Geotechnical Engineering Report

Austin F.C. Stadium at McKalla Place
Austin, Texas
February 14, 2019
Terracon Project No. 96185366

Prepared for:
Precourt Sport Ventures, LLC
Denver, CO

Prepared by:
Terracon Consultants, Inc.
Austin, Texas

February 14, 2019

Precourt Sport Ventures, LLC
c/o CAA ICON
5075 S. Syracuse Street, Suite 700
Denver, CO 80237

Attn: Mr. Dan Vaillant

Re: Geotechnical Engineering Report
Austin F.C. Stadium at McKalla Place
10715 Burnet Road
Austin, Texas
Terracon Project No. 96185366

Dear Mr. Vaillant:

We have completed the Geotechnical Engineering services for the above referenced project. This study was performed in general accordance with Terracon Proposal No. P96185366 dated November 16, 2018. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork, subgrade preparation, and the design and construction of foundations, pavements, and site improvements for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

STATE OF TEXAS
2/14/19
EDWARD ESCOBAR JAIMES
123368
LICENSED
PROFESSIONAL ENGINEER

Edward E. Jaimes, P.E.
Project Engineer

Bryan S. Moulin, P.E.
Senior Principal, Geotechnical Department Manager

REPORT TOPICS

Note: This report was originally delivered in a web-based format. **Orange Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the ***GeoReport*** logo will bring you back to this page. For more interactive features, please view your project online at client.terracon.com.

ATTACHMENTS

Note: Refer to each individual Attachment for a listing of contents.

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INTRODUCTION

This report presents the results of our subsurface exploration and geotechnical engineering services performed for the proposed Austin F.C. Stadium at McKalla Place project to be located at 10715 Burnet Road in Austin, Texas. The purpose of these services is to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil (and rock) conditions
- Groundwater conditions
- Site preparation and earthwork
- Lateral earth pressures
- Excavation considerations
- Foundation design and construction
- Floor slab design and construction
- Seismic site classification per IBC
- Dewatering considerations
- Pavement design and construction

The geotechnical engineering Scope of Services for this project included the advancement of twenty-two (22) test borings designated B-1 through B-22 to depths ranging from approximately 6 to 75 feet below existing site grades.

Maps showing the site and boring locations are shown in the **Site Location** and **Exploration Plan** sections, respectively. The results of the laboratory testing performed on soil samples obtained from the site during the field exploration are included on the boring logs and as separate graphs in the **Exploration Results** section.

SITE CONDITIONS

The following description of site conditions is derived from our site visit in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Parcel Information	The project is located on an approximately 24.14-acre tract of land located at 10715 Burnet Road in Austin, Texas. See **Site Location**

Item	Description
Existing Improvements	None, however the site was previously in use as a chemical facility. Based on information provided to us, the site was stripped of all soils, the material was screened, and existing soils and new fill soils were placed and compacted to current grades.
Current Ground Cover	Soils, grass, weeds, and scattered trees throughout the site along with spoil piles. Steel beams and miscellaneous construction materials and construction debris are located throughout the site.
Existing Topography	Based on a topographic survey provided to us, existing elevations range from a low elevation of about 748 feet along the eastern perimeter of the site to a high elevation of about 783 feet near the western entrance from Burnet Road. Within the stadium area, elevations range from a low elevation of about 755 feet to a high elevation of about 768 feet.
Geology	Based on our borings, the site consists of low to high plasticity fill soils overlying the Austin Chalk limestone of Upper Cretaceous Age. The Austin Chalk limestone is generally comprised of tan to gray chalky limestone and marls.

PROJECT DESCRIPTION

Our initial understanding of the project was provided in our proposal and was discussed during project planning. A period of collaboration has transpired since the project was initiated, and our current understanding of the project conditions is as follows:

Item	Description
Information Provided	Request for Proposal packet provided to us by CAA ICON on October 22, 2018. Since then additional information such as existing topographic information, schematic cross-sections of the stadium, and anticipated FFEs for the field and the upper concourse area has been provided to us by CAA ICON and the design team.
Proposed Structures	The project includes the construction of a Major League Soccer (MLS) stadium for the proposed Austin FC soccer team. In addition, based on the latest information provided to us, we understand that a performance space, parking/driveway areas, a detention pond, rain gardens, and water storage silos are part of the planned development.
Building Construction	■ Cast-in-place concrete for the below-grade structure. ■ Slab on grade for the seating bowl. ■ Elevated seating bowl will be constructed of precast stadia units and steel structure. ■ Structural steel long spans for the roof.

Item	Description
Finished Floor Elevation	Finished floor of the field level is anticipated to be at about 747 feet and the main concourse is anticipated to be at about 770 feet for the north, east and west portions of the concourse. The southern portion of the concourse is anticipated to be at about 747 feet.
Maximum Loads	■ Columns: 3,000 kips maximum ■ Walls: 6 to 8 kips per linear foot (klf) maximum ■ Slabs: 150 to 200 pounds per square foot (psf) maximum
Grading/Slopes	Up to 20 feet of cut and 15 feet of fill is anticipated to develop final grade within the building footprint. Cuts and fills of up to 3 feet are anticipated within non-building areas. Assumed to be no steeper than 3H:1V (Horizontal to Vertical)
Below-Grade Structures	The field is anticipated to be between 9 to 20 feet below existing grades. Below-grade walls up to 23 feet tall are anticipated along the southern portion of the concourse.
Free-Standing Retaining Walls	Walls up to 6 to 8 feet tall are anticipated.
Below-Grade Areas	A detention pond up to 10 feet below existing grades is anticipated.
Pavements	We assume both rigid (concrete) and flexible (asphalt) pavement sections will be considered. In addition, we understand that an unpaved parking lot is being considered in the northern portion of the site. This unpaved parking lot will be used on non-matchdays for community events such as farmers markets.

GEOTECHNICAL CHARACTERIZATION

We have developed a general characterization of the subsurface conditions based upon our review of the subsurface exploration, laboratory data, geologic setting and our understanding of the project. This characterization, termed GeoModel, forms the basis of our geotechnical calculations and evaluation of site preparation and foundation options. Conditions encountered at each exploration point are indicated on the individual logs. The individual logs can be found in the **Exploration Results** section and the GeoModel can be found in the **Figures** section of this report.

We also collected photographs of the rock cores while reviewing the samples in our laboratory. Photos are provided in the **Exploration Results** section of this report.

Groundwater

The boreholes were observed while drilling and after completion for the presence and level of groundwater. In addition, four piezometers were installed to depths of 25 feet each at boring locations B-1, B-3, B-9, and B-11. The water levels observed in the boreholes and piezometers can be found on the boring logs in **Exploration Results** and are summarized below.

Boring Number	Date of Reading	Approximate Depth to Groundwater (feet) [1]	Approximate Groundwater Elevation (feet) [1]
B-1	1/7/2019	12.5	755
	1/14/2019	10.2	758
	1/16/2019	9.3	758
	1/16/2019	23.8 (after bailing) [2]	N/A
	1/16/2019	22 (106 minutes after bailing) [2]	N/A
	1/28/2019	16.6	751
	1/28/2019	21.3 (after bailing) [2]	N/A
	1/28/2019	20.3 (59 minutes after bailing) [2]	N/A
	2/6/2019	15.9	752
B-3	1/7/2019	4.5	752
	1/17/2019	2.7	753
	1/16/2019	2.3	754
	1/16/2019	17.3 (after bailing) [2]	N/A
	1/16/2019	12.4 (21 minutes after bailing) [2]	N/A
	1/28/2019	4.9	751
	1/28/2019	14.6 (after bailing) [2]	N/A
	1/28/2019	8.5 (23 minutes after bailing) [2]	N/A
	2/6/2019	5	751
B-8	12/19/2018	8	747
B-9	1/7/2019	14.7	744
	1/14/2019	9.7	749
	1/16/2019	8.5	750
	1/16/2019	23.6 (after bailing) [2]	N/A

1. Below ground surface
2. Groundwater was initially recorded. The groundwater was then removed manually using a water bailer and the groundwater level was recorded. A third reading was taken again some time after bailing occurred to record the groundwater level surge over a period of time.

Boring Number	Date of Reading	Approximate Depth to Groundwater (feet) [1]	Approximate Groundwater Elevation (feet) [1]
B-9	1/16/2019	21.4 (78 minutes after bailing) [2]	N/A
	1/28/2019	11.5	747
	1/28/2019	21.1 (after bailing) [2]	N/A
	1/28/2019	20.2 (29 minutes after bailing) [2]	N/A
	2/6/2019	11.8	746
B-11	12/7/2019	6	751
	1/7/2019	4.2	752
	1/14/2019	4.7	752
	1/16/2019	4.8	752
	1/16/2019	4.8 (after bailing) [2]	752
	1/28/2019	5.5	751
	1/28/2019	5.5 (after bailing) [2]	N/A
	2/6/2019	5.8	751
B-12	12/17/2019	8	745

1. Below ground surface
2. Groundwater was initially recorded. The groundwater was then removed manually using a water bailer and the groundwater level was recorded. A third reading was taken again some time after bailing occurred to record the groundwater level surge over a period of time.

Groundwater seepage should be expected at this site, particularly in the form of seepage traveling along pervious seams/fissures in the soil, along the soil/limestone interface and or in fissures/fractures in the limestone. Please contact us if additional groundwater level checks are desired after the completion of our geotechnical report. Groundwater conditions should be evaluated immediately prior to construction.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

GEOTECHNICAL OVERVIEW

The near surface, stiff to hard medium plasticity lean clay and high plasticity fat clay could become problematic with typical earthwork and construction traffic, especially after precipitation events. Effective drainage should be completed early in the construction sequence and maintained after construction to avoid potential issues. Additional site preparation recommendations including subgrade improvement and fill placement are provided in the **Earthwork** section.

In some areas, the subgrade soils for the floor slabs consist of high plasticity fat clay, therefore extensive subgrade preparation is necessary in order to reduce post-construction movements to about 1-inch. The **Floor Slabs** section addresses subgrade preparation options for different areas.

Existing fill materials were encountered at this site and have been documented by the City of Austin. These fill materials were placed by others in the early/mid-2000s under the supervision of City of Austin. Even with the previous construction procedures monitored by the City of Austin, there is an inherent risk for the owner that compressible fill or unsuitable material within or buried by the fill will not be discovered. This risk of unforeseen conditions cannot be eliminated without completely removing the existing fill but can be reduced by following the recommendations contained in this report. To take advantage of the cost benefit of not removing the entire amount of undocumented fill, the owner must be willing to accept the risk associated with building over the existing fills. The improvements to this site can be constructed provided our recommendations provided in this report are followed.

This report provides recommendations to help mitigate the effects of soil shrinkage and expansion. However, even if these procedures are followed, some movement and (at least minor) cracking in the structures should be anticipated. The severity of cracking and other damage such as uneven floor slabs will probably increase if modification of the site results in excessive wetting or drying of the expansive soils. Eliminating the risk of movement and distress may not be feasible, but it may be possible to further reduce the risk of movement if significantly more expensive measures are used during construction.

The **Shallow Foundations** section addresses the support of the structure (and ancillary structures) on a spread/strip footing foundation bearing into on-site soils or select fill for ancillary structures and Stratum 4 limestone for the stadium structure. The **Deep Foundations** section addresses support of the stadium structure on drilled piers bearing into Stratum 4 limestone. The **Floor Slabs** section addresses slab support of the structures.

Based on elevations, planned grading, and the size/location of the site, we anticipate that the below-grade levels will be constructed in open sloped cuts without the need for temporary retention. The **Below-Grade Structures** section addresses drainage of the permanent wall system.

Lateral earth pressures are also provided for on-site retaining walls and below-grade walls. The **Lateral Earth Pressures** section address the design of retaining walls.

Asphaltic concrete and portland cement concrete pavement systems are recommended for this site. The **Pavements** section addresses the design of pavement systems. If any of the pavements will be City of Austin roadways, please let us know to re-evaluate the sections.

Slope inclinations and construction recommendations are provided for cut and fill slopes (embankments). The **Slope Stability** section addresses cut and fill slopes.

The **General Comments** section provides an understanding of the report limitations.

EARTHWORK

Earthwork is anticipated to include clearing and grubbing, excavations, and fill placement. The following sections provide recommendations for use in the preparation of specifications for the work. Recommendations include critical quality criteria, as necessary, to render the site in the state considered in our geotechnical engineering evaluation for foundations, floor slabs, and pavements.

Site Preparation

Construction areas should be stripped of all vegetation, concrete, asphalt, loose soils, top soils, construction debris, and other unsuitable material currently present at the site. Roots of trees to be removed within construction areas, should be grubbed to full depths, including the dry soil around the roots. All remnants of any existing foundations should be completely excavated and removed to at least 2 feet below finished grades. If any unusual items are unearthed during or after demolition, please contact us for further evaluation. Any utilities to be abandoned should be completely removed from all proposed construction areas. If this is not feasible, then the abandoned utility piping should be filled with flowable fill (COA Item No. 402S or TxDOT Item No. 401) and plugged such that it does not become a conduit for water flow. Site stripping and excavation operations in cut areas will encounter the Stratum 4 limestone which should either be properly broken down or removed from the site. We recommend that Terracon be retained to assist in evaluating exposed subgrades during earthwork so that unsuitable materials, if any, are removed at the time of construction.

Proof-Rolling

Once initial subgrade elevations have been achieved (i.e., after cuts but prior to fills), the exposed subgrade in all construction areas (except landscaping) should be carefully and thoroughly proof-rolled with a 20-ton pneumatic roller, fully-loaded dump truck, or similar equipment to detect weak zones in the subgrade. Proof-rolling is not necessary in intact Stratum 4 limestone subgrade areas. Weak areas detected during proof-rolling, zones containing debris or organics, and voids resulting from removal of tree roots, existing foundation elements, utilities, fill, boulders, etc. should be removed and replaced with soils exhibiting similar classification, moisture content, and density as the adjacent in-situ soils (or flowable fill). Proper site drainage should be maintained during construction so that ponding of surface runoff does not occur and cause construction delays and/or exhibit site access.

Moisture-Conditioned Subgrade

After proof-rolling, and just prior to placement of fill, the exposed soil subgrade in all construction areas (except landscaping) should be evaluated for moisture and density through field density testing. If the moisture and/or density requirements do not meet the moisture and density requirements below, the subgrade should be scarified to a minimum depth of 6 inches, moisture conditioned and compacted as per the fill compaction requirements. Moisture conditioning is not required in intact Stratum 4 limestone subgrade areas.

Existing Fill

Sixteen of the 22 borings exhibited fill to depths ranging from about 0.5 to 15 feet. We have no records to indicate if the fill was placed in a controlled manner or, the degree of control if it was placed in a controlled manner. However, based on information provided to us, we understand that the City of Austin oversaw the remediation that occurred at this site. The remediation efforts included excavating a majority of the soils down to limestone bedrock, screening the excavated soils (for contaminants due to this site being an old chemical plant), and then recompacting up to existing grades. Based on this information that has been provided to us, improvements to the site can be constructed on these existing fill soils, provided the recommendations provided in our report our followed.

As the Client is aware, this site was remediated in the early/mid 2000s to clean up contaminants from a prior chemical facility. Based on an email from Mr. Greg Kiloh of the City of Austin, the vast majority of the site was excavated to bedrock, the soil sifted, placed back and compacted. Some areas of the site, primarily where building pads had already been constructed for the Austin Water Service Center, were not excavated during the remediation process. Given the fact that the vast majority of the site was remediated/cleaned, and that the chemical plant's contaminants were isolated to a very small area, it was the opinion of the remediation experts at the City of Austin and TCEQ that the unexcavated areas are likely to also be free of contamination. However, given the explosive nature of some of the contaminants previously found on the chemical plant site, the City of Austin recommended that excavation procedures similar to those outlined in Section 3.1 of the Excavation Work Plan (prepared by Shaw Environmental, Inc., dated November 2003) be followed if and when those areas are excavated for the stadium/site construction. Based on the above, Terracon recommends that the General Contractor and Earthwork Contractor (and any others performing excavations during construction) be provided with a copy of the above-mentioned Excavation Work Plan for their review during their pre-task planning phases.

Excavations

Excavation operations at this site will penetrate through the on-site soils and into the Stratum 4 limestone. While the overlying soils should be relatively easy to excavate in comparison to the underlying limestone, there is a probability of encountering limestone cobbles, boulders, seams, and layers within these soils. Our past experience with the Stratum 4 limestone, along with the data obtained during our field and laboratory programs (compressive strength ranging from 360 psi to 4,600 psi, average of 2,200 psi), indicates that the Stratum 4 limestone will require sawcutting, jackhammering, hoe-ramming, milling, or similar techniques to excavate.

Please note that Stratum 4 limestone was encountered at varying depths ranging from at the ground surface to 15 feet below existing grades across the site, thus the weathering profile of limestone can be unpredictable. The Contractor should be prepared to encounter and properly excavate near-surface limestone anywhere on this site.

Our comments on excavation are based on our experience with the rock formation. Rock excavation depends on not only the rock hardness, weathering and fracture frequency, but also the contractor's equipment, capabilities, and experience. Therefore, it should be the contractor's responsibility to determine the most effective methods for excavation. The above comments are intended for information purposes for the design team only and may be used to review the contractor's proposed excavation methods.

Temporary Groundwater Control

As encountered during our drilling operations, groundwater seepage is expected to be encountered during construction, especially after periods of wet weather. Temporary groundwater control during construction would typically consist of perimeter gravel-packed drains sloping toward common sump areas for groundwater collection and removal. Placement of drain laterals within the excavation could be required to remediate isolated water pockets.

The volume of groundwater seeping/flowing into the excavation will vary based on rainfall patterns before and during construction, but we expect that there will be a need for temporary groundwater collecting and pumping. This could be accomplished by sloping the bottom of the excavation continually throughout construction such that water entering the excavation would flow towards one or more sump pits deeper than the excavation and then pumping the water out on a daily basis.

Fill Material Types

Fill required to achieve design grade should be classified as select/structural fill and general fill. Select/structural fill is material used below, or within 5 feet of structures. General fill is material used to achieve grade in paving, non-reinforced earthen slopes, landscape, or other general areas (non-structural areas). Earthen materials used for select fill and general fill should meet the following material property requirements:

Fill Type [1]	USCS Classification	Acceptable Specifications
Imported Select/Structural Fill [2,3,4]	CL, SC, and/or GC	■ TxDOT Item 247, Type A, Grade 3, OR ■ Percent Retained on No. 4 Sieve ≤ 40 percent with 5≤PI≤20 and rocks ≤ 2 inches in maximum dimensions, OR ■ Crushed concrete (TxDOT Item 247, Type D, Grade 3 or better)
Paving Fill and General Fill [5]	CH, CL, SC and/or GC	PI ≤ 35; Rocks ≤ 4 inches in maximum dimension

1. Structural and general fill should consist of approved materials free of organic matter and debris. A sample of each material type should be submitted to the Geotechnical Engineer for evaluation prior to use on this site.
2. As an alternative to the Acceptable Specifications above, a low-plasticity granular material which does not meet these specifications may be used only if approved by Terracon.
3. Based on the laboratory testing performed during this exploration, much of the excavated on-site soils does appear to be suitable for re-use as select fill. However, there are some on-site soils that are not suitable due to the higher plasticity, such as those present to significant depths at borings B-5, B-8, B-11, B-12, B-14, and B-19. These higher plasticity (PI>20) must not be considered as select fill. In addition, it is not permissible to blend the higher plasticity soils with other materials in attempts to produce a select fill. The Earthwork Contractor will need to segregate and stockpile different soils for further evaluations during construction.
4. The excavated Stratum 4 limestone material will be acceptable for re-use as select fill provided that it is processed such that it meets one of the Acceptable Specifications above for Imported Select/Structural Fill. The maximum rock size may be increased to 4 inches below the upper 5 feet of the building pad. Please note that removal of higher plasticity soils and layers (typically dark brown to brown in color) may be necessary to maintain plasticity indices of the material within acceptable range. In some situations, the difference between more highly plastic clay, lower plasticity silty soils, and appropriate material may not be readily distinguishable without the performance of appropriate lab testing. After initial processing of the fill material, samples should be submitted to Terracon for evaluation of proper gradation, plasticity index, and maximum rock size prior to re-use as select fill. We recommend that periodic testing be performed throughout the material excavation phase to check for conformance with the select fill requirements given above.

   Due to the potentially significant depth of fill, the varying levels of compaction effort, and the assumption that on-site rocks will be used, we recommend that full time testing be performed throughout the material excavations phase to check for conformance with the select fill requirements given above. Due to the varying levels of compaction effort, it will be imperative for the Earthwork Contractor to establish visual contractors (i.e. stakes) to delineate the planar limits, as well as elevation control, of earthwork for each type of improvement at this site. This facet of site control should be maintained throughout the earthwork operations. We suggest that this item be included in bidding.

To attain maximum rock sizes of 2 inches in the fill (4 inches below the upper 5 feet of the building pad), the Earthwork Contractor will need to utilize sufficient equipment that is capable of processing the on-site limestone and any imported rocky fill that is hauled to this site. It has been our experience that proper processing of excavated limestone often involves such processes as breaking down of larger rock with equipment, screening, removal of more highly plastic clay layers, etc. The Contractor's proposed methods of processing these materials should be reviewed prior to initiation of construction to check that these methods will produce an acceptable select fill material. Attempting to break down rock within the building pad areas while attempting to place and compact the same fill is not acceptable as there is not a definitive way of controlling rock sizes with this approach. Any rock fill to be used within the building pad should be processed away from the building pad and the placed in the building pad area. In no instance should the rock fill be processed (i.e., breaking down of rock) within the building pad area.

5. Excavated on-site soils, if free of organics, debris, and rocks larger than 4 inches may be considered for re-use as fill in pavement, landscape, pond, or other general areas. Please note that some of the on-site soils exhibit high to very high shrink/swell potential. For economic reasons, expansive soils are often used in pavement and/or flatwork areas. The owner should be aware that the risk exists for future movements of the subgrade soils which may result in movement and/or cracking of pavement and/or flatwork. If paving fill is imported, the PI should not exceed 35.

Fill Compaction Requirements

Recommended compaction and moisture content criteria for engineered fill materials are as follows.

<table>
<tr><th colspan="2">Material Type</th><th>Minimum Compaction Requirement (%)[1]</th><th>Moisture Content Range (%)</th><th>Maximum Loose Lift Thickness (in)[2]</th></tr>
<tr><td colspan="2">Select/Structural Fill</td><td>95[3]</td><td>-3 to +3</td><td>8 inches</td></tr>
<tr><td rowspan="2">Moisture Conditioned Building Subgrade</td><td>PI ≤ 25</td><td>95</td><td>-3 to +3</td><td rowspan="2">8 inches</td></tr>
<tr><td>PI > 25</td><td>95</td><td>Optimum to +4</td></tr>
<tr><td rowspan="2">Paving Fill, Paving Subgrade and General Fill</td><td>PI ≤ 25</td><td>95</td><td>-3 to +3</td><td>8 inches</td></tr>
<tr><td>PI > 25</td><td>95</td><td>Optimum to +4</td><td>8 inches</td></tr>
<tr><td colspan="2">Crushed Limestone Base (beneath pavements)</td><td>100[4]</td><td>-3 to +3</td><td>8 inches</td></tr>
</table>

1. Per the Standard Proctor Test (ASTM D 698).
2. Fill lift thickness must be reduced (typically 4 to 6 inches) if light compaction equipment is used, as is customary within a few feet of retaining walls and utility trenches.
3. **For fills greater than 5 feet in depth, the compaction should be increased to at least 100 percent of the ASTM D 698 maximum dry unit weight.**
4. Per TEX-113-E (or 95% of Modified Proctor, ASTM D1557).

Grading and Drainage

The performance of the proposed structures will not only be dependent upon the quality of construction, but also upon the stability of the moisture content of the near surface soils. Therefore, we highly recommend that site drainage be developed so that ponding of surface runoff near the structures does not occur. Accumulation of water near the structures may cause significant moisture variations in soils adjacent to the structures, thus increasing the potential for structural distress.

Effective drainage away from the structures must be provided during construction and maintained through the life of the proposed project. Infiltration of water into excavations should be prevented during construction. It is important that foundation soils are not allowed to become wetted. All grades must provide effective drainage away from the structures during and after construction. Exposed (unpaved) ground should be sloped at a minimum of 5 percent away from the structures for at least 10 feet beyond the perimeter of the structures. Locally, flatter grades may be necessary to transition ADA access requirement for flatwork.

Roof runoff and surface drainage should be collected and discharged away from the structures to prevent wetting of the foundation soils. Roof gutters should be installed and connected to downspouts and pipes directing roof runoff at least 10 feet away from the structures or discharged on to positively sloped pavements.

Irrigation sprinkler mains and spray heads should preferably be located at least 5 feet away from the structures such that they cannot become a potential source of water directly adjacent to the structures. In addition, the owner and/or builder should be made aware that placing large bushes and trees adjacent to the structures may cause significant moisture variations in the soils underlying the structures. In general, tree roots can adversely influence the subsurface soil moisture content to a distance of 1 to 1½ times the mature height of the tree and beyond the tree canopy. Watering of vegetation should be performed in a timely and controlled manner and prolonged watering should be avoided. Landscaped irrigation adjacent to the foundation units should be minimized or eliminated. Special care should be taken such that underground utilities do not develop leaks with time.

After building construction and landscaping, final grades should be verified to document effective drainage has been achieved. Grades around the structures should also be periodically inspected and adjusted as necessary as part of the structure's maintenance program. Where paving or flatwork abuts the structures, a maintenance program should be established to effectively seal and maintain joints and prevent surface water infiltration. Water permitted to pond next to the structures can result in greater soil movements than those discussed in this report. Estimated movements described in this report are based on effective drainage for the life of the structures and cannot be relied upon if effective drainage is not maintained.

Earthwork Construction Considerations

Based on our test borings, highly expansive soils that exhibit a potential for volumetric change during moisture variations are present throughout several locations at this site. However, the highly expansive soils could be difficult to discern from the low to moderately expansive soils due to similar color, texture, and gradation. These subgrade soils at the surface may experience expansion and contraction due to changes in moisture content. At existing grades, the soils at this site could exhibit a Potential Vertical Rise (PVR) of up to about 2.5 inches, as estimated by the TxDOT Method TEX-124-E, if present in a dry condition.

Excavations, for the proposed structures and utilities, are anticipated to be accomplished with conventional construction equipment utilized in the Austin area and for the Austin Group limestone. Upon completion of filling and grading, care should be taken to maintain the subgrade water content prior to construction of floor slabs. Construction traffic over the completed subgrades should be avoided as much as possible. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. Water collecting over, or adjacent to, construction areas should be removed. If the subgrade desiccates, saturates, or is disturbed, the affected material should be removed, or the materials should be scarified, moisture conditioned, and recompacted, prior to floor slab construction.

Groundwater will affect over-excavation efforts. A temporary dewatering system consisting of sumps with pumps will likely be necessary to achieve the recommended depth of over-excavation. Sump pits should preferably be excavated just outside the pad limits.

As a minimum, excavations should be performed in accordance with OSHA 29 CFR, Part 1926, Subpart P, “Excavations” and its appendices, and in accordance with any applicable local, and/or state regulations.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean Terracon is assuming responsibility for construction site safety, or the contractor's activities; such responsibility shall neither be implied nor inferred.

Construction Observation and Testing

The earthwork efforts should be documented under the direction of the Geotechnical Engineer. This should include documentation of adequate removal of vegetation and top soil, proof-rolling and mitigation of areas delineated by the proof-roll to require mitigation and density/moisture testing of subgrade and fills. In the event that unanticipated conditions are encountered, the Geotechnical Engineer should be contacted to evaluate the conditions.

Each lift of compacted fill should be tested, evaluated, and reworked as necessary until approved by the Geotechnical Engineer prior to placement of additional lifts. Fill should be tested for density and water content at a frequency of at least one test for every 5,000 square feet per lift of compacted fill in the building areas (with a minimum of 3 tests per lift) and 10,000 square feet per lift in pavement areas. A minimum of one density and water content test should be conducted for every 100 linear feet of compacted utility trench backfill in paving areas.

In addition to the documentation of the essential parameters necessary for construction, the continuation of the Geotechnical Engineer into the construction phase of the project provides the continuity to maintain the Geotechnical Engineer's evaluation of subsurface conditions, including assessing variations and associated design changes.

Possible French Drains for Wet Southern Portion

The southern portion of the site (proposed for future parking lot and driveway) contained wet/soft subgrade during all of our site visits. As such, we recommend that interceptor drains should be strongly considered along the parking lot perimeter and/or access driveway. Installation of perimeter interceptor trenches/drains (French drains) should be installed along the uphill sides of the parking lot/access drive to intercept and remove groundwater before it has an opportunity to infiltrate into the adjacent improvements. In areas where groundwater seepage is observed before or during construction, (such as in this southern portion), installation of such drains is highly recommended. If no such seepage is observed, drain installations could be considered non-mandatory. However, in situations where groundwater seepage is observed after construction is completed, such drain installations will likely be costlier as well as more intrusive to the constructed facilities.

For drains adjacent to pavement sections, we recommend that drains extend at least 30 inches below the adjacent pavement surface. The drain system should be designed to gravity flow (with a minimum slope of 1%) and outlet downhill and away from the adjacent improvements. The drains should consist of a clean, washed, gravel section (at least 18 inches wide) meeting the gradation requirements of ASTM C 33, Grade 57 (or Pipe Bedding Stone per COA Item 510), continuously wrapped in filter fabric (per COA Item 620S). Perforated collector pipes (Type 31 PVC pipe per COA Item 551) with a minimum diameter of 4 inches should be provided for all sections of the interceptor drains. The granular fill should extend to within 12 inches of final grade, where it should be capped with compacted cohesive fill to reduce infiltration of surface water into the drain system. Terracon would be pleased to review the actual location, depth, and cross-sections of the drains with the other Design Team members prior to construction.

SHALLOW FOUNDATIONS

We understand that shallow foundations may be considered for portions of the stadium structure as well as for other small structures throughout the site (statues, stairs, entryways, walls, rainwater tanks/cisterns, etc.). If the site has been prepared in accordance with the requirements noted in **Earthwork** and **Floor Slabs**, the following design parameters are applicable for shallow foundations.

Design Parameters – Footings

Principal column and wall loads for the proposed structures may be supported on isolated (spread) and/or continuous (strip) footings. Ring footings could be used to support cylindrical rainwater tanks. For the stadium structure, we recommend that if footings are considered, all footings should bear into the Stratum 4 limestone. Design parameters for spread/strip footing foundations are provided below.

Footings for non-stadium structures should bear on compacted select fill, on-site soils or Stratum 4 limestone, but not a combination of soil and Stratum 4 limestone materials for each structure. If footings or grade beams are designed to bear on soils and the Stratum 4 limestone is encountered during site preparation, the Stratum 4 limestone should be over-excavated as necessary to provide at least 12 inches of select fill under all grade beams.

Footings for Non-Stadium Structures

Description	Design Parameters			
Bearing Stratum	On-site Soils		Select Fill	Stratum 4 Limestone
Minimum Embedment below Final Grade [1]	18 inches			-
Minimum Embedment into Bearing Stratum	-			6 inches
Minimum Footing Dimensions	Spread – 3 feet by 3 feet square Strip – 18 inches wide			
Allowable Bearing Pressure	1,000 psf	2,000 psf	3,000 psf	10,000 psf
Approximate Total Settlement [2]	≤1-inch	≤2.5 inches	≤1-inch	≤¾-inch
Estimated Differential Settlement [3]	Approximately ½ to ¾ of total settlement			
Nominal (unfactored) Passive Resistance [4]	300 psf per foot of depth		350 psf per foot of depth	750 psf per foot of depth
Coefficient of Sliding Friction [5]	0.3		0.35	0.6
Nominal (unfactored) Uplift Resistance [6]	Foundation Weight (150 pcf) & Soil Weight (120 pcf)			

1. Unsuitable or soft soils must be over-excavated and replaced per the recommendations presented in **Earthwork**.
2. To bear within select fill, on-site soils, or Stratum 4 limestone.
3. Whichever condition yields a larger bearing area.

4. Values provided are for maximum loads noted in **Project Description**.
5. The estimated post-construction settlement of the shallow footings is assuming proper construction practices are followed.
6. Differential settlements may result from variances in subsurface conditions, loading conditions and construction procedures. The settlement response of the footings will be more dependent upon the quality of construction than upon the response of the subgrade to the foundation loads.
7. Passive resistance should be neglected in the first 12 inches below finished grades. Care should be taken to avoid disturbance of the footing bearing area since loose material could increase settlement and decrease resistance to lateral loading. If the footing is formed during construction, the open space between the footings and the in-situ soils should be backfilled with concrete.
8. Lateral loads transmitted to the footings will be resisted by a combination of soil-concrete friction on the base of the footings and passive pressure on the side of the footings. We recommend that the allowable frictional resistance be limited to 500 psf in select fill/on-site soils and 1,000 psf in Stratum 4 limestone.
9. The nominal values should be reduced by an appropriate factor of safety to compute allowable values.

Footings for Stadium Structure

Description		Design Parameter
Bearing Stratum [1]		Stratum 4 Limestone
Minimum Embedment Below Final Grade [2]		12 inches into Stratum 4 Limestone
Minimum Footing Dimensions		Spread – 3 feet by 3 feet square Strip – 18 inches wide
Allowable Bearing Pressures [3,4]	**12 inches into Stratum 4 Limestone**	20,000 psf
	1.5-2 feet into Stratum 4 Limestone	40,000 psf
	2.5-4 feet into Stratum 4 Limestone	70,000 psf
	4+ feet into Stratum 4 Limestone	110,000 psf
Approximate Total Movement [5]		≤ ¾-inch
Estimated Differential Movement [6]		Approximately ½ to ¾ of total settlement
Nominal (unfactored) Passive Resistance [7,9]		1,000 psf per foot of depth into Stratum 4 Limestone
Coefficient of Sliding Resistance [8]		0.7 for Stratum 4 Limestone
Nominal (unfactored) Uplift Resistance [9]		Foundation Weight (150 pcf) & Soil Weight (120 pcf)

1. Soil layers within the Stratum 4 limestone must be over-excavated and replaced per the recommendations presented in **Earthwork**.

2. To bear within Stratum 4 limestone.
3. Whichever condition yields a larger bearing area.
4. Values provided are for maximum loads noted in **Project Description**.
5. The estimated post-construction settlement of the shallow footings is assuming proper construction practices are followed.
6. Differential settlements may result from variances in subsurface conditions, loading conditions and construction procedures. The settlement response of the footings will be more dependent upon the quality of construction than upon the response of the subgrade to the foundation loads.
7. Passive resistance should be neglected in the first 12 inches below finished grades. Care should be taken to avoid disturbance of the footing bearing area since loose material could increase settlement and decrease resistance to lateral loading. If the footing is formed during construction, the open space between the footings and the in-situ soils should be backfilled with concrete.
8. Lateral loads transmitted to the footings will be resisted by a combination of soil-concrete friction on the base of the footings and passive pressure on the side of the footings. We recommend that the allowable frictional resistance be limited to 2,000 psf in Stratum 4 limestone.
9. The nominal values should be reduced by an appropriate factor of safety to compute allowable values.

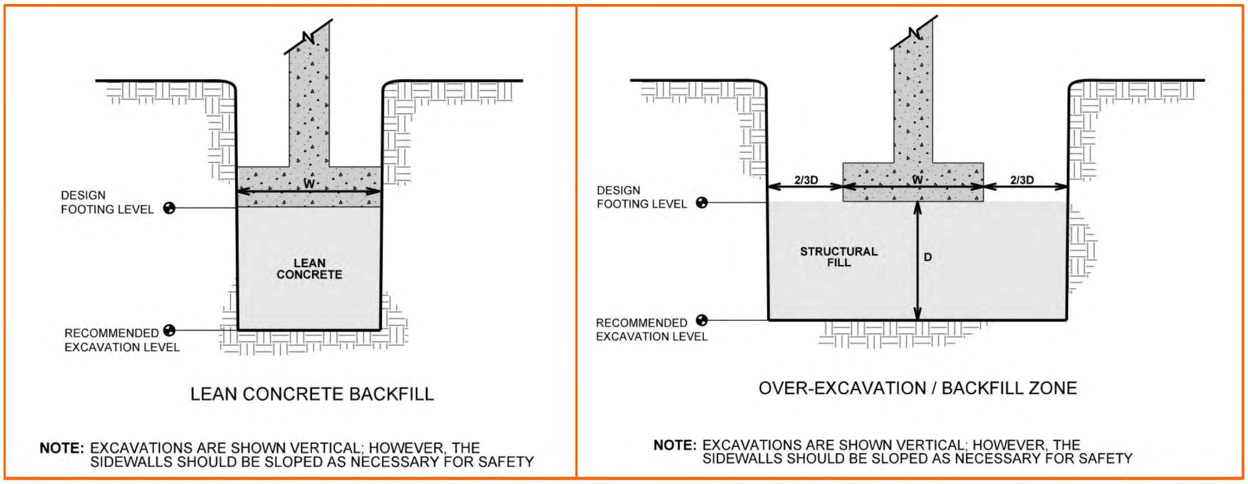
Construction of Structures with Different Foundation Systems

Differential settlement between the improvements on shallow foundations and the improvements on deep foundations is expected to approach the magnitude of the differential settlement of the improvements. Expansion joints should be provided between the various structures to accommodate differential movements between the two structures. Underground piping between the structures should be designed with flexible couplings and utility knockouts in foundation walls should be oversized, so minor deflections in alignment do not result in breakage or distress.

Foundation Construction Considerations

Footings should be neat excavated, if possible. If neat excavation is not possible, the foundation should be properly formed. If a toothed bucket is used, excavation with this bucket should be stopped approximately 6 inches above final grade of the footings and the footing excavation be completed with a smooth-mouthed bucket or by hand labor. In limestone subgrade areas, rock-trenching or saw-cutting equipment will be required. Debris in the bottom of the excavation should be removed prior to steel reinforcement placement. The foundation excavation should be sloped sufficiently to create internal sumps for runoff collection and removal. If surface runoff water or groundwater seepage in excess of ½-inch accumulates at the bottom of the foundation excavation, it should be collected, removed, and not allowed to adversely affect the quality of the bearing surface.

If unsuitable bearing soils are encountered at the base of the planned footing excavation (such as low strength or disturbed soils), the footing excavations should be deepened to expose suitable bearing materials. The footings could then bear directly on these soils at the lower level, on lean concrete backfill placed in the excavations, or on compacted structural fill backfilled in the excavations and compacted as in **Earthwork**. This is illustrated in the figure below.



Concrete should be placed as soon as possible after excavation to reduce bearing soil disturbance. Soils at bearing level that become disturbed or saturated should be removed prior to placing reinforcing steel and concrete. Adequate water control/dewatering system will aid in minimizing the need for over-excavation and backfill of any soils disturbed by prolonged exposure. It is important that the foundation subgrade not be disturbed by construction activities (e.g., setting forms and placing reinforcing steel). If disturbance occurs, we recommend that the disturbed soils be removed and that the foundation subgrade be protected with the placement of a lean concrete “mud mat”.

Foundation Construction Observation

The performance of the foundation system for the proposed structure will be highly dependent upon the quality of construction. Thus, we recommend that the foundation construction be monitored by Terracon to identify the proper bearing strata and depths and to help evaluate foundation construction. We would be pleased to develop a plan for foundation observation to be incorporated in the overall quality assurance program.

DEEP FOUNDATIONS

Drilled Pier Design Parameters

Bearing pressures of piers founded in rock are dependent upon the secondary structure of the rock, as well as the compressive strength. Although these secondary features are taken into account in our recommendations, a pier should not be terminated on a soft clayey layer, a void, or a severely weathered zone within the Stratum 4 limestone. While drilling, the driller and field technician should be continuously monitoring for these softer layers. At location where the design embedment results in the pier terminating on one of these secondary features, the pier should be extended deeper to bear at least one foot below the features into competent limestone. Side friction may be counted above and below (but not within) these secondary features.

Due to the subsurface conditions previously mentioned, along with planned cuts and fills, the total pier lengths will vary across the structure, therefore, appropriate base bid depths should be determined for the project. Due to the fact that many of the piers may extend deeper due to the presence of clay layers, the contract documents should include unit rates for additional drilled pier footage at various pier diameters. In addition, the construction budget should include overages due to the likelihood of additional costs associated with extending many of the drilled piers to greater depths.

Rock design parameters are provided below in the **Drilled Pier Design Summary** table for the design of straight-sided drilled pier foundations. The values presented for allowable side friction and end bearing include a factor of safety.

Drilled Pier Design Summary	
Description	**Drilled Pier Design Parameters**
Minimum Embedment into Bearing Stratum [1]	4 feet into Stratum 4 limestone
Minimum Concreted Pier Length [1]	4 feet or 2 times the pier diameter, whichever is greater
Minimum Pier Diameter	18 inches
End Bearing Pressure (net allowable) [1,3]	110,000 psf for piers bearing into Stratum 4 limestone
Side Friction (net allowable) [2,3]	10,000 psf for pier lengths beyond the 4-foot minimum embedment
Minimum Percentage of Steel [4]	½ percent
Approximate Total Settlement [5,6]	¾-inch maximum
Estimated Differential Settlement [5,6]	Approximately ½ to ¾ of total maximum

1. To bear at least 4 feet into the Stratum 4 limestone.
2. For pier lengths embedded beyond the 4-foot minimum embedment. In addition, side friction may not be accounted for in any permanently cased portions of the pier.
3. A one-third increase in allowable bearing and side friction may be used with the alternative load combinations given in Section 1605.3.2 of the IBC. This is permitted on the basis of reducing the factor of safety for transient loads such as wind or seismic loads in the allowable values for end bearing from about 3 to 2.25 and for side friction from about 2 to 1.5.
4. Soil-related uplift does not appear to be a concern at this site, assuming the recommendations in **Earthwork** and **Floor Slabs** are followed. However, we do recommend that the minimum percentage of reinforcing steel be no less than ½ percent of the gross shaft area and extend over the full length of the pier.

5. Provided proper construction practices are followed. For adjacent piers, we recommend a minimum edge-to-edge spacing of at least 1 pier diameters (or 2 pier diameters center to center) based on the larger pier diameter of the two adjacent piers. In locations where this minimum spacing criterion cannot be accomplished, Terracon should be contacted to evaluate the locations on a case-by-case basis.
6. Will result from variances in the subsurface conditions, loading conditions and construction procedures, such as cleanliness of the bearing area or flowing water in the shaft.

Drilled Pier Lateral Loading

The following table lists input values for use in LPILE analyses. LPILE will estimate values of k_h and E_{50} based on strength; however, non-default values of k_h should be used where provided. Since deflection or a service limit criterion will most likely control lateral capacity design, no safety/resistance factor is included with the following lateral parameters.

Stratum [1]	L-Pile Soil Model	S_u (psf) [2]	f [2]	g (pcf) [2,3]	ε_{50} [2]	K (pci) [2]		RQD [2]
						Above GWT	Below GWT	
1	Stiff Clay w/o Free Water	1,000	---	120	0.01	---	---	---
4	Strong Rock	2,200 psi [4]	---	130	---	---	---	---

1. See **Subsurface Profile** in **Geotechnical Characterization** for more details on Stratigraphy.
2. Definition of Terms:

   S_u: Undrained shear strength

   f : Internal friction angle,

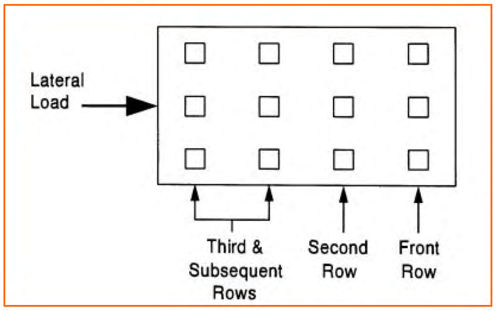
   g: Total unit weight

   ε_{50}: Non-default E50 strain

   K: Horizontal modulus of subgrade reaction

   RQD: Rock Quality Designation
3. Buoyant unit weight values should be used below water table.
4. This value is the uniaxial compressive strength of the rock in psi.
5. Lateral resistance should be neglected in the upper 2 feet of soil in contact with the pier.

When piers are used in groups structurally connected together with a large pier cap or mat, the lateral capacities of the piers in the second, third, and subsequent rows of the group should be reduced as compared to the capacity of a single, independent shaft. Guidance for applying p-multiplier factors to the p values in the p-y curves for each row of pier foundations within a pier group are as follows:



- Front row: $P_m = 0.8$;
- Second row: $P_m = 0.4$
- Third and subsequent row: $P_m = 0.3$.

For the case of a single row of piers socketed into limestone supporting a laterally loaded grade beam, group action for lateral resistance of piers would need to be considered when spacing is less than two pier diameters (measured center-to-center). However, spacing closer than 2D (where D is the diameter of the pier) is not recommended, due to potential for the installation of a new pier disturbing an adjacent installed pier, likely resulting in axial capacity reduction.

Spacing between Footings and Drilled Piers

We understand that there exist situations that may require footings and drilled piers to be closely spaced. While full design is not completed, we anticipate that drilled piers will use a combination of end bearing and side friction to develop capacity, while the footings will use end bearing only to develop capacity. In addition, we anticipate that the drilled piers will be founded several feet (or more) deeper than the adjacent footings. Based on this, there will be stress overlap between the adjacent foundation units. To alleviate this concern, the following options are available for consideration.

If the adjacent footings and piers are bearing into limestone the following may be considered.

- Design the footings for the lowest bearing values in limestone tabulated in **Shallow Foundations**.
- No end bearing reduction will be necessary for either the footings or the piers regardless of the very close spacing.

For side friction of the drilled pier, there is no reduction necessary, provided that the edge-to-edge distance from the pier to the footing excavation is equal to or greater than 1 pier diameter or 1 footing width, whichever is greater. If the footing excavation encroaches closer than that, interpolate a reduction factor from 1.0 at ≥1 times the pier diameter/footing width spacing to a reduction factor of 0.0 if the footing/pier elements are touching (i.e., tangent). If that is the case, ignore side friction in the pier to a depth of 1 foot below the bottom of the tangent footing.

If the footings are bearing into on-site soils or select fill (while the piers are bearing into limestone) the following may be considered. We understand that this may be the case where Stadium piers/columns will extend through and below the Concessions level.

- No reduction in end bearing for the footings is necessary, provided that the edge-to-edge distance from the pier to the footing excavation is equal to or greater than 1.5 times the footing width. If the footing excavation encroaches closer than that, interpolate a reduction factor from 1.0 at ≥1.5 times the footing width spacing to a reduction factor of 0.3 if the footing element is as close to the piers as 0.5 times the footing width. No reductions in side friction or end bearing is necessary for the driller pier since it will bear deeper into the Stratum 4 limestone.

Drilled Pier Construction Considerations

Drilled pier foundations should be augered and constructed in a continuous matter. Concrete should be placed in the pier excavations following drilling and evaluation for proper bearing stratum, embedment, and cleanliness. The piers should not be allowed to remain open overnight before concrete placement. Surface runoff or groundwater seepage accumulating in the excavation should be pumped out and the condition of the bearing surface should be evaluated immediately prior to placing concrete. The drilling equipment utilized should be readily capable of excavating the Stratum 4 limestone observed at this site. Drilling equipment with insufficient torque and/or augers/bits/core barrels that are not suited for variable and/or hard rock conditions will likely result in poor production rates.

As encountered during our field program, zones of groundwater inflow and/or sloughing soils are a possibility during pier construction at this site. Therefore, provisions must be incorporated into the plans and specifications to utilize casing to control sloughing and/or groundwater seepage during pier construction.

The use of casing should help to minimize groundwater inflow into the pier excavation. If soil sloughing or groundwater seepage is encountered at the proposed depth of a pier, it may be necessary to extend the excavation to a depth where the casing can control sloughing and/or seal off groundwater. If seepage persists even after casing installation and casing extension, the water should be pumped out of the excavation immediately prior to placing concrete. If groundwater inflow is too severe to be controlled by pumping, the concrete should be tremied to the full depth of the excavation to effectively displace the water. In this case, a “clean-out” bucket should be used to remove loose soil and/or rock fragments from the pier bottom before placing steel and concrete.

Care should be taken to not disturb the sides and bottom of the excavation during construction. The bottom of the shaft excavation should be free of loose material before concrete placement. Water or loose soil should be removed from the bottom of the drilled shafts prior to placement of the concrete. Concrete should be placed as soon as possible after the foundation excavation is completed, to reduce potential disturbance of the bearing surface.

Concrete should exhibit slump as designated in Structural Engineer's specifications. A design concrete slump of 6 to 8 inches helps to facilitate removal of casings and reduces the possibility of concrete arching/honeycombing. Under no circumstance should loose soil be placed in the space between the casing and the pier sidewalls. The concrete should be placed using a rigid tremie or by the free-fall method provided the concrete falls to its final position through air without striking the sides of the hole, the reinforcing steel cage, or any other obstruction. A drop chute should be used for this free-fall method.

While withdrawing casing, care should be exercised to maintain concrete inside the casing at a sufficient level to resist earth and hydrostatic pressures acting on the casing exterior. Arching of the concrete, loss of seal, mixing of the surrounding soil and water with fresh concrete, and other problems can occur during casing removal and result in contamination of the drilled shaft. These conditions should be considered during the design and construction phases. Placement of loose soil backfill should not be permitted around the casing prior to removal.

The drilled shaft installation process should be monitored under the direction of the Geotechnical Engineer. The Geotechnical Engineer should document the shaft installation process including soil/rock and groundwater conditions encountered, consistency with expected conditions, and details of the installed shaft.

Grade Beams between Drilled Piers

Grade beams spanning between drilled piers may be cast at-grade provided the subgrade in the beam areas is prepared as outlined in **Floor Slabs**. Grade beams should be designed to span across the drilled pier foundations without subgrade support, due to stress/strain incompatibility between different bearing materials at varying depths.

We recommend that fat clay soils (LL≥50 and PI>30) be utilized for backfill adjacent to grade beams at the exterior surface of the structure (to reduce potential infiltration of surface water into the subgrade areas). The exterior backfill should be compacted as outlined in **Earthwork**. On the interior sides of the perimeter grade beams, backfill should consist of properly compacted select fill or flowable fill (COA Item 402 or TxDOT Item 401), not sand or gravel. Compaction of select fill on the interior sides of beams should be performed by the Earthwork Contractor's personnel and equipment, not by concrete or utility contractors inexperienced with proper soil placements and compaction.

Foundation Construction Observation

The performance of the foundation system for the proposed structure will be highly dependent upon the quality of construction. Thus, we recommend that the foundation installation be monitored by Terracon to identify the proper bearing strata and depths and to help evaluate foundation construction. We would be pleased to develop a plan for foundation monitoring to be incorporated in the overall quality assurance program.

SEISMIC CONSIDERATIONS

The seismic design requirements for buildings and other structures are based on Seismic Design Category. Site Classification is required to determine the Seismic Design Category for a structure. The Site Classification is based on the upper 100 feet of the site profile defined by a weighted average value of either shear wave velocity, standard penetration resistance, or undrained shear strength in accordance with Section 20.4 of ASCE 7 and the International Building Code (IBC). Based on the bedrock properties encountered at the site and as described on the exploration logs and results, it is our professional opinion that the **Seismic Site Classification is B**. Subsurface explorations at this site were extended to a maximum depth of 75 feet. The site properties below the boring depth to 100 feet were estimated based on our experience and knowledge of geologic conditions of the general area. Additional deeper borings or geophysical testing may be performed to confirm the conditions below the current boring depth.

FLOOR SLABS

The subgrade soils at existing grades are comprised of moderate to high plasticity soils exhibiting the potential to shrink/swell with changes in water content. Construction of the floor slabs and revising site drainage creates the potential for gradual increased water contents within the soils. Increases in water content may cause the soils to swell and potentially damage the floor slabs.

Due to the potential for significant moisture fluctuations of subgrade material beneath the select fill pad, the exposed final subgrade should be prepared as discussed in the first three sub-sections of **Earthwork**.

The post-construction performance of the foundation will likely be influenced more by post-construction volumetric changes of the subgrade due to in-situ moisture variations than upon settlement due to foundation loads. Settlement response of select fill supported slabs will be influenced as much by the quality of construction and fill placements as by soil-structure interaction. Therefore, it is essential that the recommendations for foundation construction be strictly followed during the construction phases of the building pad and foundation.

Floor Slab Subgrade Preparation

Based on information provided to us, we understand that an FFE of 747 feet is anticipated for the pitch (playing field) as well as for the below-grade slab portions located on the southern side of the stadium structure. In addition, an FFE of 770 feet is anticipated for the at-grade (upper level) of the stadium structure. Existing grades at the time of our report in the structural area range from about 755 feet to 768 feet. If the FFE changes, Terracon should be notified to review and modify or verify recommendations in writing. For illustration purposes, please see the **Subgrade Preparation Model** in **Figures**.

Pitch Subgrade Preparation

We understand that the pitch surface will most likely consist of either natural turf or artificial turf. Typically, these systems contain a growing medium (for the natural turf) and/or a drainage medium (both pitch surfaces). These systems are to be designed by others, including the drainage system. In order to reduce PVR to 1-inch or less, we recommend that the on-site soils be excavated to a depth of 4 feet below FFE or until the Stratum 4 limestone is encountered whichever occurs first. The removed soils must then be replaced with properly compacted select fill up to finished grades. In shallow limestone subgrade areas, a minimum thickness of 12 inches of select fill must be provided underneath all pitch surfaces. The excavations must also be sufficiently deep in order to achieve the turf requirements for growing/drainage medium. Based on our borings, it is anticipated that the Stratum 4 limestone will be encountered almost throughout the entire pitch excavation, except for the southeast corner of the site, where the Stratum 1 fat clay fill soils were encountered to an elevation of 741.5 feet at boring B-11. Corresponding to Area "A" in the **Subgrade Preparation Model*.***

Slab On Grade at Stage Area Subgrade Preparation

We understand that the southern portion of the stadium structure will be located below grade with an anticipated FFE of 747 feet. We recommend that for these areas, the recommendations provided for the pitch subgrade preparation be followed in order to reduce PVR to 1-inch or less. Corresponding to Area "B" in the **Subgrade Preparation Model*.***

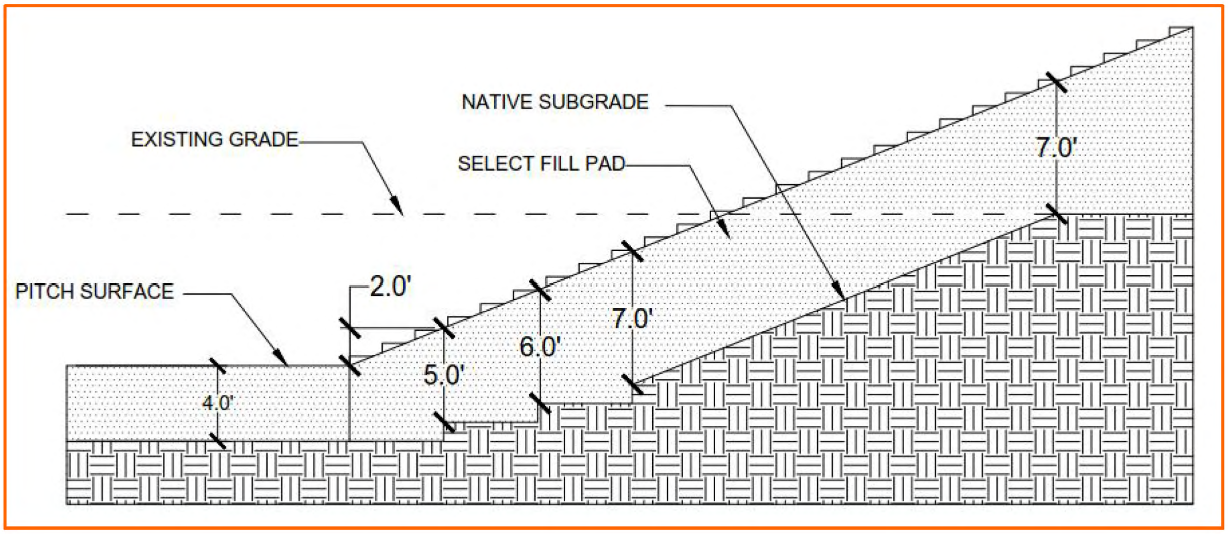
Slab On Grade Subgrade Preparation

We understand that the north, west, and east portions of the stadium structure will be located at-grade with an anticipated FFE of 770 feet. In order to reduce PVR to 1-inch or less, we recommend that the on-site soils be excavated to a depth of 6 inches below existing grades or until the Stratum 4 limestone is encountered whichever occurs first. The excavated soils must then be moisture conditioned and properly recompacted. Properly compacted select fil must be used to raise grades to finished grades. Corresponding to Area "C" in the **Subgrade Preparation Model*.***

Slab On Grade Bowl Seating Subgrade Preparation

We understand that the seating for the southern portion of the stadium structure will be grade-supported from an elevation of about 747 feet to 770 feet. In order to reduce PVR to 1-

inch or less, we recommend that the on-site soils under the stadium seating near the pitch (at EL 747 feet) be excavated to a depth of 4 feet below finished grades or until the Stratum 4 limestone is encountered whichever occurs first. As finished grades rise for the stadium seating (up to ~EL753 feet), the select fill thickness must be increased accordingly to 7 feet (i.e., a 1-foot increase in select fill for every 2 feet of rise in finished grades). From EL 753 feet to 770 feet, select fill pad should remain at least 7 feet thick below existing grades, but should also be increased in thickness to account for needed fill above existing grades. All grade changes above existing grades must be made with properly compacted select fill. Please see the sketch below for an illustration of this condition. Corresponding to Area "D" in the **Subgrade Preparation Model*.***



Performance Venue Subgrade Preparation

We understand that a performance venue is planned in the eastern portion of the site (near boring B-12). Finished grades for the performance venue are currently unknown at this time, however we anticipate that finished grades will be within 2 feet of existing grades. In order to reduce PVR to 1-inch or less, we recommend that the on-site soils be excavated to a depth of 5 feet below existing grades. The removed soils must then be replaced with properly compacted select fill up to finished grades.

General Slab Preparation Comments

The above building subgrade preparation recommendations (for at-grade stadium structure floor slab subgrade preparation) should be applied to an area including attached flatwork, sidewalks, ramps, etc. to reduce differential movements between the flatwork and the adjacent structures. If subgrade preparation as given above is not implemented in the exterior flatwork areas, those areas may be susceptible to post-construction movements between 1 and 2½ inches, potentially leading to differential movements that can result in trip hazards. In all pad areas, we suggest the use of crushed limestone base in the upper 6 inches of the select fill pad from a standpoint of construction access during wet weather, as well as from a standpoint of floor slab support.

The use of a vapor retarder should be considered beneath concrete slabs on grade covered with wood, tile, carpet, or other moisture sensitive or impervious coverings, or when the slab will support equipment sensitive to moisture. When conditions warrant the use of a vapor retarder, the slab designer should refer to ACI 302 and/or ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder.

The potential movement values indicated are based upon moisture variations in the subgrade due to circumstances such as moisture increases due to rainfall and loss of evapotranspiration. In circumstances where significant water infiltration beneath the floor slab occurs (such as a leaking utility line or water seepage from outside the buildings resulting from poor drainage), movements in isolated floor slab areas could potentially be in excess of those indicated in this report.

Where floor slabs are tied to perimeter walls or turn-down slabs to meet structural or other construction objectives, our experience indicates differential movement between the walls and slabs will likely be observed in adjacent slab expansion joints or floor slab cracks beyond the length of the structural dowels. The Structural Engineer should account for potential differential settlement through use of sufficient control joints, appropriate reinforcing or other means. Saw-cut control joints should be placed in the slab to help control the location and extent of cracking. For additional recommendations refer to the ACI Design Manual.

Floor Slab Construction Considerations

Design recommendations for floor slabs assume the requirements in **Earthwork** have been followed. Specific attention should be given to positive drainage away from the structure and positive drainage of the subgrade and select fill pad beneath the floor slab.

Finished subgrade within and for at least 10 feet beyond the floor slab should be protected from traffic, rutting, or other disturbance and maintained in a relatively moist condition until floor slabs are constructed. If the subgrade should become damaged or desiccated prior to construction of floor slabs, the affected material should be removed and structural fill should be added to replace the resulting excavation. Final conditioning of the finished subgrade should be performed immediately prior to placement of the floor slab support course. Attention should be paid to high traffic areas that were rutted and disturbed earlier, and to areas where backfilled trenches are located.

BELOW-GRADE STRUCTURES

We understand that permanent below-grade basement walls (up to 23 feet below finished grades) are planned and we anticipate that these will be constructed in open cut/sloped excavations. The following discussion should be reviewed and updated if finished floor elevations are changed.

Lateral earth pressure conditions and drainage requirements are discussed in the following sections.

The below-grade drainage system should be designed to intercept, collect, and pump out the groundwater. The actual groundwater flowrates will depend on many factors such as in-situ permeability of the soils, fissures in the soils, the variable climate and precipitation amounts in the Austin area over the life of the structure, changes in surrounding groundwater flowpaths due to nearby below-grade construction and/or utility installations, etc. Based on our current information and experience in the Austin, our estimates for groundwater flowrates vary from a low end of 5 gallons per minute (gpm) to a high end of about 50 gpm (This estimate does not include any rainfall or irrigation that may infiltrate directly from the pitch. We anticipate that the pitch will have its own separate drainage and dewatering system). The MEP should take this into account when designing the overall system and include an appropriate factor of safety in the dewatering capacity. We do recommend that the General Contractor and Earthwork Contractor monitor and document actual groundwater seepage during construction and attempt to measure actual flowrates as water is being pumped out of the excavation. When this is done, final verification (and/or modification) to the pump sizing can be performed prior to pump installation(s).

Below-Grade Wall Drainage

A permanent perimeter drainage system should be designed and constructed adjacent to the below-grade walls to reduce potential hydrostatic pressures on the permanent walls. For the project, we recommend that the walls be designed to resist soil-related lateral earth pressures with complete wall drainage extending all the way down to the lowest basement level for collection and removal of water at the lowest elevations. This wall system would need to connect into the perimeter drainage system and then to a sump-and-pump removal system. Based on the observed groundwater elevations, this system will most likely be operating continuously.

Wall Drainage Components

The wall drainage system should be located behind the perimeter wall system. The system should be designed to gravity flow toward common sump areas for collection and removal of water.

- The below-grade walls should be waterproofed.
- A drainage mat (or a 12-inch wide “chimney” of clean washed drainage gravel) should also be provided behind the permanent below-grade walls. (The manufacturer of the geotextile drainage mat should be consulted in regards to applicability, selection and placement of the drainage mat. In addition, a representative of the drainage mat manufacturer should be present during initial and/or critical phases of the installation such that proper installation techniques are used.) The only exception to the above would be the sump pit area(s).
- The drainage mat or clean washed drainage gravel should extend over the full height and length of the below-grade walls. Proper control of surface water percolation will help to

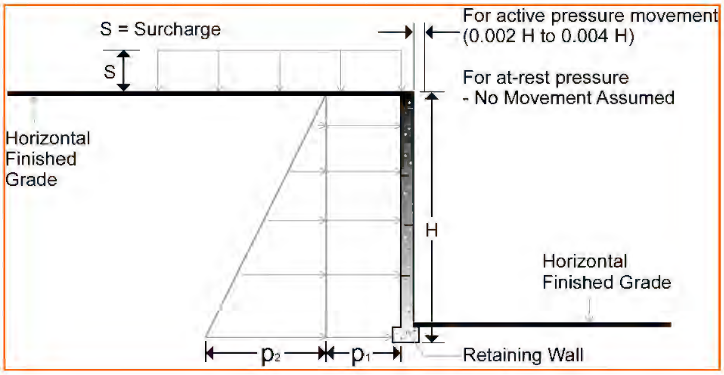
prevent buildup of higher wall pressures. The final 12 to 18 inches of backfill near final grades should preferably consist of fine-grained cohesive clay soils (CH or CL) or flowable fill (COA Item 402 or TxDOT Item 401). This will help to reduce percolation of surface water into the wall backfill.

- The perimeter drainage trenches should be sloped to drain toward common sump pit area(s). We suggest the southeast corner of the stadium as B-11 exhibited the lowest elevations to the top of limestone.
- The perimeter drainage trenches should extend to a depth of at least 18 inches below the below-grade floor slab with a minimum width of at least 12 inches but should also extend at least 6 inches into the Stratum 4 limestone to intercept the water. At the area near B-11 with the lowest top of limestone elevation, this will mean deepening the trench in that area.
- Perforated collector pipes with a minimum diameter of 4 inches should be provided near the bottom of all gravel-packed trenches (within 1 to 2 inches).
- The gravel should be a clean, washed aggregate meeting the specifications for a Type B or C material according to TxDOT Item 556, or Pipe Bedding Stone per COA Item (Gravel meeting ASTM C33, Grade 57 or 67 would also be acceptable).
- Periodic maintenance of drainage systems is necessary so that they do not become plugged and inoperative, thus periodic cleanout locations should be installed in the system.
- The sizing of the perimeter collection system and the pump capacity should be designed by the MEP (mechanical, electrical, and plumping) engineer to transmit the intercepted water to a permanent sump-and-pump system. We recommend that the MEP engineer design a dual-pump system with an adequate pumping capacity to easily exceed our estimated flowrates given in **Below-Grade Wall Drainage** due to rainfall infiltration plus some additional safety-factored capacity. The actual flowrate can be monitored by the General Contractor during construction, thus allowing some modifications (if needed) to be made to the permanent system prior to installation.

LATERAL EARTH PRESSURES

Design Parameters

Structures with unbalanced backfill levels on opposite sides should be designed for earth pressures at least equal to values indicated in the following table. Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction and/or compaction and the strength of the materials being restrained. Two wall restraint conditions are shown. Active earth pressure is commonly used for design of free-standing cantilever retaining walls and assumes wall movement. The "at-rest" condition assumes no wall movement and is commonly used for basement walls, loading dock walls, or other walls restrained at the top. The recommended design lateral earth pressures do not include a factor of safety and do not provide for possible hydrostatic pressure on the walls (unless stated). The recommendations in this section apply to those walls which are installed in open cut or embankment fill areas such that the backfill extends out from the base at an angle of at least 45 degrees from vertical for the entire height and length of the wall.



Lateral Earth Pressure Design Parameters				
Backfill Type	Estimated Total Unit Weight, pcf[1]	Lateral Earth Pressure Coefficients[2]		
		At Rest, K_o	Active, K_A	Passive, K_P
Crushed Limestone (Select Fill) [3]	135	0.45	0.3	3.5
Clean Sand	120	0.5	0.35	3.0
Clean Gravel	120	0.45	0.3	3.5
Alternate Select Fill (such as on-site low plasticity soils with PI ≤ 20)	125	0.5	0.35	3.0

1. Compaction should be maintained between 95 and 100 percent of Standard Proctor (ASTM D 698) maximum dry density. Overcompaction can produce lateral earth pressure coefficients in excess of those provided.
2. Coefficients represent nominal (unfactored) values. Appropriate safety factors should be applied.
3. In areas where the retaining wall backfill “intersects” the select fill pad, the material for the retaining wall backfill must consist of crushed limestone or alternative select fill.

The above values do not include a hydrostatic or ground-level surcharge component. To prevent hydrostatic pressure build-up, retaining walls should incorporate functional drainage (via free-draining aggregate or manufactured drainage mats) within the backfill zone. The effect of surcharge loads, where applicable, should be incorporated into wall pressure diagrams by adding a uniform horizontal pressure component equal to the applicable lateral earth pressure coefficient times the surcharge load, applied to the full height of the wall.

All retaining walls should be checked against failure due to overturning, sliding and overall slope stability. Such an analysis can only be performed once the dimensions of the wall and cut/fill scenarios are known. For retaining wall bearing capacity design, we recommend the values provided in **Design Parameters – Footings** in **Shallow Foundations** be applied.

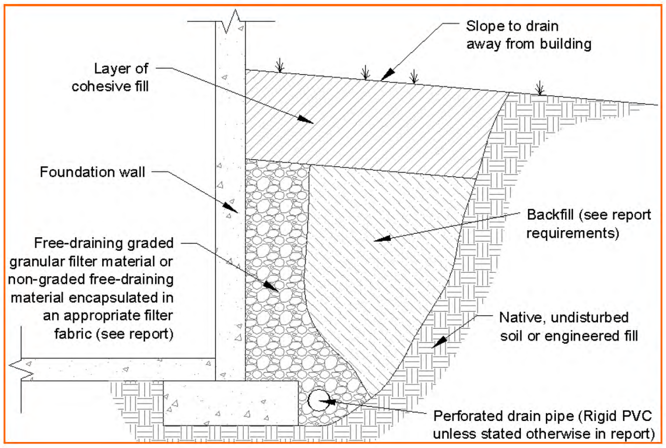
There exists a high movement potential for any retaining walls bearing on the native on-site soils (up to 2.5 inches). If lower movement potential is desired, wall areas should be prepared with select fill as outlined in **Floor Slabs** or wall footings should extend to bear on Stratum 4 limestone.

Frequent joints should be provided throughout the length of the retaining wall to reduce cracking due to differential movements caused by the shrink/swell movement of the fat clay subgrade.

We recommend that a "buffer zone" of at least 5 feet wide be applied between pavement areas and retaining walls (with a minimum height of 4 feet or more). This buffer zone should be increased to 10 feet for building areas. These recommended buffer zones are to reduce the potential of distress from any long-term ("creep") movements of the wall and backfill. Pedestrian sidewalks may be exempted from the above criteria, however some distress could still be observed in the sidewalks due to movements of the retaining walls and backfill.

Subsurface Drainage for Site Retaining Walls

A perforated rigid plastic drain line installed behind the base of walls and extending below adjacent grade is recommended to prevent hydrostatic loading on the walls. The invert of a drain line around an exterior retaining wall should be placed near foundation bearing level. The drain line should be sloped to provide positive gravity drainage to daylight or to a sump pit and pump. The drain line should be surrounded by clean, free-draining granular material having less than 10 percent passing the No. 8 sieve, such as No. 57 aggregate. The free-draining aggregate should be encapsulated in a filter fabric. The granular fill should extend to within 2 feet of final grade, where it should be capped with compacted cohesive fill to reduce infiltration of surface water into the drain system. Below-grade slab and wall drainage is discussed in **Below-Grade Structures**.



As an alternative to free-draining granular fill, a pre-fabricated drainage structure may be used. A pre-fabricated drainage structure is a plastic drainage core or mesh which is covered with filter fabric to prevent soil intrusion and is fastened to the wall prior to placing backfill.

PAVEMENTS

General Pavement Comments

Pavement designs are provided for the traffic conditions and pavement life conditions as noted in the following sections of this report. A critical aspect of pavement performance is site preparation. Pavement designs, noted in this section, must be applied to the site, which has been prepared as recommended in the **Earthwork** section.

Pavement designs are intended to provide structural sections with adequate thickness over a particular subgrade such that wheel loads are reduced to a level the subgrade can support. Support characteristics of the subgrade for pavement design do not account for shrink/swell movements of an expansive clay subgrade, such as the Stratum 1 and Stratum 2 soils encountered on this project. Thus, the pavement may be adequate from a structural standpoint, yet still experience cracking and deformation due to shrink/swell related movement of the subgrade. It is therefore important to minimize moisture changes in the subgrade to reduce shrink/swell movements. Proper site perimeter drainage should be provided so that infiltration of surface water from unpaved areas surrounding the pavement is minimized.

Pavement Design Parameters

Design of Asphaltic Concrete (HMAC) pavements are based on the procedures outlined in the 1993 Guideline for Design of Pavement Structures by the American Association of State Highway and Transportation Officials (AASHTO-1993). Design of Portland Cement Concrete (PCC) pavements are based upon American Concrete Institute (ACI) 330R-01; Guide for Design and Construction of Concrete Parking Lots.

Detailed traffic loads and frequencies were not available, however we anticipate that traffic will consist primarily of passenger vehicles in the parking areas and passenger vehicles combined with emergency vehicles, occasional garbage trucks, team buses, service trucks, maintenance vehicles, and delivery trucks in driveways. If heavier traffic loading is expected or other traffic information is available, Terracon should be provided with the information and allowed to review the pavement sections provided herein. Tabulated below are the assumed traffic frequencies and loads used to design pavement sections for this project.

Pavement Area	Traffic Design Index	Description
Parking Areas (Passenger Vehicles Only)	DI-1	Light traffic – (ESALs [1] <5) Passenger cars and pickup trucks, no regular use by heavily loaded two axle trucks or lightly loaded larger vehicles.
Secondary Driveways (non-Delivery or Loading Areas)	DI-2 [2]	Light to medium traffic – (5≤ESALs≤20) Passenger cars and pickup trucks with no more than 50 heavily loaded two-axle trucks or lightly loaded three axle trucks per day. No regular use by heavily loaded trucks with three or more axles.
Primary Driveways, Delivery Areas, Loading/Unloading Areas and Dumpster Enclosures	DI-3	Medium traffic – (20<ESALs≤75) No more than 300 heavily loaded two axle trucks or lightly loaded three axle trucks and no more than 30 heavily loaded three axle trucks per day.

1. 18-kip equivalent single axle load applications.
2. For Fire Lanes to withstand the occasional HS-20 loading of 32,000 pounds per axle and 90,000-pound gross truck weight, use DI-2 pavements or thicker.

An estimated average subgrade CBR of 4 was used for HMAC pavement designs, and an estimated modulus of subgrade reaction of 100 pci was used for the PCC pavement designs. The values were empirically derived based upon our experience with the described subgrade soils and our understanding of the quality of the subgrade as prescribed by the **Site Preparation** conditions as outlined in **Earthwork**.

Local drainage characteristics of proposed pavements areas are considered to vary from poor to fair. For purposes of this design analysis, poor drainage characteristics are considered to control the design. These characteristics, coupled with the approximate duration of saturated subgrade conditions, results in a design drainage coefficient of 1.0 when applying the AASHTO criteria for design.

Pavement Section Thicknesses

The following tables provides options for private HMAC and PCC pavement sections. If any pavement areas to be public City of Austin roads, please let us know to re-evaluate.

Asphaltic Concrete Design		
Layer	Thickness (inches)	
	DI-1	DI-2
Asphaltic Concrete (HMAC)	2.0	2.5
Crushed Limestone Base [1]	10.0	12.0

Asphaltic Concrete Design		
Layer	Thickness (inches)	
	DI-1	DI-2
Moisture Conditioned Subgrade [2]	6.0	6.0

1. If the on-site soils are completely removed to expose the Stratum 4 limestone, the crushed limestone base thickness may be reduced by up to 2 inches, but in no case less than 6 inches thick.
2. Moisture conditioning is not necessary in areas where Stratum 4 limestone is exposed.

Portland Cement Concrete Design			
Layer	Thickness (inches)		
	DI-1	DI-2	DI-3
Reinforced Concrete (PCC) [1,2]	5	6	7
Moisture Conditioned Subgrade	6	6	6

1. A thin course of crushed limestone base or clean sand at least 1 to 2 inches thick is recommended under the reinforced concrete in exposed Stratum 4 limestone subgrade areas. Moisture conditioning of the subgrade is not necessary in intact limestone areas.
2. In Stratum 4 limestone areas, the DI-2 and DI-3 concrete thicknesses may be reduced by ½ inch.

Rigid PCC pavements will perform better than HMAC pavements in areas where short-radii turning and braking are expected (i.e. entrance/exit aprons) due to better resistance to rutting and shoving. In addition, PCC pavements will perform better in areas subject to large or sustained loads, such as loading docks, dumpster enclosures, and loading/unloading areas.

Areas for parking of heavy vehicles, concentrated turn areas, and start/stop maneuvers could require thicker pavement sections. Edge restraints (i.e. concrete curbs or aggregate shoulders) should be planned along curves and areas of maneuvering vehicles. As an option, thicker sections could be constructed to decrease future maintenance.

Permeable Grass Pavers

We understand that a permeable grass paver system is being considered for the north parking lot and possibly for a fire lane on the northeast side of the stadium. Most of these permeable grass paver systems include design details provided by the manufacturer. Some examples of these products include the TrueGrid ProPlus® and Invisible Structures GrassPave2®. If implemented correctly, the permeable grass paver systems can support occasional HS20 loadings (16 kips/wheel) and a total live load of 90,000 pounds. Any non-paved surfaces that will be used for vehicle access, especially fire truck access, must consist of permeable grass pavers. The final design for the grass pavers will be dependent on what system is selected, however the following recommendations apply for all systems.

- Once initial subgrade is exposed (after initial cuts and prior to any fills) the soil subgrade must be thoroughly proof-rolled as outlined in **Proof-Rolling** in **Earthwork**.
- Once the subgrade passes proof-roll, the soil subgrade should then be scarified to a depth of 12 inches and compacted as per **Fill Compaction Requirements** in **Earthwork**. If the permeable grass paver system requires fill to achieve finished grades, grade changes can be made with paving fill material as outlined in **Fill Material Types** in **Earthwork**.
- Once the subgrade and/or paving fill has been properly compacted, the permeable grass paver system may be installed. We recommend a minimum crushed limestone base thickness of 12 inches in fire lane areas and a minimum base thickness of 10 inches in vehicular parking lots. If the grass paver system will be used for water detention and infiltration, please contact us for further recommendations.
- Ribbon curbs are required along all perimeters of the permeable grass paver areas to provide perimeter lateral confinement. The depth of the concrete ribbon curb should extend from final ground surface to at least 2 inches below the bottom of the stone/base material.
- The porous grass paver areas must be designed and constructed to provide positive drainage away from the fire lanes such that ponding of surface water does not occur in, on, or adjacent to the fire lanes following rainfall events.
- The porous grass paver system and surrounding grades should be maintained throughout the years and not allowed to deteriorate.
- All wheel loads (and outriggers) associated with the fire trucks and other emergency vehicles must remain on the porous grass paver system at all times and the vehicles should not pull out onto the adjacent soils outside of the ribbon curbs.

Pavement Materials

Presented below are our recommended material requirements for the various pavement sections.

Item	Value
Hot Mix Asphaltic Concrete (HMAC) [1]	Plant mixed, hot laid Type D (Fine-Grade Surface Course) meeting the specifications in TxDOT Item 340 or COA Item 340.
Reinforced Portland Cement Concrete (PCC)	28-day flexural strength (third-point loading) ≥ 500 psi, or 28-day compressive strength ≥ 3,500 psi
Crushed Limestone Base [2]	TxDOT Item 247, Type A, Grade 1-2 or COA Item 210S compacted as outlined in **Earthwork**.
Moisture Conditioned Subgrade [3]	As outlined in **Earthwork**.

1. For acceptance and payment evaluation purposes, we recommend the use of the provisions in COA Item 340.

2. Each lift of base should be thoroughly proof-rolled just prior to placement of subsequent lifts and/or asphalt. Particular attention should be paid to areas along curbs, above utility trenches, and adjacent to landscape islands, manholes, and storm drain inlets. Preparation of the base material should extend at least 18 inches behind curbs.
3. Subgrade should not dry out or become saturated prior to pavement construction. The initial (prior to any fill) and final (prior to any base) pavement subgrade should be thoroughly proof-rolled as outlined in **Earthwork**. Particular attention should be paid to areas along curbs, above utility trenches, and adjacent to landscape islands, manholes, and storm drain inlets. Preparation of the moisture conditioned subgrade should extend at least 18 inches behind curbs.

Presented below are our recommendations for the construction of the reinforced concrete pavements.

Item	Value
Reinforcing Steel	#3 bars spaced at 18 inches on center in both directions
Control (i.e., Contraction) Joint Spacing	In accordance with ACI 330R-08, control joints should be spaced no greater than 12.5 feet for 5-inch thick concrete and 15 feet for 6-inch thick or greater concrete. If sawcut, control joints should be cut within 6 to 12 hours of concrete placement. Sawcut joint should be at least ¼ of the slab thickness.
Expansion (i.e., Isolation) Joint Spacing	ACI 330R-08 indicates that regularly spaced expansion joints may be deleted from concrete pavements. Therefore, the installation of expansion joints is optional and should be evaluated by the design/construction team. Expansion joints, if not sealed and maintained can allow infiltration of surface water into the subgrade.
Dowels at Expansion Joints	¾-inch smooth bars, 18 inches in length, with one end treated to slip, spaced at 12 inches on centers at each joint.

Pavement Drainage

On most projects, rough site grading is accomplished relatively early in the construction phase. Fills are placed and compacted in a uniform manner. However, as construction proceeds, excavations are made into these areas, dry weather may desiccate some areas, rainfall and surface water saturates some areas, heavy traffic from concrete and other delivery vehicles disturbs the subgrade, and many surface irregularities are filled in with loose soils to temporarily improve subgrade conditions. As a result, the pavement subgrade should be carefully evaluated as the time for pavement construction approaches. This is particularly important in and around utility trench cuts. All pavement areas should be moisture conditioned and properly compacted to the recommendations in this report immediately prior to paving. Thorough proof-rolling of pavement areas should be performed no more than 36 hours prior to surface paving. Proof-rolling should be repeated if the site received rainfall prior to paving. Any problematic areas should be reworked and compacted at that time.

Openings in pavements, such as landscaped islands, are sources for water infiltration into surrounding pavement systems. Water can collect in the islands and migrate into the surrounding subgrade soils thereby degrading support of the pavement. This is especially applicable for islands with raised concrete curbs, irrigated foliage, and low permeability near-surface soils. The civil design for the pavements with these conditions should include features to restrict or to collect and discharge excess water from the islands. Examples of features are self-contained planters, edge drains connected to the storm water collection system, longitudinal subdrains, or other suitable outlet, and impermeable barriers preventing lateral migration of water such as a cutoff wall installed to a depth below the pavement structure.

Pavements should be sloped to provide rapid drainage of surface water. Water allowed to pond on or adjacent to the pavements could saturate the subgrade and contribute to premature pavement deterioration. In addition, the pavement subgrade should be graded sufficiently to provide positive drainage within the granular base section. Appropriate sub-drainage or connection to a suitable daylight outlet should be provided to remove water from the granular subbase.

Pavement Maintenance

The pavement sections represent minimum recommended thicknesses and, as such, periodic maintenance should be anticipated. Therefore, preventive maintenance should be planned and provided for through an on-going pavement management program. Maintenance activities are intended to slow the rate of pavement deterioration and to preserve the pavement investment. Maintenance consists of both localized maintenance (e.g. crack and joint sealing and patching) and global maintenance (e.g. surface sealing). Preventive maintenance is usually the priority when implementing a pavement maintenance program. Additional engineering observation is recommended to determine the type and extent of a cost-effective program. Even with periodic maintenance, some movements and related cracking may still occur and repairs may be required.

Pavement performance is affected by its surroundings. In addition to providing preventive maintenance, the civil engineer should consider the following recommendations in the design and layout of pavements:

- Final grade adjacent to paved areas should slope down from the edges at a minimum 2%.
- Subgrade and pavement surfaces should have a minimum 2% slope to promote proper surface drainage.
- Install perimeter pavement drainage systems (i.e., French drains) surrounding areas anticipated for frequent wetting, such as the depressed loading dock area.
- Install joint sealant and seal cracks immediately.

- Seal all landscaped areas in or adjacent to pavements to reduce moisture migration to subgrade soils.
- Place compacted, low permeability backfill against the exterior side of curb and gutter.
- Construct curb and gutter directly on clay subgrade soils rather than on granular base course materials (or use the deepened "San Antonio-style" curb).

SLOPE STABILITY

Cut Slopes

The table below provides the recommended slope inclinations for both permanent cut slopes and temporary cut slopes. In our opinion, cut slopes at the inclinations discussed below should be stable against a large-scale slide, however the potential for sloughing of loose soils zones exists.

Slope Type	Maximum Slope Inclinations
Temporary	1½(H):1(V) in on-site soils ½(H):1(V) in Stratum 4 limestone
Permanent	3(H):1(V) in on-site soils 1(H):1(V) in Stratum 4 limestone

Exposed cut slopes will also be susceptible to further erosion due to the nature of the on-site soils and limestone. Installation of erosion control measures in such areas would be beneficial in reducing the potential slope stability which could result from excessive erosion. In addition to initial erosion control measures, the cut slopes should be periodically checked for erosion (particularly after heavy rainfall events) and maintenance performed on areas exhibiting erosion.

In regards to worker safety, Occupation Safety and Health Administration (OSHA) Safety and Health Standards require the protection of workers adjacent to excavations. The OSHA guidelines and directives should be adhered by the Contractor during construction to provide a safe working environment.

Buffer Zones Adjacent to Cut Slopes

Excavation methods which fracture the limestone significantly could result in decreased slope stability. To allow for some sloughing to occur, we recommend that a "buffer zone" at least 5 feet wide adjacent to pavement and other general areas be provided between the proposed construction areas and the permanent cut slopes (both at the toe and the crest). If buildings are planned near these areas, the buffer zones should be increased to at least 10 feet. This should help reduce the possibility of sloughing soils/rock from contacting the adjacent improvements on the downhill side and from undermining the improvements on the uphill side.

Embankment Fill Slopes

The table below provides the recommended slope inclinations for embankment fill slopes which are constructed in association with building, pavement, and/or general site improvements.

Slope Type	Maximum Slope Inclinations
Embankment Fill Slopes [1,2]	3(H):1(V)

1. For slopes to be used by mowers or other maintenance equipment, 4H:1V slopes are generally acceptable.
2. Fill placement for the embankments should proceed as outlined in **Earthwork**.

The embankment slopes should be properly protected from erosion. The use of rock rip-rap, erosion control fabrics, and/or vegetation is common). In addition to initial erosion control measures, the embankments should be periodically checked for erosion (particularly after heavy rainfall events) and maintenance performed on areas exhibiting erosion.

Embankments which are constructed on natural subgrade sloping steeper than 5(H):1(V) should be “keyed” into the subgrade at the toe of the embankment. The keyed-in toe should consist of a 12-foot wide section which is excavated into the subgrade such that a horizontal working surface is attained for compaction of the first embankment lift. Successive lifts should remain horizontal and should not tend to follow the slope of the natural subgrade.

The edges of fill embankments are often undercompacted in the field due to loose material being pushed off the edges as the embankment lifts are compacted. To reduce the possibility of this impacting the stability of the embankment fill, the embankments should be overbuilt and compacted as outlined in **Earthwork**. Then the embankment should be cut back to the slopes recommended above.

GENERAL COMMENTS

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Natural variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence or collaboration through this system are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client, and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly impact excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety, and cost estimating including, excavation support, and dewatering requirements/design are the responsibility of others. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.