonitic shale and weak shale are expected to locally impact the stability of the tunnel during construction. Because of their low angle with respect to the tunnel alignment, they have the potential to form roof slabs in the crown and arch sections of the tunnel that can extend for several tens of feet.

At the base of the Austin Chalk, the limestone becomes argillaceous with frequent interbeds and laminations of sandy material. The lowermost 1 to 2 ft of this zone is very gritty with abundant fine black fish bone fragments and shark teeth fossils. Geologic literature (Dawson et al., 1983) refers to this zone as a lag deposit that also includes quartz and glauconite sand grains, phosphate pebbles, and clasts. This marker bed represents the transition zone from the Austin Chalk to the underlying Eagle Ford Shale. It is locally referred to as the “Fish Bed Conglomerate.” Since this bed lies at the contact with the Eagle Ford Shale, it is at a depth that will not impact the tunnel construction.
2.4.2 Geologic Structure

The term discontinuity, as applied herein, refers to any natural break in the rock mass, such as a joint, shear (zone), or fault. Joints are distinguished from shears and faults in that they are not the result of displacement or offset of the rock mass. Structural discontinuities of similar orientation are herein referred to as "sets", or "joint sets."

Due to the paucity of exposures or outcroppings of the Austin Chalk in the project area, preliminary characterization of the bedrock structure is based on available relevant literature and observations made in the rock core. Bedding is a term used to denote a plane observed in the rock believed to reflect the original depositional surface. During sediment deposition, variations in such factors as the flow velocity of the depositing medium and/or sediment source often result in individual layers of varying grain size, texture, or composition, termed "beds." If the original bedding plane is believed to be horizontal, then variations in the bedding orientation can be linked to post depositional deformation such as folding or faulting.

Bedding joints were the most frequently observed feature in the core from the project borings. The terms bedding joints, partings, and seams were used to characterize these features in the core. Bedding joints were often manifested as horizontal to subhorizontal thicknesses of intact limestone separated by very soft-to-soft shale or bentonitic partings and layers (1/16 in to 3 in thick). In some cases the weak infilling was probably washed out during coring. As mentioned above, two thicker bentonite marker beds are prevalent throughout the entire proposed tunnel alignment area. Only very infrequently were moderate to high-angle joints or joint sets observed in the core, although it must be noted that steeply dipping to vertical joint sets cannot be readily investigated with vertical test borings.

Several shear fractures were observed in the core consisting of a moderate to high angle (25° to 85°), slickensided surfaces varying from planar to wavy and observed as either separated surfaces or healed intact hairline discontinuities. Generally, the shear fractures are not evenly spaced, but tend to occur in sets of closely spaced sub-parallel fractures described as "swarms." Fractures on the periphery of a swarm tend to be spaced farther apart and typically show less offset. Fractures at the center of swarms tend to be closely spaced and may show evidence of greater offset and rock damage including brecciation, mineral cements, mineral alteration, and in some cases, gouge. Larger swarms of high angle fractures form shear zones and in some cases can accompany or define a fault or fault zone. The observed shear swarms were generally a few inches to about 2 ft. thick, however, two such swarms were observed as follows: one occurred over an eight-foot thickness of boring T18C, at a depth of 58-66 ft, and another occurred over a ten-foot thickness of boring T23R at a depth of 70-80 ft. Occasionally, shear swarms included calcite infillings, and (in one occurrence) brecciated fragments of limestone (boring T33C, 675 ft deep). As noted on the
core logs, however, the shear swarms were generally healed and tight with no observed soft rock or soil-like weathering (gouge).

The local geologic literature notes the occurrence of several faults in the Austin Chalk throughout the greater Dallas area. As reported, these faults are most commonly normal faults with generally less than 15 ft. of observed vertical offset. Though situated several miles east and south of the project area, the Balcones Fault System is believed to have influenced geologic structure in the Dallas area. Collins, et al. (1991) cites that along the Balcones Fault Zone, joints (and most faults and veins) in the Austin Chalk typically strike in one of three azimuth orientations: northeast (30 to 60°), eastward (70 to 100°), and northward (350 to 20°). Allen et al. (1986) cites that major faults in the Dallas area were found to strike at N10°W (azimuth 350°). Due to the extent of the project alignment, it is anticipated that similar features of these orientations will be encountered. Based on the test boring data, the possibility of at least one to three faults or shear zones may be present along the project tunnel alignment, particularly between Phase I borings T20C to T23C (generally between approximate Sta. 260+00 and Sta. 275+00). As noted on the Geotechnical Profile (Figure 2-2), the generally consistently flat-lying bentonite marker bed shows possible relative vertical offsets of up to 15 ft within this area. Depending on the character of the shear zones or faults and their orientation with respect to the orientation of the tunnel, the impact to tunnel construction may be minimal and generally localized (within a zone or zones of 5 to 30 ft. in length). However, if such features are sub-parallel to the tunnel alignment, greater lengths of the tunnel may be affected. It is the intent of the Phase II site investigations to determine the character, orientation, and extent of any shear zones or faulting, if present, in this part of the tunnel alignment.

2.4.3 Intact Rock Properties

Geotechnical laboratory tests were conducted on Austin Chalk rock samples to obtain information on the compressive strength, tensile strength, and slake durability characteristics of intact rock samples; the results of which testing are presented in the Phase I-GDR. The intact rock properties presented in sections 2.4.3.1 through 2.4.3.7 are based only on those data obtained from the relatively unweathered (fresh to slightly weathered) rock mass below the upper weathered/fractured zone (UWFZ) and from samples obtained from within or immediately adjacent to the tunnel envelope as presently designed. Intact rock properties for the managed box sections and the weathered/fractured zone are present in section 2.4.3.8.

Table 2-6 reports a summary of the maximum, minimum, and mean values of several laboratory tests conducted on the Austin Chalk for the Tunnel Project vicinity.
Table 2-6: Summary of Representative Engineering Properties of the Austin Chalk

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Min</th>
<th>Mean</th>
<th>Max</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry Density ($y_a$)</td>
<td>pcf</td>
<td>97</td>
<td>123</td>
<td>143</td>
<td>126</td>
</tr>
<tr>
<td>Unconfined Compressive Strength ($q_u$)</td>
<td>psi</td>
<td>1204</td>
<td>2791</td>
<td>3997</td>
<td>83</td>
</tr>
<tr>
<td>Young's Modulus ($E_{50}$)</td>
<td>ksi</td>
<td>150</td>
<td>576</td>
<td>1094</td>
<td>77</td>
</tr>
<tr>
<td>(at 50% strain)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Modulus Ratio ($E + 50 / q_u$)</td>
<td>---</td>
<td>74</td>
<td>204</td>
<td>376</td>
<td>77</td>
</tr>
<tr>
<td>Axial Point Load Strength</td>
<td>psi</td>
<td>125</td>
<td>480</td>
<td>780</td>
<td>171</td>
</tr>
<tr>
<td>Diametric Point Load Strength</td>
<td>psi</td>
<td>26</td>
<td>296</td>
<td>656</td>
<td>72</td>
</tr>
<tr>
<td>Unconsolidated Undrained Compressive Strength ($q_uuu$)</td>
<td>psi</td>
<td>3621</td>
<td>4545</td>
<td>6048</td>
<td>16</td>
</tr>
<tr>
<td>Brazilian Tensile Strength ($a_t$)</td>
<td>psi</td>
<td>94</td>
<td>245</td>
<td>350</td>
<td>39</td>
</tr>
<tr>
<td>Slake Durability (after 2 cycles)</td>
<td>%</td>
<td>89</td>
<td>95</td>
<td>98</td>
<td>10</td>
</tr>
</tbody>
</table>

N = Number of Tests

2.4.3.1 Unconfined Compressive Strength

Table 2.6 presents a summary of unconfined compressive strength (UCS) results for unweathered Austin Chalk. UCS tests were generally conducted on two samples selected from the proposed tunnel horizon at each respective boring; and on at least one sample from the respective borings in the cut-and-cover sections. For a total of 83 tests on intact rock core samples from the Tunnel Project borings, the average UCS is 2791 psi. The range of UCS values was 1204 psi to 3997 psi. Chart 2-1 shows the distribution of UCS values described above. The practical maximum value is 4000 psi, which corresponds to 1.0% of the tunnel rock.
2.4.3.2 Young's Modulus

Strain measurements were also taken on the UCS tests described above. From this data, tangent Young's Modulus at 50% strain was obtained from the stress-strain data plotted during the UCS testing.

For the 77 tests considered, tangent Young's Modulus at 50% strain ranged from 150 ksi to a high of 1,094 ksi, with an average of 576 ksi. Chart 2-2 shows the distribution of tangent Young's modulus at 50% strain for fresh Austin Chalk along the tunnel alignment.
2.4.3.3 Modulus Ratio

Another method of evaluating the engineering classification of intact rock is to compare the modulus of elasticity (Young’s Modulus at 50% strain – $E_{t50}$) with the unconfined compressive strength ($q_u$). This comparison yields the modulus ratio, $E_{t50} / q_u$, and is categorized as follows:

- High modulus ratio – over 500
- Average (medium) ratio – 200 to 500
- Low modulus ratio – < 200

Chart 2-3 shows the distribution of the modulus ratio for fresh Austin Chalk along the Tunnel Project alignment.
The average modulus ratio for this rock is 204. As observed, this ratio is at the low end of the Average (medium) ratio range. This may be attributed to 1) the argillaceous character of the weak microgranular limestone; and 2) the somewhat anisotropic nature of the rock which is further discussed in Section 2.4.3.4.

2.4.3.4 Point-Load Strength Index

Point load strength index tests were conducted on a total of 243 samples of intact rock core samples of the Austin Chalk that were selected from the proposed tunnel horizon in the respective borings. Generally four samples from each boring were tested with the load applied axially, or parallel to the length of core, and two samples from each boring were tested diametrically, or perpendicular to the axis of the core.

Axial Point Load Strength (ACLS) indices for 171 tests averaged 480 psi and Diametric Point Load Strength (DPLS) indices for 72 tests averaged 290 psi. Chart 2-4 and 2-5 show the distribution of the APLS and DPLS indices, respectively.
Chart 2-4: Distribution of Axial Point Load Values for Austin Chalk below the UWFZ in the Tunnel Project Area.

Chart 2-5: Distribution of Diametrical Point Load Values for Austin Chalk below the UWFZ in the Tunnel Project Area.
It should be noted that the majority of the samples tested axially, failed through intact rock. The majority of samples tested diametrically generally failed along bedding or along a preferential weakness imparted by the rocks horizontally layered depositional history. As such, the mean point load index value for those samples tested axially is comparatively higher than the mean value for those samples tested diametrically. These results indicate the somewhat anisotropic nature of the Austin Chalk across the project area.

2.4.3.5 Unconsolidated Undrained Compressive Strength

Samples for Unconsolidated Undrained Compressive Strength (UUCS) tests were taken from approximately half of the borings along the Tunnel Project alignment. One sample was collected from within the proposed tunnel horizon at each of the respective borings.

For a total of 16 tests, the (UUCS) values ranged from 3621 psi to 6048 psi, with a mean value of 4545 psi. The mean value of the strength gained under confining pressure is approximately 1.5 times greater than the mean value of the UCS value. Chart 2-6 presents the distribution of the (UUCS) values for fresh Austin Chalk along the tunnel alignment.

![Chart 2-6: Distribution of Unconsolidated Undrained Compressive Strength values for Austin Chalk below the UWFZ in the Tunnel Project Area.](image)
2.4.3.6 Brazilian Tensile Strength

Brazilian Tensile Strength (BTS) tests were conducted on a total of 39 samples of rock core. Approximately one sample was selected from each boring along the tunnel alignment and from a depth that approximated the crown or upper arch level in each respective boring.

For the 39 samples tested, the range of BTS values was 94 psi to 350 psi, with a mean value of 245 psi. Chart 2-7 shows the distribution of BTS values for fresh Austin Chalk in the Tunnel Project area. The practical maximum value for the Tunnel Project is 375 psi based on the testing completed to date.

![Chart 2-7: Distribution of Brazilian Tensile Strength values for Austin Chalk below the UWFZ in the Tunnel Project Area.](image)

2.4.3.7 Slake Durability Tests

Slake durability tests were conducted on one rock core sample taken from each of 10 borings along the tunnel alignment. Samples of nonargillaceous to moderately argillaceous Austin Chalk were obtained from depths representing the approximate tunnel horizon in the respective borings.
Slake durability index values for Austin Chalk (after two cycles) range from 89% (medium) to 98% (very high), with a mean value of 95% (high). The test results indicate that the Austin Chalk does not exhibit significant slaking behavior. However, more argillaceous or calcareous shale variations are expected to slake upon extended exposure to the atmosphere in an underground environment. Slake durability values under 50% (low) are estimated for some shale beds in the Austin Chalk. The bentonitic shales from the bentonite marker beds are anticipated to have very low to negligible slake durability based on observations made while logging and handling the core samples. Additional testing is planned during the Phase II investigations to quantify these estimates.

2.4.3.8 Intact Rock Properties Managed Box Sections / Weathered Zone

Geotechnical laboratory tests were also conducted on Austin Chalk in the East and West Managed Box Section as well as on samples obtained from the weathered zone. Tests were performed to obtain information on the classification and strength characteristics of intact rock samples. The results are presented in the Phase I-GDR.

Tables 2-7 and 2-8 report a summary of the maximum, minimum, and mean values of several laboratory tests conducted on the fresh Austin Chalk for the west and east Managed Box sections.

### Table 2-7: Summary of Fresh Austin Chalk Test Results West Managed Box

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Min</th>
<th>Mean</th>
<th>Max</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content, %</td>
<td>%</td>
<td>9</td>
<td>12</td>
<td>14</td>
<td>8</td>
</tr>
<tr>
<td>Dry Density ($y_d$)</td>
<td>pcf</td>
<td>121</td>
<td>127</td>
<td>139</td>
<td>8</td>
</tr>
<tr>
<td>Unconfined Compressive Strength ($q_u$)</td>
<td>psi</td>
<td>1929</td>
<td>2755</td>
<td>3666</td>
<td>8</td>
</tr>
<tr>
<td>Point Load Strength</td>
<td>psi</td>
<td>319</td>
<td>432</td>
<td>705</td>
<td>5</td>
</tr>
</tbody>
</table>

N = Number of Tests

### Table 2-8: Summary of Fresh Austin Chalk Test Results East Managed Box

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Min</th>
<th>Mean</th>
<th>Max</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content, %</td>
<td>%</td>
<td>2</td>
<td>5</td>
<td>14</td>
<td>28</td>
</tr>
<tr>
<td>Dry Density ($y_d$)</td>
<td>pcf</td>
<td>115</td>
<td>122</td>
<td>128</td>
<td>29</td>
</tr>
<tr>
<td>Unconfined Compressive Strength ($q_u$)</td>
<td>psi</td>
<td>1997</td>
<td>2858</td>
<td>4159</td>
<td>22</td>
</tr>
<tr>
<td>Point Load Strength</td>
<td>psi</td>
<td>238</td>
<td>455</td>
<td>6338</td>
<td>5</td>
</tr>
</tbody>
</table>

N = Number of Tests

Tables 2-9 through 2-11 report a summary of the maximum, minimum, and mean values of several laboratory tests conducted on the weathered...
Austin Chalk. The range in weathering from completely to moderately weathered is evident in the tests results. Some samples show clay-like behavior that is shown by the plasticity testing. Other samples show rock-like behavior as seen in the unconfined and point load testing.

### Table 2-9: Summary of Weathered Austin Chalk Test Results Tunnel

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Min</th>
<th>Mean</th>
<th>Max</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content, %</td>
<td>%</td>
<td>10</td>
<td>21</td>
<td>37</td>
<td>13</td>
</tr>
<tr>
<td>Dry Density (γ₀)</td>
<td>pcf</td>
<td>100</td>
<td>111</td>
<td>121</td>
<td>5</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>%</td>
<td>31</td>
<td>43</td>
<td>58</td>
<td>6</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>%</td>
<td>15</td>
<td>19</td>
<td>22</td>
<td>6</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>%</td>
<td>15</td>
<td>24</td>
<td>36</td>
<td>6</td>
</tr>
<tr>
<td>Unconfined Compressive Strength (qᵤ)</td>
<td>psi</td>
<td>13</td>
<td>576</td>
<td>2698</td>
<td>5</td>
</tr>
<tr>
<td>Point Load Strength</td>
<td>psi</td>
<td>241</td>
<td>379</td>
<td>505</td>
<td>4</td>
</tr>
</tbody>
</table>

N = Number of Tests

### Table 2-10: Summary of Weathered Austin Chalk Test Results West Managed Box

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Min</th>
<th>Mean</th>
<th>Max</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content, %</td>
<td>%</td>
<td>9</td>
<td>13</td>
<td>16</td>
<td>5</td>
</tr>
<tr>
<td>Dry Density (γ₀)</td>
<td>pcf</td>
<td>125</td>
<td>129</td>
<td>135</td>
<td>3</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>%</td>
<td>33</td>
<td>34</td>
<td>35</td>
<td>2</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>%</td>
<td>16</td>
<td>17</td>
<td>17</td>
<td>2</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>%</td>
<td>16</td>
<td>18</td>
<td>19</td>
<td>2</td>
</tr>
<tr>
<td>Unconfined Compressive Strength (qᵤ)</td>
<td>psi</td>
<td>3253</td>
<td>3460</td>
<td>3666</td>
<td>2</td>
</tr>
<tr>
<td>Point Load Strength</td>
<td>psi</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>0</td>
</tr>
</tbody>
</table>

N = Number of Tests
Table 2-11: Summary of Weathered Austin Chalk Test Results

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Min.</th>
<th>Mean</th>
<th>Max</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content, %</td>
<td>%</td>
<td>14</td>
<td>20</td>
<td>31</td>
<td>3</td>
</tr>
<tr>
<td>Dry Density ($\gamma$)</td>
<td>pcf</td>
<td>NA</td>
<td>98</td>
<td>NA</td>
<td>1</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>%</td>
<td>49</td>
<td>52</td>
<td>56</td>
<td>2</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>%</td>
<td>22</td>
<td>23</td>
<td>24</td>
<td>2</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>%</td>
<td>27</td>
<td>30</td>
<td>32</td>
<td>2</td>
</tr>
<tr>
<td>Unconfined Compressive Strength ($q_u$)</td>
<td>psi</td>
<td>NA</td>
<td>42</td>
<td>NA</td>
<td>1</td>
</tr>
<tr>
<td>Point Load Strength</td>
<td>psi</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>0</td>
</tr>
</tbody>
</table>

N = Number of Tests

2.4.4 Rock Mass Characteristics

2.4.4.1 Tunnel Alignment

Because the proposed Project Tunnel alignment lies entirely within the Austin Chalk and a minimum of 45 ft. above the underlying Eagle Ford Shale, analyses and discussions of rock mass characteristics are limited to the Austin Chalk only. Rock mass properties in the vicinity of the Project Tunnel alignment have been characterized by evaluating bedrock conditions of the Austin Chalk through observations and measurements of rock core. Based on the results of this evaluation, two distinguishable zones are apparent; 1) an Upper Weathered/Fractured Zone at the top of bedrock, and 2) a generally fresh to slightly weathered rock mass below the Upper Weathered/Fractured Zone.

An Upper Weathered/Fractured Zone (UWFZ) was identified to account for the weathering continuum that extends downward into the rock mass from the top of bedrock. The concept was applied systematically by identifying an UWFZ based on an assessment of RQD, weathering, and discontinuity spacing. This zone is below the overburden and typically consists of rock that is completely weathered to moderately weathered, and also including rock that is slightly weathered to fresh, but which has relatively lower RQD than the underlying rock mass.

In many borings, the observed boundary between the UWFZ and underlying rock is gradational and transitional and as such the criteria for evaluating the thickness of the UWFZ at individual test boring locations is somewhat subjective. Rock core characterized as slightly weathered to fresh was included in the UWFZ when it was observed to be fractured and when the discontinuities were observed to be discolored and decomposed, or when it was auger drilled instead of cored.
The thickness of this zone ranges from approximately 2 ft. (T29C) to approximately 22 ft. (T17C), but the UWFZ is typically less than 9 ft. thick. The depth below ground surface to the bottom of the upper weathered/fractured zone ranges from about 6 ft (T14C, T29C and T33C) to 49 ft (T21C). There does not appear to be a correlation between argillaceous content and the thickness of this zone. Below the UWFZ, the rock mass is generally slightly weathered to fresh.

The Rock Quality Designation (RQD) of the core recovered from the UWFZ varied from 0 to 96 percent, with an average RQD of 67 percent for the Tunnel Project area. It should be noted that only 27 percent of the core recovered from this zone in the Tunnel Project area was classified as very poor to poor, with an RQD ranging from zero to 50 percent. The distribution of RQD values for the core recovered from the UWFZ in the Tunnel Project area is shown in Chart 2-8.

![Chart 2-8: Distribution of RQD Values for Austin Chalk in the UWFZ in the Tunnel Project Area.](chart)

Poorer quality and/or weathered rock of the UWFZ will create issues of localized instability in the areas of the cut-and-cover tunnels and tunnel portals. See discussions in Chapter 3 for further details.

Below the UWFZ, the average RQD for the rock core (Austin Chalk) obtained from the Tunnel Project vicinity was 97 percent. Approximately
90 percent of all rock cored below the UWFZ in the Tunnel Project vicinity can be categorized as excellent quality (RQD greater than 90), and zero percent is classified as either poor to very poor (RQD less than 50).

The zones of lower RQD included in this evaluation are typically associated with soft to very soft shale, highly argillaceous or bentonitic shale seams or partings, closely spaced joints, vertical joints, or possible faults/shear zones. In the vicinity of the Tunnel Project, these zones of lower RQD are up to 10 ft. thick (i.e. T18C and T23R), and occur in generally fresh rock with hardness varying from very soft (bentonitic seams) to moderately hard. The distribution of RQD values for the core recovered from below the UWFZ in the Tunnel Project vicinity is shown in Chart 2-9.

2.4.4.2 Managed Box Section

Because structures in the Managed Box Sections will be designed and constructed within the entire stratigraphic section, a distinction was not made between the rock above and below the UWFZ, as deemed significant for tunneling.
For the West Managed Box area, the Rock Quality Designation (RQD) of the core recovered varied from 42 to 100 percent, with an average RQD of 90 percent. The distribution of RQD values for the core recovered is shown in Chart 2-10.

![Chart 2-10: Distribution of RQD Values for Austin Chalk in the West Managed Box](image)

For the East Managed Box area, the Rock Quality Designation (RQD) of the core recovered varied from 60 to 100 percent, with an average RQD of 97 percent. The distribution of RQD values for the core recovered is shown in Chart 2-11.
2.5 Eagle Ford

2.5.1 Lithology

The Eagle Ford shale lies stratigraphically below the Austin Chalk and is present throughout the Section 4-West Project alignment. Due to the southeasterly regional dip and erosional truncation, the Eagle Ford subcrops throughout much of the greater Dallas vicinity to the west of the Austin Chalk subcrop. Approximately the western-most one-third of the overall Section 4-West project alignment will be founded in the Eagle Ford. In the immediate vicinity of the Tunnel Project area, the Eagle Ford shale generally lies a minimum of about 45 ft below the proposed invert of the presently planned tunnel arrangements. As such, the Eagle Ford does not impact the tunnel alignment as it is presently designed nor the East Managed Box. The West Managed Box will be in the Austin Chalk and the Eagle Ford.

The project borings were not advanced through the Eagle Ford into the underlying Woodbine formation. However, in the general area of the project alignment, the total thickness of the Eagle Ford Shale is expected to range from approximately 200 to 300 ft. Five deep borings were advanced through the Austin Chalk to locate the top of the Eagle Ford along the Tunnel alignment. These borings from west to east are T12R, T19L, T23R, T30L, and BE1L1. Each
was drilled through the overlying Austin Chalk and advanced from 14 to 24 ft. into the Eagle Ford Shale. In these five borings, the shale was encountered at depths ranging from 139 ft. to 211 ft (from west to east) below the ground surface.

Otherwise, the Eagle Ford was present east to Boring BW26R. On the far west end of the alignment the Eagle Ford is present below the alluvium. East of the alluvium, slightly weathered to fresh Eagle Ford shale is present below a typical profile consisting of fill over residual soils over weathered shale. However, because the boring's depths were based on the proposed elevations of the planned structures, borings were not advanced to identify this complete profile at each location. Borings were terminated in alluvium, weathered shale, or fresh shale. Based on the limited data for the Eagle Ford, the top of slightly weathered to fresh shale was found to range from El 552± to El 404±. In a non-alluvial environment, the weathered shale ranged in thickness from 10 ft to 41 ft. Beneath the alluvium, the minimum thickness was 3 ft. A maximum thickness was not determined.

The Eagle Ford Shale is a weak rock unit consisting of fissile dark gray to black calcareous to non-calcareous clay shale with trace to few thin (0.5 to 2 in. thick) limestone beds in the lower portion. The fresh shale is generally dark gray, soft, with horizontal to subhorizontal laminar, fissile bedding. Occasional pyrite nodules, very thin limestone, and fine sandstone interbeds were also observed. In addition, at a few locations, very hard septarian nodules were encountered as inclusions in the shale. The formation is divided into two members, the upper Arcadia Park member overlying the Britton member. Where the Eagle Ford Shale has been exposed at the surface, the weathering profile can extend to a considerable depth.

The uppermost portions of the Eagle Ford Shale are locally considered a "surficial" unit in that it is completely weathered to a clay-like material. This upper completely weathered shale is locally known as "shaly clay." The weathered shale varies from mottled gray and yellow-brown to dark gray in color, is generally very soft, retains a relic laminar fissility and is frequently stained with iron oxide along relic bedding and fractures.

The Arcadia Park member consists of a dark gray to grayish black, non-calcareous, laminated shale. Approximately 40 to 60 ft. below the contact with the overlying Austin Chalk, a septarian concretionary layer may be found. Septarian concretions in the Eagle Ford are very hard, can be relatively large (3 in. to 36 in. diameter), are roughly spheroidal, and consist of argillaceous ironstone characterized by irregular polyhedral blocks cemented by calcite. The base of the overlying Arcadia Park member is marked by a 1- to 3-ft thick bed of detrital limestone locally referred to as the Kamp Ranch Limestone.

The upper Britton member consists of softer, less calcareous shale than the lower Britton Limestone and claystone concretions up to 12 in. thick are common in this interval. The lower portion of the Britton member consists of
moderately hard, calcareous clay shale with up to 21 poorly developed ashy bentonite seams. Separating the upper Britton from the lower Britton are two marker beds consisting of bentonite seams approximately 6 to 12 in. thick.

The fresh shale is generally dark gray, soft, with horizontal to subhorizontal laminar, fissile bedding. Occasional pyrite nodules, very thin limestone, and fine sandstone interbeds were observed. At a few locations, very hard septarian nodules were encountered as inclusions in the shale.

2.5.2 Geologic Structure

Using the terminology for discontinuities as described in the Austin Chalk section of this report, such natural breaks in the rock mass, such as a joint, shear (zone), or fault are difficult to identify in the Eagle Ford shale. The reason for this is the shale largely heals such features except very near the surface. As described above, near the surface the shale is subject to significant weathering. The weathering and desiccation leads to the development of apparent discontinuities that otherwise may not be present in the fresh shale. Further, other research has shown that even major faults in the Austin are difficult to trace in the underlying Eagle Ford shale. Two possible explanations that have been proposed are (1) the healing effect described and (2) that fault traces through the Austin Chalk begin to quickly curve on an arc to a point where they become parallel to the shale bedding.

2.5.3 Intact Rock Properties

Geotechnical laboratory tests were conducted on shale samples from the Eagle Ford to obtain information on the classification information, strength, and the expansive characteristics of intact soil samples. Field-testing included SPT and CPT tests. The results of all testing are presented in the Phase I-GDR. Tables 2-12 and 2-13 report the summary of the maximum, minimum, and mean values of several laboratory tests conducted for weathered and fresh Eagle Ford.
Table 2-12: Summary of Weathered Eagle Ford Shale Test Results

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Min</th>
<th>Mean</th>
<th>Max</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content, %</td>
<td>%</td>
<td>13</td>
<td>20</td>
<td>31</td>
<td>50</td>
</tr>
<tr>
<td>Dry Density (ρ_d)</td>
<td>pcf</td>
<td>89</td>
<td>105</td>
<td>124</td>
<td>31</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>%</td>
<td>45</td>
<td>65</td>
<td>77</td>
<td>14</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>%</td>
<td>19</td>
<td>24</td>
<td>31</td>
<td>14</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>%</td>
<td>26</td>
<td>41</td>
<td>49</td>
<td>14</td>
</tr>
<tr>
<td>Unconfined Compressive Strength (q_u)</td>
<td>psi</td>
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<td>72</td>
<td>237</td>
<td>27</td>
</tr>
<tr>
<td>UU Triaxial, Failure Stress</td>
<td>psi</td>
<td>31</td>
<td>75</td>
<td>135</td>
<td>3</td>
</tr>
<tr>
<td>Free Swell</td>
<td>%</td>
<td>6</td>
<td>9.5</td>
<td>13</td>
<td>2</td>
</tr>
</tbody>
</table>

N = Number of Tests

Table 2-13: Summary of Fresh Eagle Ford Shale Test Results

<table>
<thead>
<tr>
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<th>Units</th>
<th>Min</th>
<th>Mean</th>
<th>Max</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content, %</td>
<td>%</td>
<td>10</td>
<td>16</td>
<td>19</td>
<td>24</td>
</tr>
<tr>
<td>Dry Density (ρ_d)</td>
<td>pcf</td>
<td>113</td>
<td>118</td>
<td>134</td>
<td>24</td>
</tr>
<tr>
<td>Unconfined Compressive Strength (q_u)</td>
<td>psi</td>
<td>105</td>
<td>181</td>
<td>322</td>
<td>19</td>
</tr>
<tr>
<td>Point Load Strength</td>
<td>psi</td>
<td>61</td>
<td>64</td>
<td>62</td>
<td>6</td>
</tr>
</tbody>
</table>

N = Number of Tests

2.5.4 Rock Mass Characteristics

The Rock Quality Designation (RQD) of the Eagle Ford shale core varied from 0 to 100 percent, with an average RQD of 86 percent. The distribution of RQD values for the core recovered is shown in Chart 2-12.
2.6 Groundwater Conditions

A total of eleven (11) vibrating wire-type piezometers (VWP) and eight (8) standpipe-type piezometers (SPP) were installed in boreholes in the overburden and bedrock over the entire Section 4-West project area. Seven (7) of the VWP and three (3) of the SPP were installed in fresh Austin Chalk bedrock in the immediate vicinity of the Tunnel Project alignment. Of these installations, each were set at depths within or immediately above or below the proposed tunnel envelope at their respective borehole locations. Seven piezometers were installed along the West Managed Box alignment and two for the East Managed Box alignment. The monitoring period considered in this report generally varies between late September and early February 2004, depending on when the installation was made.

Along the West Managed Box alignment, the piezometers are indicating a piezometric level from 9 to 22 ft below ground surface. The piezometers along the Tunnel alignment are indicating piezometric levels varying from 12 ft below ground surface to as much as 7 ft above ground surface (artesian). The two piezometers for the East Managed Box are showing levels of 4 and 7 ft below ground surface. The two piezometers showing artesian piezometric conditions are in borings T30L and BE1R. Both of these are VWP’s. Through mid-December, VWP’s T10R and T13L also showed artesian conditions. Three borings BW12R, T9L, and T10R were completed with two piezometers. A VWP was placed below the standpipe piezometer. The piezometric levels in BW12R, T9L in the lower VWP correspond to those in the SPP above. In T13L
the piezometer level is higher in the VWP than the corresponding SPP above, suggesting a possible upward hydraulic gradient within the bedrock. This was initially the case in T10R, also

Preliminary indications show that the piezometric level is generally at or near the ground surface to approximately 10 ft. below ground surface throughout the project alignment area.

Hydraulic conductivity (k, in cm/sec) of the rock mass was evaluated based on the results of 14 water pressure tests performed in 13 borings all in the immediate vicinity of Tunnel Project area with exception of BW18R. The 14 tests were conducted in the Austin Chalk and at the depth intervals of the proposed tunnel horizon in the respective boreholes. In the case of BW18R, the test was performed at an elevation relevant to the proposed box structure. The calculated hydraulic conductivity values range from $1.375 \times 10^{-7}$ to $2.566 \times 10^{-5}$ cm/s. One test indicated no water flow, and is assumed to have a hydraulic conductivity value of about $1 \times 10^{-6}$ cm/s. Because of a pump malfunction, one test, T23R was performed as a falling head test. This test yielded the largest result by one order of magnitude. Therefore, because of the test method and results, this value is questionable. For the 12 other tests, the Austin Chalk showed an average hydraulic conductivity of $8.8 \times 10^{-7}$ cm/s. All but the two identified yielded hydraulic conductivity values between $10^{-7}$ cm/sec to $10^{-6}$ cm/sec. Examination of the test data reveals no clear correlation between depth below top of bedrock, lithology, weathering, or location and hydraulic conductivity as measured by the project water pressure tests. However, the range of permeability measured is very narrow, indicating relatively uniform permeability conditions throughout the project alignment. All of those measurements indicate low to very low bedrock permeability. Water inflows are expected to be minimal throughout most of the alignment. The presence of faults or significant shear zones may manifest localized zones of increased flush flows; however, these flows are expected to decline rapidly (within a few hours) in view of the overall low permeability.

2.7 Seismic Considerations

The Uniform Building Code has assigned Central Texas as a zone of no damage (Zone 0) due to earthquakes or seismic activity. Although underground structures are not immune to damage from earthquakes, the strains generated from seismic activity typically have little effect on them. At the levels of seismic acceleration that would be expected from earthquakes estimated to affect the Dallas area, seismic loadings are negligible and can be considered to have no effect on the Tunnel Project structures.

3.0 GROUND CHARACTERIZATION FOR TUNNEL SECTION

3.1 General

This section provides a discussion of the subsurface conditions, as presented in Section 2 of this report and in the Phase I Geotechnical Data Report (GDR), that are anticipated
to impact the tunnel excavation and construction along the tunnel alignments proposed for this project. These characterizations will be used to further the development of the anticipated mining procedures and support systems relevant to this project. The purpose of this chapter is to convey these characterizations to prospective CDA teams. Four previous tunnel projects excavated in the Austin Chalk in the Dallas area are summarized at the end of this section and the method of excavation and some of the ground conditions encountered are briefly outlined.

A combined subsurface profile for both the eastbound and westbound tunnel alignments is presented as Figure 2-2. Based upon this figure, the thickness of "cover" between the tunnel crown and existing ground surface will vary from 6 ft. to approximately 75 ft. This "cover" consists of fill, alluvium, and residual soil up to 22 ft of the Upper Weathered/Fractured Zone (UWFZ) of bedrock, and the underlying bedrock. Some or all of these layers are present above various sections of the tunnel alignments. Figure 2-2 presents the interpreted stratigraphy and piezometric water levels along the combined project tunnel alignments. As additional investigations are carried out, details of ground conditions specific to each alignment will be presented in subsequent issues of this document.

3.2 West End Cut-and-Cover Tunnels

As described earlier, and as shown on the profile Figure 2-2, the length of cut-and-cover tunneling for the west end of the eastbound HOV tunnel is approximately 295 ft. and for the west end of the westbound HOV tunnel is approximately 325 ft. The depth of excavation for both of these portions of the tunnels at the west end of the project will vary from approximately 40 to 55 ft. Both of these excavations will be through similar materials comprising up to 13 ft. of overburden, including fill and completely weathered soil-like Austin Chalk, and up to 14 ft. of UWFZ. Excavation of fresh Austin Chalk bedrock will vary in thickness from approximately 18 to 38 ft. The primary bentonite marker bed is expected to be exposed near the tunnel crown in the portal face in the westbound cut-and-cover excavation. The fresh rock thickness is only 5 ft. above the crown of the westbound portal at this location, and will create local stability issues especially if combined with unfavorably oriented vertical joints. In the eastbound cut-and-cover excavation, the primary bentonite marker bed will be exposed in the springline to upper one-third of the tunnel horizon. Local stability issues may be encountered if combined with jointing in the sidewall and portal face.

3.3 East End Cut-and-Cover Tunnels

At the east end of the tunnel project area, there is presently a cut-and-cover tunnel planned only for entry to the westbound HOV tunnel. The cut-and-cover tunnel at this location is approximately 587 ft long with depths of excavation varying from approximately 54 to 38 ft. to invert grade. The excavation in this vicinity is anticipated to consist of approximately 1 to 5 ft. of fill, about 5 to 9 ft. of UWFZ materials, and the remainder of the depth comprises fresh Austin Chalk bedrock. The upper bentonite marker bed lies below the invert of the tunnel excavation as planned at this time. However, smaller/thinner bentonite and weak shale partings and seams (up to 2 in.
thick) may be encountered, which when combined with joints perpendicular to the bedding can create localized block instabilities in the sidewalls and portal face.

Groundwater inflow into both the east end and west end cut-and-cover tunnel sections is expected to be minimal, with most infiltration coming from the overburden soils or the UWFZ. The maximum rate of groundwater inflow is estimated to be less than 50 gpm.

Fresh, intact Austin Chalk is not highly susceptible to slaking or degradation. The average slake durability for the Austin Chalk encountered is high to very high, as described in more detail in Section 2.4.3.6. Certain features such as highly argillaceous to shaly beds and seams and particularly the bentonite marker beds, are expected to slake and decompose more rapidly, leading to instability of the surrounding rock mass. The Contractor will need to take into account this potential behavior when selecting excavation techniques, soil and rock support measures, and over-excavation limits. Similarly, the bottoms of rock cuts will require protection from construction traffic and water accumulation to preserve the in situ strength of the rock mass.

3.4 Tunnel Portals

3.4.1 General

The tunnel portals are perhaps the most critical and challenging portions of the underground structures to be developed for the Tunnel Project. Each of the four portals will be exposed in a vertical face to the full depth of the tunnel invert grade and turned under and advanced horizontally under relatively shallow cover and potentially unstable ground conditions. Stabilization of the ground at the brow of each portal is critical to establishing a sound structure for continued development of the tunnel and mitigation of possible third party impacts, such as adjacent utilities and exiting highway structures.

3.4.2 West End Portals

The West End portal for the Westbound HOV Tunnel is at approximate Sta. 219+15 and the West End portal for the Eastbound HOV Tunnel is at approximate Sta. 226+40. The ground conditions at these locations are depicted in Figures 3-1 and 3-2, respectively, and may be summarized as follows.

- Depths to portal inverts vary from approximately 48 to 57 ft.
- Ground cover over the portal crowns varies from approximately 18 to 26 ft., comprising up to 13 ft. of fill (including possible boulders, concrete, asphalt, and wood debris; ref boring T9L), 6 ft. of UWFZ (weathered to fractured fresh limestone), and only 7 to 10 ft. of relatively sound, fresh Austin Chalk with good to excellent RQD.
- Austin Chalk throughout the portal horizons is generally sound with RQD's ranging from good to excellent.
• The upper bentonitic marker bed (5.5 to 9 in. thick) occurs at crown level of the Westbound portal, and in the arch approximately 9 ft below the center of the crown in the Eastbound tunnel.

• Other 1/4 to 1/2 in thick bentonitic shale seams lie within approximately 2 to 5 ft. above and below the center of the crown and will be present in the upper arch section.

• Still other soft shale seams are located near the tunnel springline below the arch and will be present in the face and sidewalls of the portal opening.

These bentonitic and soft shale seams when combined with unfavorably oriented joints, such as high angle longitudinal joints and/or high angle transverse joints will create unstable blocks in the tunnel crown and arch, and possibly unstable slabs in the face of the portal excavation. The presence of high angle joints and fractures will need to be examined by angled (and possibly oriented core borings) during the Phase II investigations.

3.4.3 East End Portals

The East End portal for the Westbound HOV Tunnel is at approximate Sta 319+60 and the East End portal for the Eastbound HOV Tunnel is at approximate Sta. 305+70. The ground conditions at these locations are depicted in Figures 3-3 and 3-4, respectively, and may be summarized as follows:

• Depth to invert grade of the portals varies from approximately 52 to 62 ft.

• Ground cover over the portal crowns varies from approximately 21 to 32 ft. comprising approximately of 5 ft. of fill, 5 to 6 ft of UWFZ (moderately weathered to fresh fractured limestone), and 11 to 21 ft. of fresh Austin Chalk with excellent RQD.

• Austin Chalk throughout the portal horizon is generally sound with excellent RQD.

• 1 to 3 in. thick Bentonitic shale seams are present at and below the springline level in the Westbound portal and in the upper and mid-level arch level in the Eastbound portal.

• The upper bentonitic marker bed (5-1/2 in. thick) is below the invert at the Westbound portal and at approximate springline level in the Eastbound portal.

The ground conditions at the East End portals are generally more favorable than at the West End portals. At the East End-Westbound portal, there is still only about 11 ft. of fresh, relatively sound Austin Chalk above the center of the tunnel.
crown, however, the vertical section appears to be generally free of any soft shale or bentonitic seams and partings throughout the crown and arch. At the East End-Eastbound portal, the potential exists for significant instability in the upper and mid-level arch sections if weak shale seams are combined with unfavorably oriented fractures.

3.5 Mined Tunnels

3.5.1 General

The entire length of the tunnel will be excavated through the generally good to excellent quality (i.e., RQD) Austin Chalk as further described below. Rock in the tunnel horizon and immediate crown is anticipated to be of sound quality, generally unweathered to slightly weathered, having an average unconfined compressive strength of 2790 psi (ranging between 1200 and 4000 psi).

As described in Sections 2.5.1 and 2.5.2, rock for the eastbound and westbound tunnels is thin to thickly bedded with up to three sets of joints. The bentonite (marker beds 3 to 10 in. thick), bedding joints including the weak shale and bentonitic partings and seams (1/8 to 3 in. thick), and the moderate to high angle shear fractures are the dominant discontinuity features that will create instabilities during tunnel excavation. In addition, the possibility of encountering from one to three faults, particularly in the area between Sta. 260+00 and Sta. 275+00, may result in unstable ground conditions. Further geotechnical investigations will be scheduled during Phase II to examine these features and areas in more detail.

In general, the bedding is oriented with the strike approximately transverse to the tunnel alignment and dips at about 1 to 3 degrees towards the east-southeast.

Some of the ground characterization factors to be considered in the design and construction of the mined tunnels include:

1. The thickness of sound quality bedrock (i.e., rock below the UWFZ) in the tunnel horizon and in the immediate vicinity of the tunnel crown.
2. The proximity of the upper bentonite marker bed to the crown and arch of the tunnel and within the tunnel horizon.
3. Presence of other thinner soft shale/bentonitic partings and seams (<3 in. thick) in the tunnel horizon and immediate tunnel crown area.
4. Presence of moderate to high angle joints or shear fractures or faults and their orientation with respect to the tunnel alignment.
5. Presence of soft bentonitic marker beds and moderate to highly argillaceous beds at tunnel invert level.
The position of mined cross-over tunnels between the eastbound and westbound tunnels with respect to the factors listed above.

The ground character and response of the rock "pillar" left in place during and after mining of the two parallel tunnels.

Groundwater inflow into the mined tunnels is also expected to be minimal. Most of the tunnel will yield no water; and most of the inflow may come from only a few places, such as open rock joints or fractures. Depending on the character of suspected shear zones and/or faults, flush flows of up to 200 gpm may be encountered in the heading at any one location or open fracture. However, it is expected that these flows would dissipate rapidly due to the lack of any source of sustained recharge. Total long term inflow over the entire length of the mined tunnel is estimated to be less than 300 gpm.

3.5.2 Summary of Observed Features

At the West end of the mined tunnels, the ramp portions of the Westbound and Eastbound tunnels lie between approximate Sta. 219+00 and Sta. 237+00.

- Within this reach, the thickness of fresh, sound rock above the tunnel crown varies from about 6 ft to over 35 ft, as the ramps plunge toward the east

- The Primary Bentonite Marker Bed will be at or near the crown in both tunnels for several hundred feet, being a maximum of only 15 ft. above the crown

- Only a few scattered shear fractures were observed in this reach, however, several minor soft shale and bentonite partings and seams are present and generally widely spaced throughout the tunnel horizon and crown.

Between the separate ramp portions of the westbound and eastbound tunnels, the two parallel tunnels share a common profile for over one mile (5800 ft.) between approximate Sta. 237+00 and Sta. 295+00.

- In this reach the thickness of sound, fresh Austin Chalk is generally greater than 25 ft. above the tunnel crown throughout

- The Primary Bentonite Marker Bed is generally less than 15 ft. above the crown and is alternately present 5 ft above the crown to 5 ft. below the crown in the suspected faulted area between approximate Sta. 260+00 and Sta. 275+00

- The tunnel horizons will cross through the Primary Bentonite Marker Bed between approximate Sta. 273+00 to Sta. 293+00, to the east of which the tunnel inverts will be above the marker bed
In addition to the Primary Bentonite Marker Bed, several minor soft shale and bentonite partings and seams are present and generally widely spaced throughout the tunnel horizon and crown.

8 to 10 ft. thick high angle shear zones were observed in borings T18C (approx. Sta. 253+00) and T23C (approx. Sta. 273+50).

The shear zones and possible faults, when combined with the primary bentonite marker bed, will present potentially unstable conditions in the crown and upper arch, particularly where the marker bed is in close proximity to the tunnel opening (i.e. ± 10 ft. of the tunnel crown).

At the East end of the mined tunnels, the ramp portions of the Westbound and Eastbound tunnels lie between approximate Sta. 295+00 and Sta. 319+60.

Within this reach the thickness of fresh, sound rock above the tunnel crown is generally 25 ft. or greater along the Eastbound ramp; and varies from over 30 ft. to approximately 10 ft. along the Westbound ramp.

The Primary Bentonite Marker Bed lies more than 5 ft. below the tunnel invert in both tunnels.

The Upper Bentonite Marker Bed lies within 10 ft. of the crown and transects the tunnel horizon along the Eastbound ramp between Sta. 299+00 and the portal.

The Upper Bentonite Marker Bed is within 5 ft. of the crown in the Westbound ramp between approx. Sta. 300+00 and Sta. 308+00. The marker bed transects the Westbound tunnel horizon from crown to invert between approx. Sta. 308+00 and Sta. 317+00.

Occasional scattered shear fractures and a few smaller (< 3 ft thick) shear zones were observed in the borings in this reach, and several minor soft shale and bentonite partings and seams are present and generally widely spaced throughout the tunnel horizon and crown.

3.6 Previous Experience with Tunnels in Austin Chalk

There has been considerable excavation experience with respect to tunneling in the Austin Chalk in the Dallas vicinity. It is the intent that this document and subsequent issues and/or supplements to the Phase 1 GBR will summarize relevant information with respect to the design and construction issues of a number of these projects as applicable. The following is a list of local projects and their respective means of excavation. A brief summary of four of these projects is outlined below.
3.6.1 Superconducting Super Collider Project

This project was centered in Ellis County, Texas approximately 35 miles south of the central business district of Dallas. This $8 billion project as designed comprised a 54-mile tunnel ring and about 45 vertical shafts and various other underground structures. The following is but a cursory review of a few relevant issues encountered in the design and construction of some of the tunnels before funding for the project was terminated.

- Project Participants (Partial List):
  - Owner/Sponsor: U.S. Department of Energy
  - Prime Engineering/Design Team: PB/MK
  - Tunnel Contractors:
    - Traylor Brothers, Inc./Frontier Kemper Construction, Inc., N20 to N25 Tunnel, 12,517 ft. of tunnel at 15.8 ft diameter, approximate cost $14.4M, and
    - Kiewit Construction Co., N25 to N40 and N40 to N55 Tunnels, approximately 44,000 ft of tunnel at 16 ft. diameter at approximate cost of $24M and $27M, respectively.

- Geotechnical Considerations:
  - Tunnels constructed in Eagle Ford Shale and Austin Chalk.
  - Austin Chalk is light to medium gray microgranular limestone, soft to moderately hard with average unconfined compressive strength of 2300 to 2500 psi, with several bentonitic layers 1 to 2 in. thick; low permeability; piezometric level was slightly artesian in some areas.
  - Two major faults from Balcones Fault System encountered approximately 875 ft. apart, forming a graben structure. Faults oriented northeasterly and dipping 70 degrees.

- Equipment and Excavation (Traylor/Frontier Kemper team):
- Two reconditioned Robbins TBM's, one was 16 5 ft. diameter, the other upsize to 15.3 ft. diameter. An extensive conveyor system was constructed to move tunnel muck.

- Tunnel Production rates were as follows:
  
  - Best Shift - 268 ft
  - Best Day - 472 ft.
  - Best Week - 2074 ft
  - Best Month - 7934 ft.

- Equipment and Excavation (Kiewit Construction):
  
  - New full-face, fully shielded TBM, built by Construction Tunneling Services, Model SSC-N20
  - A rail system was installed for muck handling.
  - Tunnel excavated in Eagle Ford Shale (EFS) and mixed EFS/Austin Chalk was supported by four piece precast segment liner.
  - Tunnel excavated in Austin Chalk was supported in the crown with resin encapsulated rock dowel pattern of 2 to 4 bolts, 6 ft. in length.
  - Tunnel production rates were as follows:
    
    - Best Shift - 101 ft.
    - Best Day - 220 ft
    - Average Rate - 104 ft per day

- Mining was impacted by natural cementitious properties of the Austin Chalk fines generated in boring and muck handling. Build up of wet sticky fines on cutter head impeded flow of muck materials requiring considerable downtime to remove.

- Gassy tunnel conditions of unknown origin were encountered in a portion of the tunnel.

Further information and details of this project are presented in the following references:


Construction of the 1122 / 1130 Tunnels on the Superconducting Super Collider Project, by T.B. Corry, Project Sponsor, Kiewit Construction Company, in 1995 RETC Proceedings, Chapter 27, pp 409 to 423

Underground Facilities for the Superconducting Super Collider, by T.K Lundin, J.E. Shively, and F. Fong, in 1993 RETC Proceedings, Chapter 38, pp.589 to 606
3.6.2 Dallas Area Rapid Transit Tunnel Project

The following discussion outlines the NC-1B portion of the overall underground transit project. This portion of the project included twin 16,700 ft. long tunnels, a principal underground station (City Place), a partially completed station (Knox Henderson), a surface station (Mockingbird), and eight associated ventilation/exit shafts.

- **Project Participants.**
  - Owner. Dallas Area Rapid Transit Authority.
  - Constructors: S.A. Healy Co.; (Stone & Webster Engineering Corp, Construction Managers).

- **Project Cost:** $86.8 million

- **Geotechnical Considerations:**
  - All underground facilities constructed in Austin Chalk.
  - Austin Chalk comprises microgranular limestone with marly (argillaceous) zones and bentonite layers.
  - Weathered limestone thickness was 3 ft at south portal and 6.5 ft at north portal.
  - Average unconfined compressive strengths ranged from 2000 to 3000 psi; and RQD values were greater than 90%.
  - Approximate depth to groundwater was 25 ft.
  - At south portal as much as 20 ft. of groundwater above the Austin Chalk within the alluvial deposits.
  - Joints in area of north portal were vertical, oriented northeasterly, and subparallel to the tunnel alignment.
  - Generally negligible rock permeability with minimal groundwater inflow except at major fault zones and occasionally at some widely spaced fractures.
  - Two types of faults encountered: 1) simple faults consisting of series of single very tight joints with displacements typically less than 5 ft. and with spacing of 100 to 300 ft., approximately 200 observed; 2) major faults with two observed south of City Place Station, one with offset greater than 20 ft, faults were permeable and yielded water.
- **Equipment, Excavation and Construction (Railway Tunnels).**
  - Reconditioned Robbins model 222-183-2 standard open face hard rock TBM, with rebuilt 21.5 ft. cutterhead
  - Continuous conveyor belt system installed to move tunnel muck
  - Tunnel production rates
    - Best Shift (8 hr) 148 ft
    - Best Day (24 hr) 376 ft
    - Best Week (5 days) 1640 ft
    - Best Month 6010 ft
    - Overall average for both tunnels was 194 ft. per day.
  - Initial tunnel support: Sodium silicate rock sealant to mitigate rock degradation; plus pattern bolts comprising four each resin encapsulated #8 rock dowels, 10 ft long at 5 ft centers.
  - When bentonite seams were encountered, they were overexcavated and replaced with concrete.
  - Final tunnel lining. Majority of rail tunnel received 4 in. thickness of polyfiber reinforced shotcrete (5000 psi), rail tunnel invert paved with polyfiber reinforced concrete; 1640 linear ft. of rail tunnel received 12 in. thick concrete lining.
  - Mining of rail tunnels impacted/delayed by encountering inflow of groundwater contaminated with gasoline and cleaning solvents in pair of open faults. Faults were about 50 ft. apart and oriented NW-SE transverse to tunnel orientation. Pool of contaminated water quickly and completely drained when intercepted by the TBM Delay of 3 months to explore, detect, and mitigate source of contamination. Faulted areas were slightly over-excavated then sealed with concrete
  - Later encounter with methane gas and other natural gases from deep-seated sources. Delay of 25 months to upgrade electrical and ventilation systems and switch to explosion proof (permissible) equipment where feasible.

- **Equipment, Excavation and Construction (Underground Stations).**
  - Enlargement of tunnels pilot bored by 21.5 ft. TBM.
  - Modified roadheader: AC-Eickoff 120L cutterhead and boom mounted to Koehring model 6644 hydraulic excavator capable of 40 ft reach.
  - Supported by Caterpillar 973 crawler loader and Wagner ST-5 LHD mockers
  - Drill jumbo Tamrock model Paramatic HS206T with TR400 hydraulic rotary drill used to install 10 to 23 ft. long dowels
- Wet mix polyfiber reinforced shotcrete applied as initial rock support using Schwing model 750 concrete pumps, and Healy built robotic arms attached to bucket of ST-5 LHD.
- Felt cushion and PVC membrane overlay was pinned to initial shotcrete liner to capture groundwater and channel flow to invert drains
- Final lining of 12 in. thick reinforced concrete.

Further details of this project are presented in the following references:


3.6.3 Addison Airport Tunnel Project

This tunnel was constructed during the period from September 1997 to early 1999 and comprises a two-lane vehicular toll tunnel under the Addison Airport, located in the north Dallas metropolitan area.

- Project Participants:
  - Owner: North Texas Tollway Authority (NTTA), Addison Airport of Texas, Inc
  - Prime Engineering Design Team HDR Engineering with Lachel & Associates.
  - Constructors. H B Zachary, Co. and Zachary/Monterey, JV.; (Brown and Root, Construction Managers)

- Project Cost $13 million.

- Geotechnical Considerations
  - 1590 ft. long tunnel, horseshoe configuration, 39 ft. wide by 28 ft. high
  - Advanced entirely through Austin Chalk with average unconfined compressive strength of 2400 psi.
  - Ground cover above tunnel crown was 10 ft. at west portal, 28 ft. under runway; and 18 ft. at east portal
• Equipment, Excavation and Construction:
  - Heading and bench method with 14 ft. top heading via roadheader and 14 ft bench excavated in ten 3.5 ft. passes with modified trenching machine.
  - Mitsui roadheader and modified Vermeer trenching machine with 12 ft wide milling head.
  - Initial tunnel support generally comprised of No. 7 tensioned rock bolts at 5 ft. centers, and 3 in. thick steel fiber reinforced shotcrete.
  - Portals and difficult ground driven by multi-drift top heading method (NATM) and supported with No. 7 fully grouted rebar spiles, No. 7 tensioned rock bolts at 4 ft. centers, 6x6 in.-W4/W4 welded wire fabric, and 6 in. thick steel fiber reinforced shotcrete.
  - 1/4 in. thick drainage fabric and 80-mil thick PVC waterproofing geomembrane applied to shotcrete lining. A finished lining of 12 in. thick reinforced concrete was placed against the geomembrane.
  - After first 80 ft. of top heading, excavation averaged 15 ft per workday.
  - Bench removal averaged 1000 c.y per workday (which equates to approximately 50 linear ft. per work day).
  - Placement of 53 concrete arch sections (30 ft. length) were completed in 65 workdays.

Reference:

3.6.4 Cole Park Detention Vault Project

This project consisted of two access shafts and other ancillary tunnel structures, however, the bulk of the effort comprised of constructing 13 parallel storm water storage vaults, each approximately 865 ft long with a horseshoe cross-section 41 ft high by 24 ft wide with 15 ft wide rock pillars between vaults.

• Project Participants: Owner, City of Dallas, Department of Public Works; Engineering and Design, A H. Halff Associates, Inc., and Jenny Engineering Corp., Construction Manager, HCB Contractors; and Contractor, Granite Construction Co.

• Project Cost $28.1M.

• Geotechnical Considerations:
- Constructed in Lower Chalk unit of Austin Chalk
- No dominant jointing system identified at project site.
- Limestone is medium to soft hardness, moderately argillaceous, with unconfined compressive strength average of 2000 psi.
- Bentonite/bentonitic shale marker bed approximately 10 in. thick.
- Rock RQD generally above 95%.
- In situ, rock density approximately 137 pcf
- Permeability tested at $10^{-8}$ cm/sec.
- Slake durability of limestone ranged from medium to high.

**Equipment and Excavation**

- Multistaged vault excavation with 21 ft. top heading via roadheader and 5 ft. bench lifts via continuous miner.
- Voest-Alpine ATM-70 roadheader, supported by Paurat 169B and Voest-Alpine F6A roadheaders.
- Joy 12CM11-9BX Continuous Miner.
- CAT D7G Dozer and ripper.
- Ground conditions better than expected: only few discontinuities with evidence of extensive calcite filling. Negligible groundwater encountered. No significant deterioration of exposed rock except for thin bands of bentonitic shale.
- Excavated Austin Chalk broke down readily with repeated handling and mechanical action. Developed into sticky mass when mixed with water. Non-stick coatings and linings used in the material handling equipment.
- Bentonite marker bed was encountered in two ancillary tunnel/weir structures, but no significant problems were encountered.
- Tunnel crown support comprised an 8-dowel fan pattern of 10 ft long resin encapsulated dowels at 4 ft centers.
- Support in the ribs/pillars comprised 3 levels of dowels of variable length (13.5 to 17 ft.) at 6 ft centers.
- Final lining of the vault excavations (not required for structural support) comprised 3 in. thickness of 4000-psi shotcrete reinforced with 6x6-W4/W4 welded wire fabric.
- No details were provided as to the excavation progress rates.

Further information and details of this project are presented in the following reference.
4.0 GROUND CHARACTERIZATION FOR MANAGED BOX SECTIONS

4.1 General

This section provides a discussion of the subsurface conditions, as presented in Section 2 of this report and in the Phase I Geotechnical Data Report (GDR), that are anticipated to impact the excavation and construction along the planned managed box alignments. The purpose of this chapter is to convey characterizations of the ground conditions along the alignment in perspective with the proposed structures. The box structure along the East Managed Box alignment, as currently planned, will be almost entirely within the slightly weathered to fresh Austin Chalk. This will also be true for the east portion of the West Managed Box. The middle section will be founded in residual soil, weathered shale, and fresh shale of the Eagle Ford. The west portion will be founded in the alluvium.

A generalized subsurface profile is presented in the GDR for centerline borings, the left (north) borings, and the right (south) borings. Considering the design schematic and these profiles, the invert of the managed box structures will be approximately 20 to 30 ft. below grade. Knowing there will be ramps and that these managed box sections will be constructed from the surface, the complete stratigraphy from the surface soils to the foundation stratum will impact the managed box design and construction.

As additional investigations are carried out, details of ground conditions specific to each alignment will be presented in subsequent issues of this document.

4.2 East Managed Box

At the east end of the tunnel project area, there is presently a cut-and-cover tunnel planned only for entry to the westbound HOV tunnel. The cut-and-cover tunnel at this location is approximately 587 ft. long with depths of excavation varying from approximately 54 to 38 ft. to invert grade. The excavation in this vicinity is anticipated to consist of approximately 1 to 5 ft. of fill, about 5 to 9 ft. of weathered Austin Chalk materials, and fresh Austin Chalk to total depth. The upper bentonite marker bed lies below the invert of the tunnel excavation as planned at this time. The managed box will remain in this geologic environment to the project limits on the east end. The overburden fill and weathered Austin Chalk are relatively thin throughout. The final boring (BE10C) for the East Managed Box shows thicker fill and alluvium strata. At this time, the elevation of the box is not clear at this location; however, it appears the box will be at or near the existing surface grade and excavation into these materials will be limited.

Fresh, intact Austin Chalk is not highly susceptible to slaking or degradation. The slake durability for the Austin Chalk is high to very high. Certain features such as highly argillaceous to shaly beds and seams and particularly the bentonite marker beds are
expected to slake and decompose more rapidly, leading to instability of the surrounding rock mass. Smaller/thinner bentonite and weak shale partings and seams (up to 2 in. thick) may be encountered, which when combined with joints perpendicular to the bedding can create localized block instabilities in the sidewalls and portal face. The Contractor will need to take into account this potential behavior when selecting excavation techniques, soil and rock support measures, and over-excavation limits. Similarly, the bottoms of rock cuts will require protection from construction traffic and water accumulation to preserve the in situ strength of the rock mass.

Groundwater inflow into both the east end and west end cut-and-cover tunnel sections is expected to be minimal, with most infiltration coming from weathered Austin Chalk. The maximum rate of groundwater inflow is estimated to be less than 50 gpm.

4.3 West Managed Box

To maintain similarity in the geologic stratigraphy, the ground is characterized from east to west beginning at the west tunnel portal. As stated, a generalized profile is present in the GDR. The length of cut-and-cover section for the west end of the eastbound HOV tunnel is approximately 295 ft. For the west end of the westbound HOV tunnel, it is approximately 325 ft. The depth of excavation for these portions of the box structures varies from approximately 40 to 55 ft. Both of these excavations will be through similar materials comprising up to 13 ft. of overburden, including fill, residual Austin Chalk soils and up to 14 ft. of weathered Austin Chalk. The fresh Austin Chalk will vary in thickness from approximately 18 to 38 ft. The primary bentonite marker bed is expected to be exposed near the tunnel crown in the portal face in the westbound cut-and-cover excavation. The thickness of the fresh rock is about 5 ft. above the crown at the westbound portal. This thin fresh rock section may create local stability issues especially if combined with unfavorably oriented vertical joints. In the eastbound cut-and-cover excavation, the primary bentonite marker bed will be exposed in the springline to the upper one-third of the tunnel horizon. Local stability issues may be encountered if combined with jointing in the sidewall and portal face. Ground considerations for the fresh Austin Chalk are described in the preceding section. As the managed box continues east from the west portal, it will remain in similar strata to the BW24 boring line at approximate Sta. 160+00. At roadway crossings the depth of the box invert may exceed 45 ft., but for the most part, the managed box will be about 20 to 30 ft. below existing grade.

Around Sta. 170+00, the Austin Chalk begins to thin (taper) to the point where the fresh Eagle Ford appears near the bottom of the borings. At approximate Sta. 163+00, the box invert may be within the fresh Eagle Ford. From this point, the ground conditions changes rather rapidly. The left (north) borings show the invert may stay within the fresh Eagle Ford to about Sta. 135+00. The right (south) borings show the invert may be out of the fresh Eagle Ford and into the weathered Eagle Ford by approximate Sta. 153+00. Although the invert of the box will remain about 20 ft. below grade as it continues east, the geological conditions will continue to change. As the invert moves out of the weathered Eagle Ford, it moves into the residual Eagle Ford overburden soils. The invert will remain in these soils until the transition to alluvium at approximate boring line.
The managed box will remain in alluvial soils to the end of the project.

The Eagle Ford is shale primarily composed of expansive minerals. Some of the behavioral characteristics amplify as the shale weathers. An example is the swelling behavior. The shale has been shown on other projects to exhibit a swelling behavior. When weathered, it can be described as highly expansive. When the swelling movements are restrained, significant swell pressures will develop. The shale ravels, spalls, and slakes very quickly. Joints and other fissures create block stability issues. When combined with hydrostatic pressure, large block movements can occur. Residual friction along such joints has been reported as little as 10°.

4.4 Previous Experience with Roadway Box Structures

Numerous roadways throughout the Dallas area have been constructed using box type structures. Two significant depressed highway projects excavated in the Austin Chalk and the Eagle Ford are Central Expressway (HWY 75) and the George Bush Tollway (HWY 190), respectively. These projects can provide background information on the means and methods for excavation, ground support, and ground behavior.

5.0 DESIGN CONSIDERATIONS FOR TUNNEL SECTION

5.1 General

The purpose of Chapter 5.0 is to discuss issues associated with tunnel design that both the PcE and the CDA teams will need to take into account during planning and layout studies. It is emphasized that these discussions are based on the available information described in Section 1.3. It is also noted that final design for this project will be performed under a design/build procurement scenario, the details of which have yet to be finalized. In any case, this report represents the beginning of geological and geotechnical input for design of the proposed LBJ tunnels.

The Lachel Report referenced in Section 1.3 shows tunnel support provided by rock bolts and shotcrete, with a staged excavation sequence. Two types of support are shown depending on the amount of weathered rock located above the tunnel crown. The "final" lining consists of 18 in. of concrete with a waterproofing membrane. In general, this method of approach seems reasonable subject to clarification by both the PcE and CDA teams. Additional comments about the tunnel construction considerations for this project are given in Sections 7.0 of this report.

5.2 Final Lining

The final lining for this tunnel will consist of cast-in-place, reinforced concrete primarily to provide for the finished surface needed for a large, highway tunnel. In general, the ground conditions uncovered by the Phase I test borings reveal a rock mass that is compatible with this method of approach. Groundwater infiltration into the finished
tunnel will be minimal and will be controlled to a large degree by the waterproofing membrane. In those areas with highly fractured rock, shotcrete can be used to help control ground movement, to minimize water inflow, and to serve as a platform for installation of the waterproofing membrane.

A decision concerning waterproofing must be made relative to the final lining. A complete waterproofing membrane for this project will be expensive, but a "dry" tunnel offers many advantages relative to operations and maintenance. Because rock bolts will be used for crown support, these bolts will need to be covered by shotcrete prior to installation of the membrane. A decision will also need to be made relative to allowing the water to either drain into the invert or to provide a 100 percent enclosure. If drainage is allowed, then the water table in the vicinity of the tunnel will be depressed to tunnel invert within the first year after construction. In some cases, water table lowering could adversely impact third parties in the vicinity of the tunnel and may need to be avoided.

5.3 Portal Design

The portals for these tunnels will be large, will be located fairly high in the rock profile where weathering and fracturing are more prevalent, and will need to be undercut in order to begin tunnel construction. In addition, shale seams and layers of bentonite could result in highly unstable configurations at the portals depending on where those layers are located relative to the crown of the tunnel. It is recommended, therefore, that each portal location be further investigated by test borings, including angled borings and borings drilled into the face of the portal, and that each portal be carefully designed for all possible combinations of rock wedges that could move out of the portal into the open cut excavation. It is anticipated that long, pattern rock bolts with steel mesh and a comprehensive program of shotcreting will be needed to stabilize the portal areas.

5.4 Pillar Design and Cross Passages

As currently envisioned, the project layout includes a large pillar of rock between the westbound and eastbound tunnels intersected by cross passages between the two tunnels. As the tunnels are excavated, additional overburden stress will be imposed on the pillar, which can be further increased as the cross passages are installed. In general, however, the amount of overburden is small and the strength of the Austin Chalk should be satisfactory relative to the imposed loads. It is possible that shotcrete could be used as the final lining for the cross-passages, with or without the inclusion of a cast-in-place concrete lining and a waterproofing membrane and a decision about the final lining for the cross-passages will need to be made in conjunction with the CDA team.

5.5 Adjacent Structures and Utilities

The proposed tunnels are overlain by an active interstate highway and by existing utilities. Fortunately, adjacent buildings owned by others are generally outside the zone of influence of the tunnel structures, but additional investigations will be needed relative to the impact of tunneling on existing highway bridge foundations. Bridge engineers are
extremely cautious about possible settlement of their foundations and tunneling will need to be performed in a manner that takes this caution into account. In addition, utility lines will need to be studied relative to their current condition and sensitivity to disturbances associated with tunneling. Although the possibility of negative impact to existing utilities is small, the consequences of excessive settlement or damage to utilities could be severe and steps must be taken to avoid this possibility. Design studies relative to existing facilities will be particularly important in the vicinity of highly fractured rock and in areas with thick, weathered, and fractured zones near the top of rock. In these areas, rock movements into the tunnel can be large and sudden and some form of ground improvement such as grouting or spilling could be required in those areas where bridge foundations or highly sensitive utilities are located.

5.6 Environmental and Community Impacts

It is our understanding that environmental studies have been conducted by others along the proposed alignment and that the results of those investigations will be incorporated into design. All environmental permits needed for the project must be included with the procurement documents and "surprises" relative to environmental impacts must be avoided to the greatest possible extent. In so far as the subsurface exploration program can be used to help identify potential environmental impacts, then the geotechnical and environmental subsurface investigations should be coordinated as discussed in Chapter 9.

Certain community impacts are also related to geotechnical considerations such as the disposal of excavated materials. In general, the rock materials excavated from these tunnels should be highly satisfactory for use as fill, and TxDOT might be able to utilize this material as part of other construction activities. If working restrictions are imposed on the project as a result of geotechnical or environmental considerations, then appropriate allowances must be made in the cost estimates relative to the cost and time for construction.

6.0 DESIGN CONSIDERATIONS FOR MANAGED BOX SECTIONS

6.1 General

The purpose of this section is to discuss issues associated with design of box structures that will need to be considered during planning, layout, and design of the Managed Box sections along the project alignment. These discussions are based on the current project plan as outlined in the design schematic prepared by HNTB. The design schematic is referenced in Section 1.0.

Significant portions of the managed box alignments are to be depressed below existing grades. Because of the grade changes along the managed box alignments, the cross-sections of these structures will vary significantly depending upon the foundation elevation and the finished grade elevation. The deepest point of the managed box sections will be at the tunnel section portals. Where the foundation and finished grade
elevations are close to the same, there will be little or no wall structures. The base will largely serve only as a pavement section. As these two elevations diverge, with the foundation elevation getting lower relative to the finished grade, a type of box structure will be used. Where the elevation differences are relatively small, a U-shaped structure is assumed with the base serving as the pavement surface and foundation. The walls will serve as the final retaining system. As the elevation difference reaches that minimum height necessary for traffic requirements, a cantilevered partial roof will likely be incorporated along some parts of the alignment. The deepest cross-section condition along the managed box alignment will be a closed, cut-and-cover tunnel box.

6.2 Box Walls / Retaining Walls

Retaining systems will be required during construction and may be incorporated in the final box structure designs. Considerations for temporary retaining systems for construction application are discussed in Section 8.0. Each of four different overburden conditions and two project bedrock geologies combine to present different design challenges for below grade walls (box or retaining).

By design, below-grade walls, particularly box structures will be relatively rigid and not allow lateral movement. Even a design that isolates the base slab from the retaining sidewalls will be relatively rigid and not allow lateral movement. Therefore, the below-grade walls should be designed for at-rest lateral earth pressures. The magnitude of the lateral pressures will depend on the type of soil or rock being retained. Hydrostatic pressure will also need to be addressed in the design. The geotechnical data generally shows the piezometric groundwater level to be near the ground surface.

It is assumed that the below-grade wall design will likely be a system where the final wall structure is cast against or placed against the temporary retaining system. Along most of the managed box alignment, a cast double-form retaining wall backfilled with an imported fill does not appear feasible. Likewise, a mechanically stabilized earth (MSE) type system also appears unlikely.

The design of retaining walls in the four overburden soil types (fill, two-residual types, and alluvial) should be based on standard equivalent fluid pressure loading (triangular distribution). Preliminary design equivalent fluid pressures for the overburden and non-expansive imported soil backfill with hydrostatic water pressure are presented in the following Table 6-1.
### Table 6-1: Preliminary At-Rest Equivalent Fluid Pressure

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Design Moist Density pcf</th>
<th>At-Rest Lateral Earth Pressure Coefficient</th>
<th>Equivalent Fluid Pressure with Hydrostatic Loading psf/ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill (1)</td>
<td>124</td>
<td>0.62</td>
<td>101</td>
</tr>
<tr>
<td>Residual Austin Chalk</td>
<td>124</td>
<td>0.66</td>
<td>103</td>
</tr>
<tr>
<td>Residual Eagle Ford (2)</td>
<td>122</td>
<td>0.74</td>
<td>107</td>
</tr>
<tr>
<td>Alluvium</td>
<td>126</td>
<td>0.66</td>
<td>104</td>
</tr>
<tr>
<td>Imported “Select” Non-Expansive Fill</td>
<td>125</td>
<td>0.55</td>
<td>97</td>
</tr>
<tr>
<td>Granular Fill (3)</td>
<td>125</td>
<td>0.43</td>
<td>89</td>
</tr>
</tbody>
</table>

(1) The designer should expect significant variability in the fill. Final design values may vary accordingly.

(2) Residual Eagle Ford soils can impart additional loading due to their expansive nature.

(3) Granular material shall be an angular crushed stone with less than 5% passing the number 200 sieve.

Retaining systems installed during construction and other non-box site retaining walls may be designed to allow some translational and rotational movement. These structures will be subjected to active lateral earth pressures. Typically, lateral movements on the order of 0.001 to 0.002 times the height of the wall are required to mobilize the active pressures of retained soils. The magnitude of these lateral pressures will depend on the type of soil that is being retained. These pressures will also be impacted by hydrostatic pressure. Unlike the box structures that will likely be designed for undrained conditions, these types of structures may be designed as drained or undrained. Preliminary design equivalent fluid pressures for the overburden and non-expansive imported soil backfill with and without hydrostatic water pressure are presented in the following Table 6-2.

Note that imported “select” non-expansive is a somewhat standardized local fill product. Typical “select” fill materials consists of clayey sands with a liquid limit between 20 and 40, a Plasticity Index between 4 and 15, and between 20 and 45 percent material passing the No. 200 sieve.
## Table 6-2: Preliminary Active and Passive Lateral Earth Pressure

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Design Moist Density pcf</th>
<th>Active Pressure (without hydrostatic loading)</th>
<th>Active Pressure (with hydrostatic loading)</th>
<th>Passive Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Lateral Earth Pressure Coefficient pcf</td>
<td>Equivalent Fluid Pressure pcf</td>
<td>Lateral Earth Pressure Coefficient pcf</td>
</tr>
<tr>
<td>Fill</td>
<td>124</td>
<td>0.46</td>
<td>56</td>
<td>90</td>
</tr>
<tr>
<td>Residual Austin Chalk</td>
<td>124</td>
<td>0.49</td>
<td>61</td>
<td>93</td>
</tr>
<tr>
<td>Residual Eagle Ford (2)</td>
<td>122</td>
<td>0.59</td>
<td>72</td>
<td>98</td>
</tr>
<tr>
<td>Alluvium</td>
<td>126</td>
<td>0.49</td>
<td>62</td>
<td>94</td>
</tr>
<tr>
<td>Imported &quot;Select&quot; Non-Expansive Fill</td>
<td>125</td>
<td>0.38</td>
<td>46</td>
<td>86</td>
</tr>
<tr>
<td>Granular Fill(3)</td>
<td>125</td>
<td>0.27</td>
<td>34</td>
<td>79</td>
</tr>
</tbody>
</table>

Note: (1) Granular material shall be an angular crushed stone with less than 5% passing the number 200 sieve.

The active lateral earth pressures will cause translational and rotational movement of the retaining wall. The translational movements of the retaining wall are resisted by the sliding resistance developed by the contact of the base of the retaining wall foundation and the bearing soils. The frictional coefficient will range from 0.27 for residual Eagle Ford clays to 0.7 for crushed limestone.

For box and U-shaped structures, translational movements and sliding will be resisted by balanced loading conditions on either side of the structure. These lateral loads must be considered in the slab design. For retaining wall structures, additional resistance can be developed by generating passive pressures when the base/foundation with a key is in direct contact with undisturbed soil or rock. The foundation elements of the retaining walls must be cast directly against undisturbed soil or rock to generate passive pressures. Passive earth pressures on the toe of wall foundations, keys, or similar structural members, should be considered for counteracting lateral forces only if the member is placed in direct contact with undisturbed stiff to hard cohesive soil in a "neat cut" excavation. Only long-term dead loads should be considered in calculating the available friction on the foundation base. For shallow retaining structures where the foundation soils may be subject to desiccating effects of the climate, the passive pressure will be limited below a minimum depth of 3 feet below the final grade.
In order to achieve the "without hydrostatic loading" condition for lateral earth pressures for low-permeability walls (concrete, masonry, etc.), a vertical drainage blanket or geocomposite drainage member must be installed adjacent to and behind the wall. The vertical drainage blanket may consist of a free-draining granular fill zone constructed immediately behind the wall. The drainage system used behind the wall should be protected against the infiltration of surface water into the drain system. All drainage systems must be connected to an outlet. It must be noted that the overburden soils and imported "select non-expansive fill" are not free draining.

All surcharge loads should be multiplied by the respective lateral earth pressure coefficient and added to the equivalent fluid pressure to determine the total lateral earth pressures acting on the below-grade walls and site retaining walls.

Waterproofing should be used on walls designed for hydrostatic loading to mitigate the presence of seepage and wet spots on the wall interior.

6.3 Foundations

Several foundation options will be considered during later design phases. As the managed box sections are currently planned, a raft or mat foundation appears to be the most likely for the overburden soils. Retaining walls not designed as monolithic units with the U-shaped and box structures in the Austin Chalk segments may use strip or continuous footings. Along the West Managed Box alignment, cast-in-place, drilled shaft pier foundations may be shown to be an effective alternative foundation type. Drilled shaft piers will be a likely choice for bridges and cantilevered structures. They may be used as retaining systems. They may also prove effective when used as combined retaining systems as tangent or secant walls and axial foundations.

Foundation design for any structure generally considers two primary factors: allowable bearing capacity and deformation/settlement. Often, the final foundation design is controlled by deformations rather than bearing capacity. The deformation/settlement design criteria is commonly based on limiting downward movements due to axial loading. Although this will be the case for significant portions of the managed box sections, the residual soils, weathered Eagle Ford shale, and to a lesser extent, fresh Eagle Ford shale will differ. Deformations are not likely to be in the form of settlement, but rather as uplift or heave due to the expansive nature of these materials.

Bearing capacities of structures founded on rock are primarily dependent upon the discontinuities with respect to the foundation width, discontinuity orientation, and condition. Thus, the bearing capacity considers the four basic rock mass conditions: intact, jointed, layered, and fractured. Intact rock strength is influenced by rock type. The rock type controls whether the intact mass is brittle or ductile. Considering the loading types possible for the planned structures, the Austin Chalk should behave as a brittle rock and the Eagle Ford more as a ductile rock.

With an average shear strength in excess of 1300 psi, the slightly weathered and fresh Austin Chalk will provide ample intact rock mass bearing capacity and exhibit limited deformations. Of the three discontinuity criteria, jointed, layered, and fractured, the Ground...
Characterization Section of this report shows these criteria will have limited impact on foundation design in the Austin Chalk.

The slightly weathered Eagle Ford shale will provide significantly lower allowable bearing capacity. The average shear strength of the Eagle Ford shale is about 90 psi. Because of this low rock strength and its ductile properties, design of foundations in the fresh Eagle Ford should use soil analytical methods. Accordingly, the rock discontinuity methods may not be applicable.

Bearing capacity analysis in the overburden should be based on typical soil analytical methods. The alluvium should behave as a normally consolidated, nearly or fully saturated, layered fine-grained, soil. The fill should also behave as a normally consolidated fine-grained soil, but will be more random than layered, will likely be partially saturated, and will have areas with larger particle sizes. The residual soils and weathered shale will be primarily over-consolidated, partially saturated, expansive fat clays. The principal difference between the residual Austin Chalk and residual Eagle Ford shale materials will be the expansive clay content. The residual Eagle Ford clays are significantly more expansive and produce much greater swelling pressures than the residual Austin Chalk clays.

Calculation of allowable bearing capacity is based on shear strength, foundation shape, depth, inclination, and other ground factors. At this stage of the project, the available data only supports analysis of the shear strength and some ground factors. Within these limitations, Table 6-3 provides a range of preliminary allowable bearing information for the managed box alignments.

The bearing capacities in the slightly weathered to fresh Austin Chalk and Eagle Ford should exceed the requirements for a mat or raft type foundation. As a result, continuous footing or drilled shaft pier foundations should be considered to support the retaining structures. This will permit the base slab to be designed as a pavement section only and thereby, greatly reduce the structural requirements.

Deformation may prove to be the critical criterion for many of the specific foundation designs. Downward deformation in the form of settlements can be limited to 0.5 in or less in the slightly weathered to fresh Austin Chalk and Eagle Ford. Settlements in the weathered Austin Chalk can likely be limited to 1 in. or less. With the exception of very heavily loaded foundations, deformation in the residual soils and the weathered Eagle Ford will be in the form of heave. Potential vertical movement in the form of heave is generally limited to 4 in or less in the residual Austin Chalk. Potential vertical movements in the residual and weathered Eagle Ford is typically on the order of 3 to 6 in and has been known to exceed 12 in. Swell pressure associated with these movements can vary from negligible to more than 6,000 psf.
Table 6-3: Preliminary Allowable Bearing Information

<table>
<thead>
<tr>
<th>Bearing Stratum</th>
<th>Mats of Rafts psf</th>
<th>Footings psf</th>
<th>Drilled Shaft Piers psf</th>
</tr>
</thead>
<tbody>
<tr>
<td>Austin Chalk</td>
<td>Limited only by intact rock shear strength</td>
<td>10,000 - 20,000</td>
<td>40,000 to 100,000</td>
</tr>
<tr>
<td>Eagle Ford</td>
<td>Limited only by intact rock shear strength</td>
<td>5,000 - 8,000</td>
<td>8,000 to 25,000</td>
</tr>
<tr>
<td>Weathered Austin Chalk</td>
<td>Limited only by intact rock shear strength and discontinuities</td>
<td>3,000 - 12,000</td>
<td>3,000 - 20,000</td>
</tr>
<tr>
<td>Weathered Eagle Ford</td>
<td>4,000 - 9,000</td>
<td>2,500 - 5,500</td>
<td>3,000 - 12,000</td>
</tr>
<tr>
<td>Fill</td>
<td>3,000 - 8,000</td>
<td>1,500 - 3,500</td>
<td>Not applicable</td>
</tr>
<tr>
<td>Residual Austin Chalk</td>
<td>3,000 - 8,000</td>
<td>1,500 - 3,500</td>
<td>Not applicable</td>
</tr>
<tr>
<td>Residual Eagle Ford</td>
<td>3,000 - 8,000</td>
<td>1,500 - 3,500</td>
<td>3,000 - 6,000</td>
</tr>
<tr>
<td>Alluvium</td>
<td>1,500 - 6,500</td>
<td>500 - 2,000</td>
<td>1000 - 4,000</td>
</tr>
</tbody>
</table>

Retaining wall foundations are typically subjected to non-uniform pressure across the foundation, and possibly negative pressure under a portion of the foundation due to overturning moments induced by lateral earth pressures. The foundation maximum pressure induced by the foundation loads should be limited to the allowable bearing pressures and not the average pressure under the foundation base.

6.4 Pavements

Much of the pavement along the managed box sections will be placed on a subgrade in slightly weathered to fresh Austin Chalk. This stratum will provide substantial subgrade support for pavements. Likewise, the slightly weathered to fresh Eagle Ford will provide substantial subgrade support. Although the weathered Austin Chalk will be somewhat variable, these materials are generally very satisfactory for pavement subgrades. The fill and alluvium will prove to be much softer than the aforementioned materials and will likely require a substantially thicker pavement section.

The residual overburden soils and the weathered Eagle Ford have proven to be problematic for pavements near the project area. Because these soils consist of expansive materials, movement within the pavement section should be expected. Shallow subgrade treatments are often needed for improvement of the subgrade for load-carrying purposes. These shallow treatments provide only minimal reduction in expansive soil movements. An expansive soil modification depth of several feet is usually necessary to significantly reduce the potential vertical movements from expansive soils. The standard method of estimating these movements is TxDOT Test
Method Tex-24-E, Determination of Potential Vertical Rise (PVR). The calculated PVR for dry, average, and wet moisture conditions is provided in Table 6-4

<table>
<thead>
<tr>
<th>Type</th>
<th>Dry PVR (inch)</th>
<th>Average PVR (inch)</th>
<th>Wet PVR (inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weathered Eagle Ford</td>
<td>42 – 56</td>
<td>30 – 37</td>
<td>17 – 24</td>
</tr>
<tr>
<td>Residual Austin Chalk</td>
<td>27 – 54</td>
<td>1.9 – 36</td>
<td>0.9 – 2.2</td>
</tr>
<tr>
<td>Residual Eagle Ford</td>
<td>45 – 61</td>
<td>3.2 – 4.1</td>
<td>1.8 – 2.7</td>
</tr>
</tbody>
</table>

With these residual overburden soils, there is the risk of post-construction vertical soil movement and resultant pavement maintenance costs. To help mitigate these movements, proper drainage should be provided both during and after construction. Particular emphasis should be given to areas where the pavement is placed directly adjacent to structures or other facilities. A key consideration in the pavement design is the use of joints and seals. The majority of pavement heaving problems are the result of moisture changes after construction. The pavement should be maintained properly, including the use of a flexible joint material to seal cracks, which can occur during the life of the pavement.

Clay soils generally have poor subgrade characteristics. A commonly used method to improve the strength properties is to treat the clayey soils with hydrated lime. Lime treatment is used in order to: produce a non-pumping subbase; increase its strength properties; provide a uniform, stable, and permanent support, and provide a working platform for construction equipment. Lime treatment of subgrade soils under concrete pavements is commonly used in the project area where subgrade soils with PIs greater than 15 are encountered.

The use of sand or select fill as a leveling course below pavement in expansive clay areas should be avoided. The porous soils can allow water inflow between the pavement and subgrade, causing heave and strength loss within the subgrade soil.

Typical subgrade improvement include:

1. **Lime Treatment** - Follow Texas DOT Item 260 and apply the hydrated lime at an estimated application rate as determined at time of construction.

2. **Crushed Limestone Base Material** - Texas DOT Item 247, Type A, Grade 2 or better.

3. **Compacted Subgrade** - Scarify the existing soils after removing vegetation and achieving final grades, and recompact.
In some instances, lime treated subgrades have experienced sulfate-induced heave. These problems are most common in residual and weathered Eagle Ford materials. Laboratory testing can be performed to indicate levels of soluble sulfates. The tests should be run on specific samples collected from zones to be lime treated once final grades are established. Additional soluble sulfate testing should also be considered during the construction phase on selected samples from the final subgrade area prior to lime treatment to confirm the sulfate levels.

6.5 Embankments

We assume embankments may be constructed where project elements pass over existing intersecting streets, ramps, and other elevated areas. At these locations, the embankments may be retained using MSE walls.

In our experience with local overburden soils, permanent unreinforced slopes should be gentle and preferably should not exceed about 3 horizontal to 1 vertical (3H:1V). Embankments constructed with residual and weathered Eagle Ford shale material often require slopes of 5H:1V. Special consideration should be given to any steeper slopes that exceed 15 feet in height. Slopes of these magnitudes should have a factor of safety of at least two.

Depending on the type of soil used in the embankment construction, the vegetation cover on the embankment slopes, the climatic conditions such as whether the exposed slope soils are permitted to desiccate and crack, localized slope failure is somewhat common. If desiccation cracking of the exposed slopes is limited, the possibility of localized shear failure will be minimized.

Because of the relatively stiff consistency of the overburden soils, the settlement of the subgrade soils under the weight of the embankments will be limited when compared to the internal settlement of the embankments themselves. The magnitude of settlement and time-period for this settlement to occur is a function of the compactive effort, dry density, plasticity, permeability, and drainage conditions. Embankments constructed with the site overburden soils, placed as engineered fill, will settle about 1 to 2% of its fill height. The primary settlement normally occurs over a period of several months to two to three years depending on the type of drainage conditions that exist.

6.6 Adjacent Structures and Utilities

The proposed managed box alignments are planned where there are several existing highway structures and numerous utilities. The excavation and retaining system designs should be made in such a manner to provide for their protection during construction. Structures, pipelines, or facilities should be protected from loss of end bearing or lateral support. Detailed analysis and possibly additional investigations will be needed relative to the impact of the managed box excavation on bridge foundations. The designers need to be extremely cautious about possible settlement of bridge foundations and existing utilities. By design, the possibility of negative impact to those facilities can be minimized. The consequences of excessive settlement or collapse would be severe and
steps must be taken to avoid this possibility. Design studies in highly fractured rock, in areas of a thick weathered zones, and in the alluvium will be particularly important.

6.7 Environmental and Community Impacts

Considerations will be similar to that for tunnels, see Section 5.6.

7.0 CONSTRUCTION CONSIDERATIONS FOR TUNNEL SECTION

7.1 General

Successful tunnel construction involves excavation and support of the ground prior to installation of the final lining. As with tunnel design, the PeC will establish appropriate contractual requirements for those activities prior to award to the CDA team. In general, the PeC must develop a contracting mechanism that provides adequate controls for third party impacts and design requirements, but which also provides ample flexibility for the contractor relative to the means and methods of construction. Given below, therefore, are comments about construction considerations that will assist the PeC and CDA Team in making those decisions.

7.2 Tunnel Excavation

Although the Austin Chalk is compatible with all methods of rock excavation including drilling and blasting, road headers and tunnel boring machines (TBM), it is our understanding that drilling and blasting will not be allowed because of concerns about blasting vibrations and safety. Roadheaders are highly satisfactory for the excavation of short tunnels or for tunnels with noncircular cross-sections and the Lachel cost estimate is based on the assumption that roadheaders will be used for all excavation. TBM's are the preferred method of rock excavation for longer tunnels with circular cross-sections and could be used to produce pilot bores or as part of a multiple drift approach to construction.

It is recommended that specifications be developed for the use of both roadheaders and TBM's for this project in order to provide the CDA team with maximum flexibility for rock excavation. The Austin Chalk has been shown to be highly stable during excavation for other tunneling projects in the Dallas area and face stability should not be a problem for this project even for the large openings needed for this highway project.

7.3 Initial Support

In general, the ground surrounding these proposed tunnels will be stable as the tunnels are advanced, but a few areas of concern must be highlighted.

1. It is highly likely that the main tunnels are crossed by one or more faults as described in Chapter 3.0. Although the faults or shear zones themselves...
may be only ten feet thick or less, they may occur in groups and can cross the tunnel at angles that could result in several hundred feet of tunnel being adversely impacted by the faults. For instance, as the fault crosses the tunnel, instability will occur along the sidewall, followed by the crown, and then in the pillar between the two tunnels. Additional rock bolt support will be needed to support all areas of poorer quality rock.

2. Test borings have revealed numerous horizontal beds of bentonite and shale that represent planes of weakness in the rock mass. For small diameter tunnels, these planes of weakness would not be a serious concern, but for the large diameter tunnels needed for highway construction, significant volumes of unstable rock could be generated near the crown and quarter arches of the tunnel. Of additional concern is the fact that the worst location for these planes of weakness would be 2 to 3 feet above the crown of the tunnel where they could not be observed by persons working in the tunnel. Large tunnel cross-sections also cause more deformation of the rock mass itself and this deformation results in additional movement along planes of weakness and additional fracturing along thin beds of isolated rock.

The best way to control both of these concerns is to excavate the tunnels in stages and to support the rock mass with pattern bolts. The bolts must be designed to be long enough to penetrate planes of weakness in the rock mass and to establish a stable arch of rock around the openings. Rock bolt support can be further augmented by using steel mesh, steel straps, and/or shotcrete in the more heavily fractured areas of the tunnel. Rock bolt support in the sidewalls of the tunnels can be installed, as required, with special emphasis on the need to provide a stable pillar between the tunnels if a rock pillar becomes part of the final design.

Such a method of support was described in the Lachel Report with tunnel cross-sections showing two types of support as a function of the thickness of weathered rock above the crown. As described in the report, the more expensive Type II excavation sequence with longer bolts and more shotcrete would be used when the thickness of unweathered rock above the crown was less than 30 feet. In accordance with the Phase I subsurface investigation this condition occurs for approximately 25% of both tunnels as compared to 75% of tunneling with Type I support. These numbers are substantially different than the Lachel estimate, which assumed only approximately 7% of the tunnel would be Type II support. Hence, some adjustment of the cost estimate could be made at this time based on the Phase I data.

7.4 Groundwater Control

As with rock support, groundwater control for these tunnels should not be a major problem for these tunnels except in the areas of faulted ground. Other tunnels built in the Austin Chalk have experienced high volumes of initial inflow from water stored in the faulted ground. This inflow could also be augmented from water stored in the upper weathered/fractured zone if the fault zones extend to the top of rock. In general, groundwater inflow into the tunnels will consist of a high volume of initial inflow from the fractured rock followed by small amounts of steady state seepage. The steady state
seepage conditions, however, may be of sufficient concern to require the use of a waterproofing membrane as part of the final lining. As discussed above under design considerations, the use of a waterproofing membrane results in additional cost and construction considerations for the project.

7.5 Construction Access

At the time of this report (May, 2004), the geotechnical team has very little information about construction access for this project. In general, the CDA team should be provided with as much access as possible, preferably at the portals in order to implement tunneling operations. If this is not possible, then it may not be possible to use TBM’s for the excavation of pilot bores or starter tunnels. In addition, depending on third party impacts associated with the sites of access, working restrictions may be imposed that could result in significant cost and scheduling impacts to the project and design. Serious consideration must continue to be given to site access throughout the planning and design process.

7.6 Instrumentation and Monitoring

There are three primary reasons why construction-monitoring services are needed during construction of a tunnel. The first is to make certain that the project is being constructed in accordance with the plans and specifications, the second is to make certain that the tunneling operation is not causing difficulty for adjacent third parties, and the third is to accumulate a body of factual information for use in the event of a claim for differing site conditions. In order to maintain control of a tunnel project, it is necessary to observe what is happening in the tunnel on a continuous basis. For example:

- What type of material is being excavated from the face and how is this material behaving during excavation?
- How much water is flowing into the tunnel and from what locations?
- If work ceases for any reason, what is the reason for the downtime and what else is the contractor doing during this period?
- How much temporary support is being installed as the tunnel advances?

Instrumentation observations can be divided into two groups: those observations of adjacent properties and overlying utilities, and those of the tunneling operation itself. A representative number of adjacent structures, such as bridges, should be monitored with settlement markers and by surveying periodically throughout the tunneling operation. This information can then be used in conjunction with the preconstruction survey to show the degree of damage, if any, that was caused to adjacent structures by tunneling operations. For large utilities at the site, a cased subsurface settlement marker can usually be installed on the utility itself. By monitoring the settlement marker, it is then possible to document if the utility moved or not as a result of tunneling.
Arrays of settlement markers installed both at the ground surface, and below the ground surface, can be used to monitor the behavior of the ground above the tunneling operation. For a tunnel, the single most important indicator of ground behavior is the amount of ground movement that takes place directly above the crown of excavation. If this ground is stable, then the tunneling operation can be shown to be proceeding in a satisfactory manner.

It is very important that several different types of instrumentation be included in the project. The instrumentation should include surface and subsurface settlement points, multi-position boreholes extensometers (MPBX), utility monitoring points, structure monitoring points, piezometers, convergence points, and inclinometers. The purpose of the ground settlement points (both at the ground surface and underground) are to obtain basic information on ground behavior, to evaluate tunneling performance, to verify that contract settlement criteria are achieved, and to provide information on the movement of nearby facilities, which may be effected. The purpose of MPBX's is to measure small movements in rock above, below, and adjacent to the tunnel opening at selected distances. The purpose of the structure and utility monitoring points is to directly determine whether there has been movement, and the nature of that movement. As during the design stage, piezometers are used to monitor changes in the groundwater level and water pressure as a result of tunnel excavation. Inclinometers will measure lateral deflection in retaining systems. Convergence points are used to measure deflections in the crown, floor, and sidewalls of the tunnel. In general, instrumentation is concentrated in test sections at the beginning of tunneling, at major changes in ground conditions, and at regular intervals to verify tunneling and ground performance.

It is also very important that all information obtained from the instrumentation program be processed and distributed to all parties on a regular basis so that decisions can be made about the appropriateness of tunneling activities. One of the most important aspects of successful tunneling is to monitor ground movements on a regular basis throughout all phases of excavation and ground support.

7.7 Subsurface Investigations

For a design/build procurement approach to construction utilizing a CDA team, some subsurface investigations become part of construction. Although a large amount of subsurface information will be provided to the CDA team, it is imperative that a specification be provided in the contract document that requires the CDA team to perform its own subsurface investigations both for comparison to what is provided in the contract, and to provide direct input for all decisions concerning the means and methods of construction as proposed by the CDA team. The specification should be written to ensure data collection that is consistent with that performed by TxDOT. Successful design/build construction is highly dependent on the avoidance of differing site conditions claims to the maximum possible extent. Hence, it is extremely important for the CDA team to hire its own geotechnical engineer to perform its own subsurface investigations as necessary to avoid differing site conditions claims. These activities should be coordinated with the Owner’s geotechnical representatives on a regular basis.
8.0 CONSTRUCTION CONSIDERATIONS FOR MANAGED BOX SECTIONS

8.1 General
The purpose of this section is to discuss issues associated with construction of box structures that will need to be considered during planning, layout, and design of the Managed Box sections along the project alignment. These discussions are based on current project plans as outlined in the design schematic prepared by HNTB. This is referenced in Section 10.

8.2 Excavation
Conventional hydraulic excavators should be adequate for soil and rock excavation for open cuts along the managed box alignments. Drill and blast methods for rock excavation are rarely employed for the slightly weathered and fresh Austin Chalk and will likely be prohibited for this project. Therefore, the contractor can expect to encounter areas where competent rock may prove largely non-nppable and additional mechanical breakage will be required to facilitate excavation. This mechanical breakage can be facilitated using hydraulic or pneumatic hammers mounted on excavator frames in some areas of the slightly weathered and fresh Austin Chalk. Alternatively, contractors may want to consider using rock saws to excavate slots in the bedrock and then rip the remaining material with large excavators.

It is expected that a combination of these methods will be used depending upon the geometry and size of the excavation. When planning this work and selecting equipment, potential contractors should take into account the intact rock properties summarized in this GBR, the GDR.

Significant portions of the base slab of the Managed Box sections will be founded on slightly weathered to fresh Austin Chalk. To reduce the effects of non-uniform rock subgrade conditions, the excavation base should be leveled with a 6 in. thick layer of crushed stone. In the other areas with Eagle Ford and overburden, the surface can be neat cut. Excavated Austin Chalk materials should be suitable for this crushed stone. A mud-mat may also be considered. The mud-mat would provide a level working surface and should not be considered as a structural element of the base slab. Depending upon the design, a free draining crushed stone may be required to control underslab groundwater seepage. If this becomes necessary, the mud-mat should be placed above the crushed stone. Processed Austin Chalk generally does not prove suitable for free draining purposes.

For drilled shaft piers, a drilling rig of sufficient size and weight with properly equipped rock cutting teeth will be necessary for drilling through the slightly weathered and fresh Austin Chalk. Soils augers are generally satisfactory for drilling through the project soil and rock types. Occasionally, very hard concretions of sufficient size in the Eagle Ford can create difficult drilling. When encountered, rock-coning methods may be required to achieve penetration. The installation of drilled piers requires special attention be given to: 1) verticality of the shaft excavation, 2) identification of the bearing stratum, 3)
minimum pier diameter and penetration, 4) smear zones and loose cuttings are satisfactorily removed, 5) correct handling of groundwater seepage, if encountered, and 6) related items. During construction of the drilled shafts, care should be taken to avoid creating an oversized cap ("mushroom"), particularly near the ground surface. A "mushroom" at the top of the drilled shaft could be lifted by heave of the expansive soils. The contractor can expect drilled shaft piers of 6 ft diameter or less to stand open for up to 8 hours in all project overburden soils except alluvium. Drilled shaft excavations in alluvium will remain open for very short periods of time and may exhibit rapid caving.

8.3 Temporary Excavation Support

The slightly weathered to fresh Austin Chalk and Eagle Ford can be excavated with vertical walls. The Austin Chalk should require only spot bolting to support individual rock blocks. Because it will require special attention to identify these blocks, a series of pattern bolts may be prudent. The same is largely true for the fresh Eagle Ford; however, it will deteriorate very rapidly when exposed and begin to spall. Therefore, at a minimum, shotcrete or other additional support will be necessary. The weathered Austin Chalk will generally stand on a 1H:1V for temporary construction purposes. Temporary construction slopes of 2H:1V are often used in the residual soils and fill. Temporary construction slopes may not be feasible in the alluvium, and estimates are impractical.

If the design and construction requirements do not permit temporary construction slopes, or if the excavations are surcharged, or there are adjacent roadways, bridges, utilities, etc., additional excavation support will be required. Due to the proximity of such features, consideration must be given to how the construction is sequenced and the need for temporary excavation support. It is critical that potentially damaging ground movements do not occur.

Lateral pressure for temporary excavation support will be highly dependent upon the construction means and methods. However, for preliminary design, apparent lateral pressures for the fill, residual soils, and weathered Eagle Ford can be estimated at 0.2γH to 0.4γH (γ = moist unit weight (density)) using trapezoidal loading. The alluvium may develop temporary loads approaching full triangular loading.

8.4 Earthwork

Preparation of the site for construction operations should include the removal and proper disposal of all obstructions that would hinder preparation of the site for construction. These obstructions may include abandoned structures, foundations, debris, organic matter, and other loose debris.

All trees, stumps, brush, abandoned structures, roots, vegetation, rubbish, and any other undesirable matter should be properly removed and disposed of. All surface vegetation and organic soil should be removed to a minimum depth of six (6) inches and the exposed surface should be scarified to an additional depth of at least six (6) inches. The organic material may be suitable for top soil.
After individual work areas of the site have been excavated, prior to fill placement or foundation construction, the excavated surface of all soil and weathered rock should be proofrolled. Proofrolling can be completed with a loaded, tandem-axle dump truck, scraper, or other heavy, rubber-tired vehicle weighing at least 25 tons. The proofrolling should consist of several overlapping passes in mutually perpendicular directions over a given area. The subgrade in areas where rutting or pumping occurs during proofrolling should be removed and replaced with on-site soils used for grade fill. Unless special design provisions are made, all fill should be placed in 8-inch loose lifts and compacted. The fill materials should be placed in level, uniform layers, which when compacted, should have an appropriate moisture content and density relationship. Each layer should be thoroughly mixed during the spreading to insure the uniformity of the layer.

The exposed subgrade and each lift of compacted fill should be tested, evaluated, and reworked, as necessary. Each lift of fill should be tested for density and moisture content at a frequency of one test for every 5,000 square feet of compacted fill and one density and moisture content test for every 300 linear feet of compacted utility trench backfill.

Compaction equipment should be of such design that it will be able to compact the fill to the specified density. Compaction of each layer shall be continuous over its entire area.

8.5 Groundwater Control

Groundwater elevations have been monitored in a series of standpipe and vibrating wire piezometers installed across the site and at several relative depths. Based on the results of groundwater elevation monitoring program, the groundwater regime at the project site consists of a single water table within +7 ft. and -12 ft. of existing grade.

The design groundwater level should be set at the ground surface. Depending upon the degree of drainage, interconnection with the shallow groundwater, and the soil or rock type, groundwater control requirements will vary significantly across the Managed Box alignment.

Groundwater control in the open cuts for the Managed Box alignment is expected to be accomplished from open sumps at the base of the excavations with pumps discharging to appropriate disposal facilities at the ground surface.

Based on the results of water pressure tests in the rock at the site, groundwater inflows into excavations are expected to be very small with two exceptions. Although not identified in the geotechnical investigation to date, significant inflows are known to occur in the weathered Austin Chalk. The flows more frequently occur near the contact between the weathered and slightly weathered materials. In some cases, these flows can last for a rather long period (more than a week). On rare occasions, these flows can appear as steady state conditions. The second case for significant inflow will be in the alluvium. Unconfined aquifer steady state flows should be assumed. The CDA must anticipate that potential inflows will not be evenly distributed along the length of the excavations.
Because of the shallow nature of the cuts for the Managed Box alignments, surface precipitation and runoff are expected to have a nearly instantaneous impact on infiltration rates and groundwater inflows should be expected to increase after rainstorm events.

Groundwater conditions indicate that temporary casing will be required for the drilled pier installations in the alluvium. Temporary casing is also likely to be required to prevent caving due to the more alluvial granular strata. The casing must be installed a sufficient distance into the bearing stratum to insure an adequate seal, normally a distance of 2 to 3 ft is adequate.

8.6 Construction Access
Considerations similar to that for the Tunnel, see report section 7.5.

8.7 Instrumentation and Monitoring
Considerations similar to that for the Tunnel, see report section 7.6

There are three primary reasons why construction-monitoring services are needed during construction. The first is to make certain that the project is being constructed in accordance with the plans and specifications, the second is to make certain that construction operations are not causing difficulty for adjacent third parties, and the third is to accumulate a body of factual information for use in the event of a claim for differing site conditions. In order to maintain control of the project, it is necessary to observe operations on a continuous basis. For example:

Instrumentation observations can also be divided into two groups: those observations of adjacent properties, nearby structures and utilities, and those of the excavation operations and project elements. All adjacent structures should be monitored with settlement markers and by surveying periodically throughout construction. This information can then be used in conjunction with the preconstruction survey to show the degree of damage, if any, that was caused by construction activities.

It is very important that several different types of instrumentation be included in the project. The instrumentation should include surface and subsurface settlement points, multi-position boreholes extensometers (MPBX), utility monitoring points, structure monitoring points, piezometers, convergence points, and inclinometers. The purpose of the ground settlement points is to obtain basic information on ground behavior. The purpose of the structure and utility monitoring points is to directly determine whether there has been movement, and the nature of that movement. Throughout design and construction, piezometers are used to monitor changes in the groundwater level and water pressure due to excavation. Instrumentation should be concentrated in test sections at the beginning of operation, at changes in ground conditions, and at regular intervals to verify tunneling and ground performance.
It is very important that all information obtained as a result of the instrumentation program be processed and distributed to all parties on a regular basis so that decisions can be made about the appropriateness of tunneling activities. One of the most important aspects of successful tunneling is to monitor ground movements on a regular basis throughout all phases of excavation and ground support.

8.8 Subsurface Investigations

For a design/build procurement approach to construction utilizing a CDA team, the subsurface investigations become part of construction. Although a large amount of subsurface information will be provided to the CDA team, it is important that the CDA team perform its own subsurface investigations for the Managed Box sections.

9.0 RECOMMENDATIONS FOR ADDITIONAL INVESTIGATIONS

9.1 Reports

Prepare a revised GDR to include any subsequent data. Prepare a revised GBR concurrent with the design phase to serve as the Final GBR-B. It is imperative that these documents be based on the design as it is advanced by the Procurement Engineering Consultant.

9.2 Exploration and Field Studies

Complete the remaining 43 program borings planned along the entire Section 4-West IH-635 Corridor. These borings will provide a final boring spacing on the order of 400 ft. as recommended for this project. Under the current planned program, twenty (20) of these borings will be located in the immediate vicinity of the Tunnel project. Some of these borings are needed to provide information specific to the location of the four tunnel portals. These portals are seen as key elements in the development of the Tunnel project. Valuable additional data will be collected using angle borings with oriented core at the portal areas. The proposed program includes four borings perpendicular to the tunnel, two angled from south to north and two from north to south. The remaining borings would be drilled parallel to the tunnel with one angled from west to east and three east to west.

We are recommending three deep angled oriented core borings to investigate the probability of fault and associated shear zones near the North Dallas Tollway, particularly near the central portion of the currently proposed tunnel alignment. Two additional vertical deep borings in the tunnel area to the depth of the Eagle Ford Shale also are recommended. Offsets or deviation in the depth of the contact between the Austin Chalk and the Eagle Ford Shale will be used to assist in the determination of faulting along the tunnel alignment.

A summary of the recommended additional borings and approximate locations are provided in the following table.
Additional borings that should be considered that are not shown on the table include borings at or near the vicinity of planned cross passages between the westbound and eastbound HOV tunnels. We understand these cross passages are currently spaced at 1200 ft. centers along the alignments. If the cross passage locations can be fixed, the locations of some of the remaining borings might be adjusted to satisfy this recommendation. Additional non-tunnel borings for particular bridges, retaining walls, ramps, utilities, and other structures where specific design geotechnical data is required should be considered. Detailed recommendations can be made once these are identified.

<table>
<thead>
<tr>
<th>Designation</th>
<th>Boring Type</th>
<th>Approximate Location</th>
<th>Orientation</th>
<th>Vertical Depth (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Angle</td>
<td>STA. 216 (Portal)</td>
<td>S to N</td>
<td>50</td>
</tr>
<tr>
<td>B</td>
<td>Angle</td>
<td>STA 216 (Portal)</td>
<td>E to W</td>
<td>50</td>
</tr>
<tr>
<td>C</td>
<td>Angle</td>
<td>STA. 225 (Portal)</td>
<td>N to S</td>
<td>80</td>
</tr>
<tr>
<td>D</td>
<td>Angle</td>
<td>STA 225 (Portal)</td>
<td>W to E</td>
<td>80</td>
</tr>
<tr>
<td>E</td>
<td>Angle</td>
<td>STA. 261 (DNT)</td>
<td>NW to SE</td>
<td>140</td>
</tr>
<tr>
<td>F</td>
<td>Vertical</td>
<td>STA. 264 (DNT)</td>
<td>N/A</td>
<td>180</td>
</tr>
<tr>
<td>G</td>
<td>Angle</td>
<td>STA 267 (DNT)</td>
<td>E to W</td>
<td>140</td>
</tr>
<tr>
<td>H</td>
<td>Angle</td>
<td>STA 272 (DNT)</td>
<td>E to W</td>
<td>140</td>
</tr>
<tr>
<td>I</td>
<td>Angle</td>
<td>STA 306 (Portal)</td>
<td>E to W</td>
<td>110</td>
</tr>
<tr>
<td>J</td>
<td>Angle</td>
<td>STA. 306 (Portal)</td>
<td>S to N</td>
<td>110</td>
</tr>
<tr>
<td>K</td>
<td>Vertical</td>
<td>STA 310 (Portal)</td>
<td>N/A</td>
<td>100</td>
</tr>
<tr>
<td>L</td>
<td>Vertical</td>
<td>STA. 315</td>
<td>N/A</td>
<td>220</td>
</tr>
<tr>
<td>M</td>
<td>Angle</td>
<td>STA 321 (Portal)</td>
<td>N to S</td>
<td>55</td>
</tr>
<tr>
<td>N</td>
<td>Angle</td>
<td>STA 321 (Portal)</td>
<td>E to W</td>
<td>55</td>
</tr>
<tr>
<td>O</td>
<td>Vertical</td>
<td>STA 321 (Portal)</td>
<td>N/A</td>
<td>55</td>
</tr>
</tbody>
</table>

9.3 Field Studies and In Situ Tests

We recommend an additional 20 hydraulic conductivity tests (packer tests), particularly near the potentially faulted areas and at the portals, be performed. Along with these tests, we recommend an additional 12 piezometers be installed in the vicinity of faults, adjacent to the proposed tunnel portals, and where additional groundwater information is believed necessary. Likewise, there should be a continuation of monitoring of the...
piezometers on a regular basis. For the existing piezometers, one per month should suffice.

A summary of the recommended piezometer locations is provided in the following table. The approximate locations are shown on the attached plan of borings.

Table 9-2: Recommended Piezometer Locations

<table>
<thead>
<tr>
<th>Location</th>
<th>Type</th>
<th>Location</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>BW3R</td>
<td>SPP</td>
<td>BW10L</td>
<td>SPP</td>
</tr>
<tr>
<td>BW16L</td>
<td>VWP</td>
<td>BW25R</td>
<td>VWP</td>
</tr>
<tr>
<td>T7R</td>
<td>SPP &amp; VWP</td>
<td>T10L</td>
<td>SPP &amp; VWP</td>
</tr>
<tr>
<td>F (see above)</td>
<td>SPP &amp; VWP</td>
<td>T23L</td>
<td>SPP &amp; VWP</td>
</tr>
<tr>
<td>T30R</td>
<td>SPP</td>
<td>T31L</td>
<td>VWP</td>
</tr>
</tbody>
</table>

SPP – Stand Pipe Piezometer
VWP – Vibrating Wire Piezometer

Other field studies should include:

a) A study to collect available information regarding character, frequency, and orientation of geologic structure such as joints, faults, and shears.

b) Geologic mapping of existing exposures of Austin Chalk via outcrops, excavations, or quarries.

c) Structural geology found at previous tunnel projects such as DART, SSC, and Addison Airport.

d) Examination of aerial photos and topographic surveys that predate IH-635 construction that may reveal presence of geologic structure based lineaments.

e) Review other standard sources including published and non-published papers regarding the local geologic structure.

9.4 Laboratory Testing

An appropriate number of the test types performed in Phase I should be completed on selected samples collected in Phase II. In addition to these, a series of special (non-standard) tests should be conducted to address specific issues identified in Phase I. The tests that should be considered include
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a) Cherchar and punch penetration tests on selected samples of limestone.

b) Petrographic and x-ray diffraction examination of various soil and rock types (i.e. slightly, moderately, and very argillaceous limestone, calcareous shale, bentonite, etc.) to define mineralogical composition, facilitate correlation with visual core descriptions, and further enhance ability to define ground behavior.

c) Controlled moisture-humidity tests of samples from the argillaceous and bentonitic zones to improve the understanding of slaking behavior under expected levels and cycles of humidity changes anticipated in the tunnel environment.

d) Strength and swelling properties of the bentonitic marker beds and similar material.

e) Corrosion testing on soil and groundwater samples

9.5 Other Related Studies

Gather structure foundation data, underground utility/pipeline data, and other available information regarding underground structures and support arrangements associated with existing and planned IH-635 improvements within 200 ft. of the tunnel alignments (including bridges, buildings, and retaining walls) to assess potential third part impacts.

Conduct additional environmental screening studies along the alignments to assess the risk of encountering potential subsurface pollution and contamination.

REFERENCES


11. TxDOT, Plan of Borings


Borings drilled for Phase 1
"Deep" boreng Phase 1
Borings to be drilled for Phase 2
(SPP) = Standpipe Piezometer
(VWP) = Vibrating Wire Piezometer

SCALE: 1" = 400' horizontal
1" = 40' vertical

FIGURE 2-1
- Borings drilled for Phase 1
- "Deep" borings Phase 1
- Borings to be drilled for Phase 2

(SPP) = Standpipe Piezometer
(VWP) = Vibrating Wire Piezometer

SCALE: 1" = 400' horizontal
1" = 40' vertical

PLAN OF BORINGS
LBJ 4-West
IH 635, US 75 to IH 35E
Dallas, Texas

FIGURE 2-1
Bonings drilled for Phase 1
"Deep" boning Phase 1
Bonings to be drilled for Phase 2
(SPP) = Standpipe Piezometer
(VWP) = Vibrating Wire Piezometer

Scale 1" = 400'
9 Borings drilled for Phase 1
9 "Deep" boring Phase 1
9 Borings to be drilled for Phase 2
(SPP) = Standpipe Piezometer
(VWP) = Vibrating Wire Piezometer

Scale 1" = 400'