GEOTECHNICAL ENGINEERING STUDY

FOR

ROADWAY D AT VERMENDI PRECINCT 9 UNIT 2
VERAMENDI MASTER PLANNED DEVELOPMENT
NEW BRAUNFELS, TEXAS

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Project No. ANA26-003-00
February 5, 2026

Mr. Troy Vater
Development Manager
ASA Properties, LLC
2168 Oak Run Parkway, Suite 101
New Braunfels, Texas 78132

RE: Geotechnical Engineering Study
Roadway D at Veramendi Precinct 9 Unit 2
Veramendi Master Planned Development
New Braunfels, Texas

Dear Mr. Vater:

RABA KISTNER Inc. (RKI) is pleased to submit the report of our Geotechnical Engineering Study for the above-referenced project. This study was performed in accordance with RKI Proposal No. PNA25-060-00, dated December 19, 2025. The purpose of this study was to drill a single boring within the proposed new roadway, to perform laboratory testing to classify and characterize subsurface conditions, and to prepare an engineering report presenting pavement design and construction guidelines.

The following report contains our design recommendations and considerations based on our current understanding of the project information provided to our office. There may be alternatives for value engineering of the pavement systems, and RKI recommends that a meeting be held with the Owner and design team to evaluate these alternatives.

We appreciate the opportunity to be of service to you on this project. Should you have any questions about the information presented in this report, or if we may be of additional assistance with value engineering or on the materials testing-quality control program during construction, please call.

Very truly yours,

RABA KISTNER, INC.

STATE OF TEXAS
ISAAC MOLINA
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LICENSED
PROFESSIONAL ENGINEER

Santosh Shrestha, E.I.T.
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SS/IM/alg

Attachments

Copies Submitted: Above (1) – Email Only

GEOTECHNICAL ENGINEERING STUDY

For

ROADWAY D AT VERAMENDI PRECINCT 9 UNIT 2
VERAMENDI MASTER PLANNED DEVELOPMENT
NEW BRAUNFELS, TEXAS

Prepared for

ASA PROPERTIES, LLC
New Braunfels, Texas

Prepared by

RABA KISTNER, INC.
New Braunfels, Texas

PROJECT NO. ANA26-003-00

February 5, 2026

RABAKISTNER

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ATTACHMENTS

The following figures are attached and complete this report:

INTRODUCTION

RABA KISTNER, Inc. (RKI) has completed the authorized subsurface exploration for the proposed Roadway D in the Veramendi Master Planned Development in New Braunfels, Texas, as illustrated on Figure 1. This report briefly describes the procedures utilized during this study and presents our findings along with our recommendations for pavement design and construction considerations.

PROJECT DESCRIPTION

To be considered in this study is approximately 500 LF of Roadway D connecting Loop 337/HWY 46 within Precinct 9 Unit 2 in the Veramendi Master Planned Development in New Braunfels, Texas. The proposed roadway will be designed utilizing guidance from the City of San Antonio's Pavement Design Guidance Manual. We anticipate that the roadway will be designed in accordance with Arterial Street classification.

This study is limited to the pavement design of the proposed roadway. Soil borings to develop recommendations for associated site structures, such as low water crossings, signalization poles, bridges, or retaining walls were not included in the authorized scope of services.

LIMITATIONS

This engineering report has been prepared in accordance with accepted Geotechnical Engineering practices in the region of south/central Texas and for the use of ASA Properties, LLC (CLIENT) and its representatives for design purposes. This report may not contain sufficient information for the purposes of other parties or other uses. This report is not intended for use in determining construction means and methods. The attachments and report text should not be used separately.

The recommendations submitted in this report are based on the data obtained from a single boring, and the information provided to us. If the project information described in this report is incorrect, is altered, or if new information is available, we should be retained to review and modify our recommendations.

This report may not reflect the actual variations of the subsurface conditions across the site. The nature and extent of variations across the site may not become evident until construction commences. The construction process itself may also alter subsurface conditions. If variations appear evident at the time of construction, it may be necessary to reevaluate our recommendations after performing on-site observations and tests to establish the engineering impact of the variations.

The scope of our Geotechnical Engineering Study does not include an environmental assessment of the air, soil, rock, or water conditions either on or adjacent to the site. No environmental opinions are presented in this report.

PREVIOUS STUDY

RKI has previously performed Geotechnical Engineering Study within the existing Roadway D in 2019 (RKI Project No. ANA18-043-00, dated July 2, 2019). The results of this geotechnical engineering study is on file in our office. Our previous data was utilized as supplementary information in the preparation of this report.

BORINGS AND LABORATORY TESTS

Subsurface condition at the site was evaluated by a single boring, hand augured at the location shown on the Boring Location Map, Figure 1. The location is approximate, and distance was measured using a recreational grade, hand-held, GPS Locator. The boring was hand augured to the top of the limestone.

During hand augering, grab samples were collected. Each sample was visually classified in the laboratory by a member of our Geotechnical Engineering staff. The geotechnical engineering properties of the strata were evaluated by the natural moisture content, sulfate content and Atterberg limits tests.

The laboratory test results are presented in graphical or numerical form on the boring log illustrated on Figure 2. A key to classification terms and symbols used on the boring log is presented in Figure 3. The results of the laboratory and field testing are also tabulated in Figure 4 for ease of reference.

Dynamic Cone Penetrometer (DCP) testing was also performed at boring location from the existing ground surface to approximately 2 ft or practical equipment refusal and the results are presented on Figure 5.

Samples will be retained in our laboratory for 30 days after submittal of this report. Other arrangements may be provided at the request of the Client.

SULFATE TESTING

Sulfate testing was performed on the grab sample. The results of the sulfate content test is presented in the table below.

The purpose of the sulfate testing was to determine the concentration of soluble sulfates in the subgrade soils, in order to investigate the potential for an adverse reaction to lime in sulfate-containing soils. The adverse reaction, referred to as sulfate-induced heave, has been known to cause cohesive subgrade soils to swell in short periods of time, resulting in pavement heaving and possible failure. Sulfates can also affect the durability of concrete when encountered in high concentrations.

Soil Type	Boring Number	Approximate Depth Below Existing Ground Surface (ft)	Sulfate Content (ppm)
Dark Brown Clay	P-1	0 – 1	Less than 100

Based on the laboratory test results, the reported sulfate concentration value was generally determined to be negligible. Reported sulfate concentrations above 3,000 ppm are known to cause sulfate induced heaving when the soils are mixed with lime. If the option for lime is considered, a quality assurance program should be implemented to assist in reducing the risk of sulfate induced heaving.

GENERAL SITE CONDITIONS

GEOLOGY

A review of the *Geologic Atlas of Texas, San Antonio Sheet*, indicates that this site is naturally underlain with the soils/rock (limestone) of the Edwards Group. Edwards limestone is generally considered hard in induration and typically contains harder zones/seams of chert and dolomite. Edwards limestone also typically contains karstic features in the form of open and/or clay-filled vugs, voids, and/or solution cavities that form as a result of solution movement through fractures in the rock mass.

Key geotechnical engineering considerations for development supported on this formation will be the depth to rock, the expansive nature of the overlying clays, the condition of the rock, and the presence/absence of karstic features.

STRATIGRAPHY

The soil boring was hand augured at the location indicated on the Boring Location Map (Figure 1). The soil descriptions and approximate thicknesses of each layer observed at the boring location are summarized below:

Stratum	Approximate depths, ft	General Soil Description	Notes
1	0 to 1	Dark Brown Clay	• Plastic • Moderate swell/shrink potential
2	1 to 1.5	Tan and Gray Limestone	• With chert seams • less weathered with depth

Observations during drilling indicate the limestone became less weathered with depth. Excavations that extend into the limestone will require heavy-duty excavation equipment; i.e., rock saws, milling machines, hoe rams, or other suitable equipment capable of ripping limestone.

Each stratum presented on the boring log has been designated by grouping materials that possess similar physical and engineering characteristics. The boring log should be consulted for more specific stratigraphic information. Unless noted on the boring log, the lines designating the changes between various strata represent approximate boundaries. The transition between materials may be gradual or may occur between recovered samples. The stratification presented on the boring log, or described herein, is for use by RKI in its analyses and should not be used as the basis of design or construction cost estimates without realizing there can be variation from that shown or described.

The boring log and related information depict subsurface conditions only at the specific locations and times where sampling was conducted. The passage of time may result in changes in conditions, interpreted to exist, at or between the locations where sampling was conducted.

GROUNDWATER

Groundwater was not observed in the boring either during or immediately upon completion of the hand augering operations. However, it is possible for groundwater to exist beneath this site at shallow depths on a transient basis, particularly following periods of precipitation. Fluctuations in groundwater levels occur due to variation in rainfall and surface water run-off. The construction process itself may also cause variations in the groundwater level.

Bedrock occurs near the existing ground surface. Based on our experience, perched groundwater has a tendency to collect and flow along the soil/rock interface or fractures within the rock mass itself following extended wet periods.

SWELL/HEAVE POTENTIAL

Based on the results of our laboratory testing and findings from our test boring, the estimated Potential Vertical Rise (PVR) for this site is 1 in. or less. This vale was estimated using the empirical procedure, Texas Department of Transportation (TxDOT) Tex-124-E, Method for Determining the Potential Vertical Rise (PVR). A surcharge load of 1 psi, an active zone to the top of the bedrock and dry moisture conditions were assumed in estimating the above PVR values.

Subgrade soils that are expansive when water is introduced (i.e. the surficial dark brown clay at this site) will heave, causing the pavement to become rough or uneven over time. Pavement roughness is generally defined as an expression of irregularities in the pavement surface that adversely affect the ride quality of a vehicle (and thus the user). Roughness is an important pavement characteristic because it affects not only ride quality but also vehicle costs, fuel consumption, and maintenance costs. Pavement heave can be reduced through various measures but cannot be totally eliminated without full removal of the problematic soil. Measures available for reducing heave include:

- Soil Treatment with Lime or Other Chemicals;
- Removal and Replacement of High PI Soils; and
- Drains or Barriers to Collect or Inhibit Moisture Infiltration.

Soil treatment with lime (or other chemicals) is typically used to reduce the swelling potential of the upper portion of the pavement subgrade containing highly plastic soils. Lime and water are mixed with the top 6 to 12 inches (or possibly more) of the subgrade and allowed to mellow or cure for a period of time. After mellowing the soil-lime mixture is compacted to form a strong soil matrix that can improve pavement performance and potentially reduce soil heave. However, in highly plastic soils, lime treatment of only the top portion of the expansive subgrade may not provide an acceptable reduction in PVR. For a more substantial reduction in PVR, removal and replacement of the high PI soil may be the only method available to reduce the potential vertical rise of the pavement to an acceptable level. As stated previously, it must be recognized that partial removal of expansive clay soil only reduces the potential (or risk) of the damage swell can cause to a pavement and does not completely eliminate this risk.

In addition, capturing water infiltration via French drains, pavement edge drains, or inhibiting water through the use of vertical moisture barriers would reduce the potential for heave since one important component of the heaving mechanism, water, would be reduced. Geocomposite membranes, like geogrids, are also another tool available that may help reduce the damage that heaving subgrades cause to flexible pavements and may be considered in addition to or as an alternative to other mitigation techniques.

It should be noted that the pavement sections derived in the following sections are structurally adequate for the given traffic levels and existing clay subgrade strength, but do not consider the long-term effects of pavement roughness due to heave, which can only be addressed by the measures discussed in this section.

PAVEMENT RECOMMENDATIONS

Recommendations for both flexible and rigid pavements are presented in this report. The Owner and/or design team may select either pavement type depending on the performance criteria established for the project. In general, flexible pavement systems have a lower initial construction cost as compared to rigid pavements. However, maintenance requirements over the life of the pavement are typically much greater for flexible pavements. This typically requires regularly scheduled observation and repair, as well as overlays and/or other pavement rehabilitation at approximately one-half to two-thirds of the design life. Rigid pavements are generally more "forgiving" and therefore tend to be more durable and require less maintenance after construction.

For either pavement type, drainage conditions will have a significant impact on long term performance, particularly where permeable base materials are utilized in the pavement section. Drainage considerations are discussed in more detail in a subsequent section of this report.

SUBGRADE STRENGTH CHARACTERIZATION

In our single boring we encountered about 1 ft of clay over limestone bedrock. The final site grading has not been determined. This study evaluated the California Bearing Ratio (CBR) of the near-surface clays to review the subgrade support conditions provided by the overburden clays.

Based on the DCP test results and our previous experience with the soils in this area, we have assumed a design CBR value of 6.5 for use in our pavement section analysis for the clay fill subgrade (hereafter referred to as the ‘clay subgrade’). If clay soils are imported for the purpose of constructing the roadbed, then imported materials must be selected that have a CBR value of at least 6.5. If lower quality clay fill materials are utilized, the pavement sections will have to be increased based on the quality (tested CBR value) of the clays imported.

A ‘clay subgrade’ condition with a CBR of 6.5 may be utilized where on-site clay is used as subgrade. Cuts that extend below 1 ft, depending on the different locations, will encounter rock subgrade. A ‘rock subgrade’ condition with a CBR of 10.0 may be utilized for the following conditions:

- If select fill material, in accordance with the *Select Fill* section of this report is utilized as the subgrade fill up to the bottom of the pavement section elevation from rock subgrade;
- If native, intact limestone is exposed prior to select fill placement (if necessary); or
- If the near surface on-site clays are removed and replaced with rock millings generated from site grading.

For areas that transition between clay and rock subgrade, we recommend that geogrid be utilized to relieve stress concentrations at the subgrade transitions. The geogrid should be used as a transition for 5 ft or greater on either side of the transition.

DESIGN PARAMETERS – ASPHALT CONCRETE PAVEMENTS

The roadway to be considered in this study is roadway D at Precinct 9 Unit 2 of the Veramendi Master Planned Development in New Braunfels, Texas. The proposed design was evaluated using the *City of San Antonio's Design Guidance Manual* recommended values for Arterial Street classification. The City of San Antonio (CoSA) design guide provides the following input design parameters for use in the AASHTO design method.

Street Classification	Equivalent 18-kip Single Axle Load Applications (ESALs)	Reliability	Serviceability Initial/Terminal	Standard Deviation	Structural Number Minimum/Maximum
Arterial	3,000,000	95	4.2/2.5	0.45	3.80/5.76

The required structural number is related to the CBR value of the pavement subgrade and the amount of traffic that the pavement will carry over its service life. CBR provides an estimate of the relative strength of the subgrade and consequently indicates the ability of the pavement section to carry load. This site specific CBR value is utilized in conjunction with the above specified parameters to determine the required Structural Number (SN) for use in the design of the pavement section.

To determine the required design SN value, we utilized a method based on the 1993 edition of the AASHTO "Guide for the Design of Pavement Structures." The "required by design" SN values are presented in the tables of the pavement sections as well as the values subsequently determined in the design of the pavement sections for this site.

STRUCTURAL NUMBER RECOMMENDATIONS

Structural numbers for each street classification and each subgrade condition were calculated using the parameters provided in the table presented in the previous section. The resulting Structural Numbers are presented in the recommended pavement section tables.

PAVEMENT DESIGN PARAMETERS – ASPHALT CONCRETE PAVEMENTS

The following input variables are utilized to design flexible base pavements (commonly referred to as Asphaltic Cement Concrete or Asphalt pavements) when using the procedures detailed in the *1993 AASHTO Guide for Design of Pavement Structures*:

- Performance Period, years
- Roadbed Soil Resilient Modulus, psi
- Serviceability Indices
- Overall Standard Deviation

- Reliability, %
- Design Traffic, 18-kip ESALs

Performance Period, years

The pavement structure was designed for a 20-year performance period which is typical for most flexible pavements.

Roadbed Soil Resilient Modulus, psi

The Resilient Modulus (M_R) is the material property used to characterize the support characteristics of the roadbed soils in flexible pavement design. It is a measure of the soil's deformation response to cyclic applications of loads much smaller than a failure load.

To determine the resilient modulus (M_r) of the subgrade, we utilized the correlation equation shown below:

M_r = 1,500 x CBR

Serviceability Indices

Initial serviceability is a measure of the pavement's smoothness or rideability immediately after construction. Terminal serviceability is the minimum tolerable serviceability of a pavement. When the serviceability of a pavement reaches its terminal value, rehabilitation is required. See the recommended Initial and Terminal Serviceability Indices on the table presented in the *Design Parameters –Asphalt Concrete Pavements* section of this report.

Overall Standard Deviation

Overall standard deviation accounts for both chance variation in the traffic prediction and normal variation in pavement performance prediction for a given traffic. Higher values represent more variability; thus, the pavement thickness increases with higher overall standard deviations. A value of 0.45 was utilized for the flexible pavement designs presented herein.

Reliability, %

Reliability value represents a "safety factor," with higher reliabilities representing pavement structures with less chance of failure. The AASHTO Guide recommends values ranging from 50 to 99.9%, depending on the functional classification and the location (urban vs. rural) of the roadway. See the recommended Reliability values on the table presented in the *Design Parameters – Asphalt Concrete Pavements* section of this report.

Design Traffic, 18-kip ESALs

The 18-kip ESALs were determined from the traffic data specified in the Unified Development Code for the City of San Antonio. See the recommended values on the table presented in the *Design Parameters – Asphalt Concrete Pavements* section of this report.

RECOMMENDED PAVEMENT SECTIONS – ASPHALT CONCRETE PAVEMENTS

If on-site clay fill is utilized for fill grading, it should be placed and compacted as discussed in the *On-Site Clay Fill* section of this report. If the onsite dark brown clays are used as general fill to increase elevations, the final 6 in. of fill should be treated (see *Treatment of Subgrade* section). If fill grading is not planned and clays remain in place, then the dark brown clay should be stripped to the limestone elevation (where possible), the subgrade should be prepared as described in the *Site Preparation* section of this report, and the rock subgrade pavement sections should be utilized.

If fill grading is completed utilizing select fill in accordance with the *Select Fill* section of this report, or if native, intact rock is exposed prior to fill placement (if necessary), the lime treated subgrade may be eliminated from the pavement section and the rock subgrade recommendations should be utilized. Per Appendix 10-A of the *City of San Antonio's Design Guidance Manual*, a rock credit can be given to those pavement sections overlying a rock subgrade. The rock credit provided should be equivalent to a 6 in. structural layer for stabilized subgrade.

For this site, the following options for pavement sections are available for the clay and rock subgrades described herein. Additional options are also available and can be provided upon request.

Roadway D

Clay Subgrade

Arterial; CBR=6.5; Required SN=3.99	Layer Description	Layer Thickness	Recommended SN Coeff.	S.N. Extension
	HMA Type D Surface Course	2.0 in.	0.44	0.88
	HMA Type C/D Surface or Binder Course	2.0 in.	0.44	0.88
	Flexible (Granular) Base	13.0 in.	0.14	1.82
Flexible Base	Treated Subgrade	6.0 in.	0.08	0.48
Option	**Combined Total**	**23.0 in.**		**4.06**
	HMA Type D Surface Course	3.0 in.	0.44	1.32
	HMA Type B Base	7.0 in.	0.34	2.38
Full Depth Asphalt	Treated Subgrade	6.0 in.	0.08	0.48
Option	**Combined Total**	**16.0 in.**		**4.18**
	HMA Type D Surface Course	2.0 in.	0.44	0.88
	HMA Type C/D Surface or Binder Course	2.0 in.	0.44	0.88
Mechanically Stabilized Layer	Mechanically Stabilized Layer	11.0 in.	0.17	1.87
(Geogrid)	Treated Subgrade	6.0 in.	0.08	0.48
Option	**Combined Total**	**21.0 in.**		**4.11**

Rock Subgrade

Arterial; CBR=10.0; Required SN = 3.80[1]	Layer Description	Layer Thickness	Recommended SN Coeff.	S.N. Extension
	HMA Type D Surface Course	2.0 in.	0.44	0.88
	HMA Type C/D Surface or Binder Course	2.0 in.	0.44	0.88
	Flexible (Granular) Base	12.0 in.	0.14	1.68
Flexible Base	Rock Credit	0.0 in.	--	0.48
Option 1	**Combined Total**	**16.0 in.**		**3.92**
	HMA Type D Surface Course	3.0 in.	0.44	1.32
	Flexible (Granular) Base	15.0 in.	0.14	2.10
Flexible Base	Rock Credit	0.0 in.	--	0.48
Option 2 [2]	**Combined Total**	**18.0 in.**		**3.90**
	HMA Type D Surface Course	3.0 in.	0.44	1.32
	Type B Base	6.0 in.	0.34	2.04
Full Depth Asphalt	Rock Credit	0.0 in.	--	0.48
Option	**Combined Total**	**9.0 in.**		**3.84**
	HMA Type D Surface Course	2.0 in.	0.44	0.88
	HMA Type C/D Surface or Binder Course	2.0 in.	0.44	0.88
	Mechanically Stabilized Layer	10.0 in.	0.17	1.70
Mechanically Stabilized Layer	Rock Credit	0.0 in.	--	0.48
Option	**Combined Total**	**14.0 in.**		**3.94**

(1) The calculated Structural Number (SN) was less than the COSA minimum SN, and the COSA minimum was utilized in design.
(2) Flexible base option to match the minimum section recommended by City of New Braunfels for Collector Streets.

The full-depth asphalt option results in a more rigid pavement section and should be carefully considered by the design team before including along the alignments. More rigid pavement sections have a higher likelihood of tensile cracking due to the potential for expansive soils heaving and creating isolated areas of stress concentrations. The lime treated subgrade layer will assist in reducing the potential for expansive soil related movements, but will not eliminate the potential, as discussed previously.

A Mechanically Stabilized Layer (MSL) is a composite layer consisting of flexible (granular) base and a geogrid product. Geogrid provides lateral restraint to the flexible base by confining aggregate particles within the plane of the geogrid, thereby creating a reinforced, or mechanically stabilized layer.

DESIGN PARAMETERS – PORTLAND CEMENT CONCRETE PAVEMENTS

The roadway to be considered in this study is roadway D at Precinct 9 Unit 2 of the Veramendi Master Planned Development in New Braunfels, Texas. The proposed design was evaluated using the *City of San Antonio's Design Guidance Manual* recommended values for Arterial Street classification. Based on information provided by the City of San Antonio, we understand that the following design parameters are required for use in the design of rigid pavements for these types of streets.

Street Classification	Equivalent 18-kip Single Axle Load Applications (ESALs)	Reliability	Serviceability (Initial/Terminal)	Standard Deviation	Rigid Pavement Slab Thickness (Minimum/Maximum)
Arterial	4,500,000	95	4.5/2.5	0.35	9.0/13.0

To calculate the required design rigid pavement thickness, we utilized a method based on the 1993 edition of the AASHTO "Guide for the Design of Pavement Structures."

PAVEMENT DESIGN PARAMETERS – PORTLAND CEMENT CONCRETE PAVEMENTS

The following input variables are utilized to design rigid pavements (commonly referred to as Portland Cement Concrete or PCC pavements) when using the procedures detailed in the *1993 AASHTO Guide for Design of Pavement Structures*:

- Performance Period
- 28-day Concrete Modulus of Rupture, psi
- 28-day Concrete Elastic Modulus, (M_r) psi
- Effective Modulus of Subbase/Subgrade Reaction, (k-value) psi/in.
- Serviceability Indices
- Load Transfer Coefficient
- Drainage Coefficient
- Overall Standard Deviation
- Reliability, %
- Design Traffic, 18-kip ESALs

Performance Period

The pavement structure was designed for a 30-year performance period which is typical for most rigid pavements.

28-day Concrete Modulus of Rupture (M_r), psi

The M_r of concrete is a measure of the flexural strength of the concrete as determined by breaking concrete beam test specimens. An M_r of approximately 600 psi at 28 days was used in the analysis and is typical of local concrete production.

28-day Concrete Elastic Modulus, psi

Elastic modulus of concrete is an indication of concrete stiffness and varies depending on the coarse aggregate type used in the concrete. A modulus of 4,000,000 psi is used for this pavement design.

Effective Modulus of Subgrade Reaction(k-value), psi/in.

Concrete slab support is characterized by the modulus of subgrade/subbase reaction, otherwise known as the k-value, with units typically shown as psi/in. Based on the CBR values of 6.5 and 10.0 for clay and rock subgrade, as discussed above, k-values of 130 and 160 psi/in., respectively, were used in the rigid pavement design procedure. Rigid concrete pavements are not highly sensitive to the k-value.

Serviceability Indices

Initial serviceability is a measure of the pavement's smoothness or rideability immediately after construction. Terminal serviceability is the minimum tolerable serviceability of a pavement. When the serviceability of a pavement reaches its terminal value, rehabilitation is required. See the recommended Initial and Terminal Serviceability Indices on the table presented in the *Design Parameters – Portland Cement Concrete Pavements* section of this report.

Load Transfer Coefficient

The load transfer coefficient is used to incorporate the effect of dowels, reinforcing steel, tied shoulders, and tied curb and gutter on reducing the stress in the concrete slab due to traffic loading and therefore causing a reduction in the required concrete slab thickness.

The load transfer coefficient used in this pavement design is 3.2 for pavements designed with load transfer devices (i.e. dowels) at control joints or CRCP.

Drainage Coefficient

The drainage coefficient characterizes the quality of drainage of the subbase layers under the concrete slab. Good draining pavement structures do not give water the chance to saturate the subbase and subgrade; thus, pumping is not as likely to occur. A drainage coefficient of 1.01 is utilized for rigid pavement design.

Overall Standard Deviation

Overall standard deviation accounts for both chance variation in the traffic prediction and normal variation in pavement performance prediction for a given traffic. Higher values represent more variability; thus, the pavement thickness increases with higher overall standard deviations. See the recommended Overall Standard Deviation on the table presented in the *Design Parameters – Portland Cement Concrete Pavements* section of this report.

Reliability, %

Reliability value represents a "safety factor," with higher reliabilities representing pavement structures with less chance of failure. The AASHTO Guide recommends values ranging from 50 to 99.9%, depending on the functional classification and the location (urban vs. rural) of the roadway. See the recommended Reliability on the table presented in the *Design Parameters – Portland Cement Concrete Pavements* section of this report.

Design Traffic 18-kip ESAL

The 18-kip ESALs were determined from the street classifications as discussed previously in the *Design Parameters – Portland Cement Concrete Pavements* section of this report.

RECOMMENDED PAVEMENT SECTIONS – PORTLAND CEMENT CONCRETE PAVEMENTS

The recommended concrete slab thicknesses determined with the inputs discussed above are presented in the table below. Please note that the concrete slab thickness is not very sensitive to the subgrade strength value (k-value) and the recommended pavement thickness in the table below is appropriate for both the clay and rock subgrade conditions.

Portland Cement Concrete Design - Cross Sections	Layer Description	TxDOT Spec. Item	Layer Thickness
Arterial **(to be used for both clay and rock subgrade conditions)**	PCC Surface Subbase **Combined Total**	360 ---- 	10.0 in. 0.0 in. 10.0 in.

PAVEMENT CONSTRUCTION CONSIDERATIONS

SUBGRADE PREPARATION

Preparation for the right-of-way (for streets, sidewalks, utilities, etc.) should be performed in accordance with the TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges 2024, Item 100 – *Preparing Right of Way*. Exposed subgrades should be thoroughly proofrolled in order to locate any weak, compressible zones. A minimum of 5 passes of a fully loaded dump truck or a similar heavily-loaded piece of construction equipment should be used for planning purposes. Proofrolling operations should be observed by the Geotechnical Engineer or his representative to document subgrade condition and preparation. Weak or soft areas identified during proofrolling should be removed and replaced with a suitable, compacted backfill.

In areas where the clay subgrade will remain in place, the exposed subgrade should be moisture conditioned. This should be done after completion of the proofrolling operations and just prior to flexible base placement. Moisture conditioning is done by scarifying to a minimum depth of 8 in. and recompacting to a minimum of 95 percent of the maximum density determined from the Texas Department of Transportation Compaction Test (TxDOT, Tex-114-E). The moisture content of the subgrade should be maintained within the range of optimum moisture content to 3 percentage points above optimum until permanently covered.

Upon completion of fill grading using the on-site clays, the final 6 in. of fill should be lime treated (see *Lime Treatment of Subgrade*). If fill grading is not planned, then lime treatment of the stripped clay subgrade should be performed in conjunction with the scarifying, moisture conditioning, and recompaction described previously.

If fill grading is completed utilizing select fill above limestone bedrock in accordance with the *Select Fill* section of this report or if native, intact rock is exposed prior to flexible base placement, the lime treated subgrade may be eliminated from the pavement section and the rock subgrade recommendations should be utilized.

PAVEMENT SUBGRADE SELECT FILL

Detailed site grading information was not available at the time of this study. If needed, select fill subgrade can be used to raise grades beneath the proposed roadway section to provide improved subgrade support. Select fill should be crushed stone or gravel aggregate. We recommend that materials specified for use as select fill meet the TxDOT Standard Specifications 2024, Item 247 – *Flexible Base*, Type A, Grade 1-2.

Soils classified as CH, MH, ML, SM, GM, OH, OL and Pt under the USCS are **not** considered suitable for use as select fill materials at this site. The native soils at this site are **not** considered suitable for use as select fill materials.

Select fill should be placed in loose lifts not exceeding 8 in. in thickness and compacted to at least 100 percent of maximum density as determined by TxDOT, Tex-113-E, Compaction Test. The moisture content of the fill should be maintained within the range of 2 percentage points below to 2 percentage points above the optimum moisture content until final compaction.

If select fill is placed over moisture conditioned clays, the first lift of select fill may be placed at 95 percent of the maximum density as determined by TxDOT, Tex 113-E, Compaction Test.

If excavations extend to significant depths into the limestone formation, consideration can be given to utilizing the excavated limestone as "alternative select fill". However, processing of the excavated material will be required to reduce the maximum particle size to 4 in. Furthermore, special care will be required during excavation activities to separate organics and any plastic clay seams encountered. Alternative select fill materials shall have a maximum liquid limit not exceeding 40 and a plasticity index between 7 and 20. In addition, if these materials are utilized, grain size analyses and Atterberg Limits must be performed during placement at a rate of one test each per 5,000 cubic yards of material due to the high degree of variability associated with on-site processed materials. If on-site materials cannot be processed to meet the required criteria, imported select fill materials should be utilized.

If the above-listed alternative materials are being considered for bidding purposes, the materials should be submitted to the Geotechnical Engineer for pre-approval at a minimum of 10 working days or more prior to the bid date. Failure to do so will be the responsibility of the contractor. The contractor will also be responsible for ensuring that the properties of all delivered alternate select fill materials are similar to those of the pre-approved submittal. It should also be noted that when using alternative fill materials, difficulties may be experienced with respect to moisture control during and subsequent to fill placement, as well as with erosion, particularly when exposed to inclement weather. This may result in sloughing of beam trenches and/or pumping of the fill materials.

ON-SITE CLAY FILL

We recommend that the on-site soils be placed to conform to the TxDOT Standard Specifications 2024, Item 132 – *Embankment*, Type B, and should be placed in compacted lifts not exceeding 6 in. in thickness and compacted to the requirements of Table 2 in Item 132 based on the maximum density and optimum moisture content as determined by TxDOT, Tex-114-E or ASTM D698. The moisture content of the fill should be maintained to be at least equal to the optimum water content but not exceed 3 percentage points above the optimum water content until permanently covered. Fill materials shall be free of roots and other organic or degradable material. We recommend that the maximum particle size not exceed 4 in. or one half the compacted lift thickness, whichever is smaller. If other import fill materials are utilized, RKI should be notified, as CBR testing and thicker pavement sections may be required.

It is imperative that the subgrade modulus utilized in the pavement design process be met or exceeded by the fill material. In the event that the clay fill used is different than the existing subgrade, the recommendations in this report could be invalidated and the design engineer must be consulted to determine if additional CBR testing and thicker pavement sections are required.

TREATMENT OF SUBGRADE

Lime Treatment

Lime of the subgrade soils, if utilized, should be in accordance with the TxDOT Standard Specifications 2024, Item 260 - Lime *Treatment*. A sufficient quantity of hydrated lime should be mixed with the subgrade soils to reduce the soil plasticity index to 20 or less. For estimating purposes, we recommend that at least 4 percent hydrated lime by weight be used to reduce the PI of the subgrade clays or a minimum of 22 pounds/S.Y. for 6 in. of lime treated subgrade, whichever results in the greater percentage of lime. An additional 1 percent should be added for dry placement to account for loss. **For construction purposes, we recommend that the optimum lime content of the subgrade soils be determined by laboratory testing with representative samples of the subgrade materials being used for this project.**

Treated subgrade soils should be compacted to a minimum of 95 percent of the maximum density at a moisture content within the range of optimum moisture content to 3 percentage points above the optimum moisture content as determined by Tex-113-E.

Cement Treatment

Cement treatment of the subgrade soils, if utilized, should be in accordance with the TxDOT Standard Specifications 2024, Item 275 – *Cement Treatment*. A sufficient quantity of cement should be mixed with the subgrade soils to provide a compressive strength of 150 psi. **For construction purposes, we recommend that the cement content of the subgrade soils be determined by laboratory testing with representative samples of the subgrade materials being used for this project.**

Treated subgrade soils should be compacted to a minimum of 95 percent of the maximum density at a moisture content within the range of optimum moisture content to 3 percentage points above the optimum moisture content as determined by Tex-113-E.

The moisture content of finished cement-treated subgrade soils, if any, should be maintained for a period of 24–48 hr. During this time, but no sooner than 24 hours, roll the finished course using a vibratory roller to induce microcracking. The vibratory roller must be in accordance with TxDOT Standard Specifications 2024, Item 210 - *Rolling*, with a static weight equal to or more than 12 tons, and the vibratory drum must be no less than 20 in. wide. The roller must travel at a speed of 2 mph, vibrating at maximum amplitude, and make two–four passes with 100% coverage excluding the outside 1 ft. of the surface crown, unless otherwise directed by the Engineer. Additional passes may be required to achieve the desired crack pattern as directed. Notify the Engineer 24 hours before the microcracking begins. Cement treated subgrade soils may not produce a cracking pattern during initial vibratory rolling and additional passes of the vibratory roller should be completed at the engineer's discretion.

We recommend that during site grading operations, additional laboratory testing be performed to determine the concentration of soluble sulfates in the subgrade soils. If the sulfate contents increase to 3,000 ppm or more, the sulfate in the soil may react with calcium-based stabilizers potentially causing sulfate-induced heave and the use of lime or cement should be reconsidered.

GEOGRID REINFORCEMENT

The geogrid reinforcement should be selected and placed in accordance with TxDOT Standard Specifications 2024, Item 250 – *Geogrid Base Reinforcement*, using a Type II TxDOT approved geogrid that conforms to DMS 6240. The geogrid should be placed at the bottom of the flexible (granular) base section in all flexible pavement cases. An alternative to the above geogrid should not be considered without approval from RKI. **In our opinion, incorporating geogrid into the flexible pavement sections will enhance overall pavement performance and reduce the potential for cracking and maintenance in asphalt pavements.**

FLEXIBLE BASE COURSE

The flexible base course should be crushed limestone conforming to the TxDOT Standard Specifications 2024, Item 247 – *Flexible Base*, Type A, Grades 1-2. The base course should be placed in lifts with a maximum compacted thickness of 8 in. (10 inches loose) and compacted to a minimum of 95 percent of the maximum density determined by Tex-113-E at a moisture content within the range of 2 percentage points below to 2 percentage points above the optimum moisture content as determined by Tex-113-E.

PRIME COAT

A prime coat should be placed on top of the flexible base course (if used) and should be a MC-30, AE-P, EAP&T, or PCE conforming to the TxDOT Standard Specifications 2024, Item 310 – *Prime Coat* or Item 314 – *Emulsified Asphalt Treatment* as well as TxDOT Item 300 – *Asphalts, Oils and Emulsions*. Prime coat application rates are typically between 0.1 to 0.3 gal/yd^2 and are generally dependent upon the absorption rate of the granular base and other environmental conditions at the time of placement. The prime coat layer should be placed on the prepared flexible base as soon as possible. This will facilitate plugging the capillary voids in the flexible base surface to reduce migration of moisture and providing a water-resistant surface. The asphalt layer should be placed as soon as possible after the prime coat has been properly set/cured.

TACK COAT

A tack coat should be placed between asphaltic concrete base and/or surface lifts and should be SS-1H, CSS-1H, EAP&T, or a PG binder with a minimum high-temperature grade of PG 58 conforming to TxDOT Standard Specification 2024, Item 341, para 2.5 – *Tack Coat*.

ASPHALTIC CONCRETE SURFACE AND/OR BINDER[1] COURSES

The asphaltic concrete surface and/or binder courses should conform to the TxDOT Standard Specifications 2024, Item 341 – *Dense Graded Hot-Mix Asphalt or Item 341 Paragraph 2.6.2 Warm Mix Asphalt (WMA)*, Types C or D for the surface and binder, and Type B for the base, if the full depth asphalt is selected for construction. Recycled asphalt pavement (RAP) should be limited to 20 percent of the total weight of the mix for Types C and D mixes and 30 percent for Type B mixes. Higher percentages of RAP may be permissible depending on the material source. If higher percentages of RAP are desired, contact RKI for consideration. Asphalt cement grades should conform to the table shown below, which conforms to the requirements of Item 341.

Street Classifications	Minimum PG Asphalt Cement Grade		
	Surface Courses	Binder & Level Up Courses	Base Courses
Arterials	PG 76-22	PG 70-22	PG 64-22

The asphaltic concrete should be compacted on the roadway to contain from 3 to 8 percent air voids computed using the maximum theoretical specific gravity (Rice) of the mixture determined according to Test Method Tex-227-F. Pavement specimens, which shall be either cores or sections of asphaltic pavement, will be tested according to Test Method Tex-207-F. The nuclear-density gauge or other methods which correlate satisfactorily with results obtained from project roadway specimens may be used when approved by the Engineer. Unless otherwise shown on the plans, the Contractor shall be responsible for obtaining the required roadway specimens at their expense and in a manner and at locations selected by the Engineer.

It is recommended that the asphalt concrete pavement be placed with a paving machine only and not with a motor grader unless prior approval is granted by the Engineer for special circumstances. The asphalt layer should preferably be placed as soon as possible after the flexible base has been accepted and the prime coat has been placed. This will further protect the flexible base and subgrade from undue moisture fluctuation due to precipitation or sheet flow from rain events.

PORTLAND CEMENT CONCRETE

The Portland cement concrete should be in accordance with Class P concrete of the TxDOT Standard Specifications 2024, Item 421 – Hydraulic Cement Concrete. Requirements include concrete designed to meet a minimum average compressive strength of 3,200 psi at 7-days or a minimum average compressive strength of 4,000 psi at 28-days in accordance with TxDOT standard laboratory test procedure Tex-448-A or Tex-418-A. Liquid membrane-forming curing compound should be applied as soon as practical after broom finishing the concrete surface. The curing compound will help reduce the loss of water from the concrete. The reduction in the rapid loss in water will help reduce shrinkage cracking of the concrete.

[1] A binder course is defined as the hot mixed asphalt concrete (HMAC) layer placed directly beneath the ACP surface or wearing course but is not an asphalt treated base layer.

CONCRETE PAVEMENT CONSTRUCTION CONTROL

Construction of Portland Cement Concrete Pavements should be controlled by the TxDOT Standard Specifications 2024, Item 360 – *Concrete Pavement*. The surface of all concrete pavements should be textured or tined. Texturing using carpet dragging or tining should be in accordance with Item 360, Sections 3.5.1 and 3.5.2. Other texturing techniques may be utilized as described in ACI 330.1-03, Section 3, Subparagraph 9.

CONCRETE PAVEMENT TYPE

Jointed Plain Concrete Pavement (which is referred to by TxDOT as Concrete Pavement Contraction Design or CPCD) is suggested for roadways with crosswalks, adjacent parking, or sidewalks and is recommended as the pavement type for this city street.

JOINT SPACING AND DETAILS

There are three current options for rigid pavements: 1) undoweled jointed, 2) doweled jointed, and 3) continuously reinforced concrete pavements. Continuously reinforced concretes are generally prohibitively expensive and best suited for major highways; these were not considered as part of our design approach. The following recommendations are based on ACI 330R-08 “Guide for the Design and Construction of Concrete Parking Lots.”

Joint Layout Plan: A joint layout plan should be prepared by a competent individual(s) prior to concrete pavement installation. The recommended joint spacing is 24 times the thickness of the slab up to a maximum of 12 ft in either the longitudinal or transverse direction. The length of a slab or panel should not be more than 25% greater than its width. We recommend avoiding jointing lines which create angles of less than 60 degrees, “T” joints, and interior corners. Proper curing of the concrete pavement should be initiated immediately after finishing. All control joints should be formed or sawed to a depth of at least 1/4 the thickness of the concrete slab and should extend completely through monolithic curbs (if used). Sawing of control joints should begin as soon as the concrete will not ravel, preferably within 1 to 3 hours using an early entry saw or 4 to 8 hours with a conventional saw. Timing will be dictated by site conditions. The width of the joint will be a function of the sealant chosen to seal the joint. It is recommended that a joint seal be utilized to minimize the introduction of incompressible material into the joint.

Concrete structures must be isolated from each other by isolation joints. Pavements should be isolated from structures such as light poles, manholes and other utilities, and buildings. Isolation joints are also used at “T” intersection to accommodate differential movement along different axes. Premolded joint fillers should be used around the fixed structures prior to placing the concrete pavement to prevent bonding of the slab to the structure and should extend through the depth of the slab but slightly recessed from the pavement surface to provide room for the joint sealant. Pavement reinforcement, if used on the project, and dowels should NOT cross isolation joints.

Pavement Cross Joints: Cross joint support is typically provided by one of two mechanisms: smooth steel dowels or plates or, secondly, aggregate interlock. Dowels can contribute to significant costs but provide better cross joint support than aggregate interlock. Relying on aggregate interlock for conventional concretes is not recommended for pavement thicknesses over 7 in. If a doweled option is selected, please see ACI PRC-330-08 for additional information on sizing and spacing of dowels.

Edge Support: Edge support is a significant input to rigid pavement design. While not universally defined, edge support generally refers to shoulders, or other space that separate the edge of the pavement structure from loads. As lots, parking lots, yards, warehouse lots, etc., rarely experience loading at the very edge of the pavement, our analysis was performed with the assumption of edge support. Thus, geometrically, loads should be maintained at least two feet or a minimum of two times the thickness of the pavement section away from the edge of the pavement, whichever is greater. Pavement edges should be supported from movement, either by confining curbs, shoulders, sacrificial paving panels or sidewalks, or other method.

Non-Structural Reinforcement: Concrete sidewalks, flatwork, and pavements may be non-structurally reinforced. This distributed reinforcement is usually placed approximately 2 inches below the top of the concrete surface and as indicated, is non-structural. The sole function of the distributed steel reinforcement is to hold together the fracture faces if cracks form (ACI 330R). Proper jointing is necessary with or without non-structural reinforcement.

As a minimum, the non-structural reinforcement may consist of:

- Bar mat (No. 3 reinforcing bars spaced 18 inches on center in both directions) or
- Welded wire mats (6 x 6 inches, W4.0 x W4.0).

Tie Bars: Per ACI 330R, tie bars should be used to tie only the first longitudinal joint from the pavement edge to keep the outside slab from separating from the pavement.

SUGGESTED PAVEMENT DETAILS

Suggested details that can be utilized for construction are:

- TxDOT CPCD-24, Concrete Pavement Details, Contraction Design, T-6 to 12 inches; and
- TxDOT JS-14, Concrete Paving Details, Joint Seals.

See Figure 6 of the Attachments for the above pavement details.

MISCELLANEOUS PAVEMENT RELATED CONSIDERATIONS

Drainage Considerations

As with any soil-supported structure, the satisfactory performance of a pavement system is contingent on the provision of adequate surface and subsurface drainage. Insufficient drainage which allows saturation of the pavement subgrade and/or the supporting granular pavement materials will greatly reduce the performance and service life of the pavement systems.

Surface and subsurface drainage considerations crucial to the performance of pavements at this site include (but are not limited to) the following:

- Any known natural or man-made subsurface seepage at the site which may occur at sufficiently shallow depths as to influence moisture contents within the subgrade should be intercepted by drainage ditches or below grade French drains.
- Final site grading should eliminate isolated depressions adjacent to curbs, which may allow surface water to pond and infiltrate into the underlying soils. Curbs should be installed to a sufficient depth to reduce infiltration of water beneath the curbs and into the pavement base materials.
- Pavement surfaces should be maintained to help minimize surface ponding and to provide rapid sealing of any developing cracks. These measures will help reduce infiltration of surface water downward through the pavement section.

Utilities

Our experience indicates that significant settlement of backfill can occur in utility trenches, particularly when trenches are deep, when backfill materials are placed in thick lifts with insufficient compaction, and when water can access and infiltrate the trench backfill materials. The potential for water to access the backfill is increased where water can infiltrate flexible base materials due to insufficient penetration of curbs, and at sites where geological features can influence water migration into utility trenches (such as fractures within a rock mass or at contacts between rock and clay formations). It is our belief that another factor which can significantly impact settlement is the migration of fines within the backfill into the open voids in the underlying free-draining bedding material.

To reduce the potential for settlement in utility trenches, we recommend that consideration be given to the following:

- All backfill materials should be placed and compacted in controlled lifts appropriate for the type of backfill and the type of compaction equipment being utilized and all backfilling procedures should be tested and documented.
- Consideration should be given to wrapping free-draining bedding gravels with a geotextile fabric (similar to Mirafi 140N) to reduce the infiltration and loss of fines from backfill material into the interstitial voids in bedding materials.

Curb and Gutter

It is good practice to construct curbs such that the depth of the curb extends through the entire depth of the granular base material to act as a protective barrier against the infiltration of water into the granular base. Pavements that do not have this protective barrier to moisture tend to develop longitudinal cracks 1 to 2 ft from the edge of the pavement. Once these cracks develop, further degradation and weakening of the underlying granular base may occur due to water seepage through the cracks.

Longitudinal Cracking

It should be understood that asphalt pavement sections in highly expansive soil environments, such as those encountered at this site, can develop longitudinal cracking along unprotected pavement edges. In the semi-arid climate of south-central Texas this condition typically occurs along the unprotected edges of pavements where moisture fluctuation is allowed to occur over the lifetime of the pavements.

Pavements that do not have a protective barrier to reduce moisture fluctuation of the highly expansive clay subgrade between the exposed pavement edge and that beneath the pavement section tend to develop longitudinal cracks 1 to 4 ft from the edge of the pavement. Once these cracks develop, further degradation and weakening of the underlying granular base may occur due to water seepage through the cracks. The occurrence of these cracks can be more prevalent in the absence of lateral restraint and steep embankments. This problem can best be addressed by providing either a horizontal or vertical moisture barrier at the unprotected pavement edge.

A horizontal barrier is commonly in the form of a paved shoulder extending 8 feet or greater beyond the edge of the pavement. Other methods of shoulder treatment, such as using geofabrics beyond the edge of the roadway, are sometimes used in an effort to help reduce longitudinal cracking. Although this alternative does not eliminate the longitudinal cracking phenomenon, the location of the cracking is transferred to the shoulder rather than within the traffic lane.

Vertical barriers installed along the unprotected edges of roadway pavements are also effective in preventing non-uniform drying and shrinkage of the subgrade clays. These barriers are typically in the form of a vertical moisture barrier/membrane extending 6 feet or greater below the top of the subgrade at the pavement edge. Both types of barriers must be sealed at the edge of the pavement to prevent a crack that would facilitate the drying of the subgrade clays.

At a minimum, we recommend that the curbs are constructed such that the depth of the curb extends through the entire depth of the granular base material and into the subgrade to act as a protective barrier against the infiltration of water into the granular base.

In most cases, a longitudinal crack does not immediately compromise the structural integrity of the pavement system. However, if left unattended, infiltration of surface water runoff into the crack will result in isolated saturation of the underlying base. This will result in pumping of the flexible base, which could lead to rutting, cracking, and potholes. For this reason, we recommend that the owner of the facility immediately seal the cracks and develop a periodic sealing program.

Pavement Maintenance

Regular pavement maintenance is critical in maintaining pavement performance over a period of several years. All cracks that develop in asphalt pavements should be regularly sealed. Areas of moderate to severe fatigue cracking (also known as alligator cracking) should be sawcut and removed. The underlying base should be checked for contamination or loss of support and any insufficiencies fixed or removed and the entire area patched. All cracks that develop in concrete pavements should be routed and sealed regularly. Joints in concrete pavements should be maintained to reduce the influx of incompressible materials that restrain joint movement and cause spalling and/or cracking. Other typical TxDOT, City of San Antonio/New Braunfels maintenance techniques should be followed as required.

Construction Traffic

Construction traffic on prepared subgrade, granular base or asphalt treated base (black base) should be restricted as much as possible until the protective asphalt surface pavement is applied. Significant damage to the underlying layers resulting in weakening may occur if heavily loaded vehicles are allowed to use these areas.

CONSTRUCTION RELATED SERVICES

CONSTRUCTION MATERIALS TESTING AND OBSERVATION SERVICES

As presented in the attachment to this report, *Important Information About Your Geotechnical Engineering Report*, subsurface conditions can vary across a project site. The conditions described in this report are based on interpolations derived from a limited number of data points. Variations will be encountered during construction, and only the geotechnical design engineer will be able to determine if these conditions are different than those assumed for design.
Construction problems resulting from variations or anomalies in subsurface conditions are among the most prevalent on construction projects and often lead to delays, changes, cost overruns, and disputes. These variations and anomalies can best be addressed if the geotechnical engineer of record, RKI is retained to perform construction observation and testing services during the construction of the project. This is because:

- RKI has an intimate understanding of the geotechnical engineering report's findings and recommendations. RKI understands how the report should be interpreted and can provide such interpretations on site, on the client's behalf.
- RKI knows what subsurface conditions are anticipated at the site.
- RKI is familiar with the goals of the owner and project design professionals, having worked with them in the development of the geotechnical workscope. This enables RKI to suggest remedial measures (when needed) which help meet the owner's and the design teams' requirements.
- RKI has a vested interest in client satisfaction and thus assigns qualified personnel whose principal concern is client satisfaction. This concern is exhibited by the manner in which contractors' work is tested, evaluated and reported, and in selection of alternative approaches when such may become necessary.
- RKI cannot be held accountable for problems which result due to misinterpretation of our findings or recommendations when we are not on hand to provide the interpretation which is required.

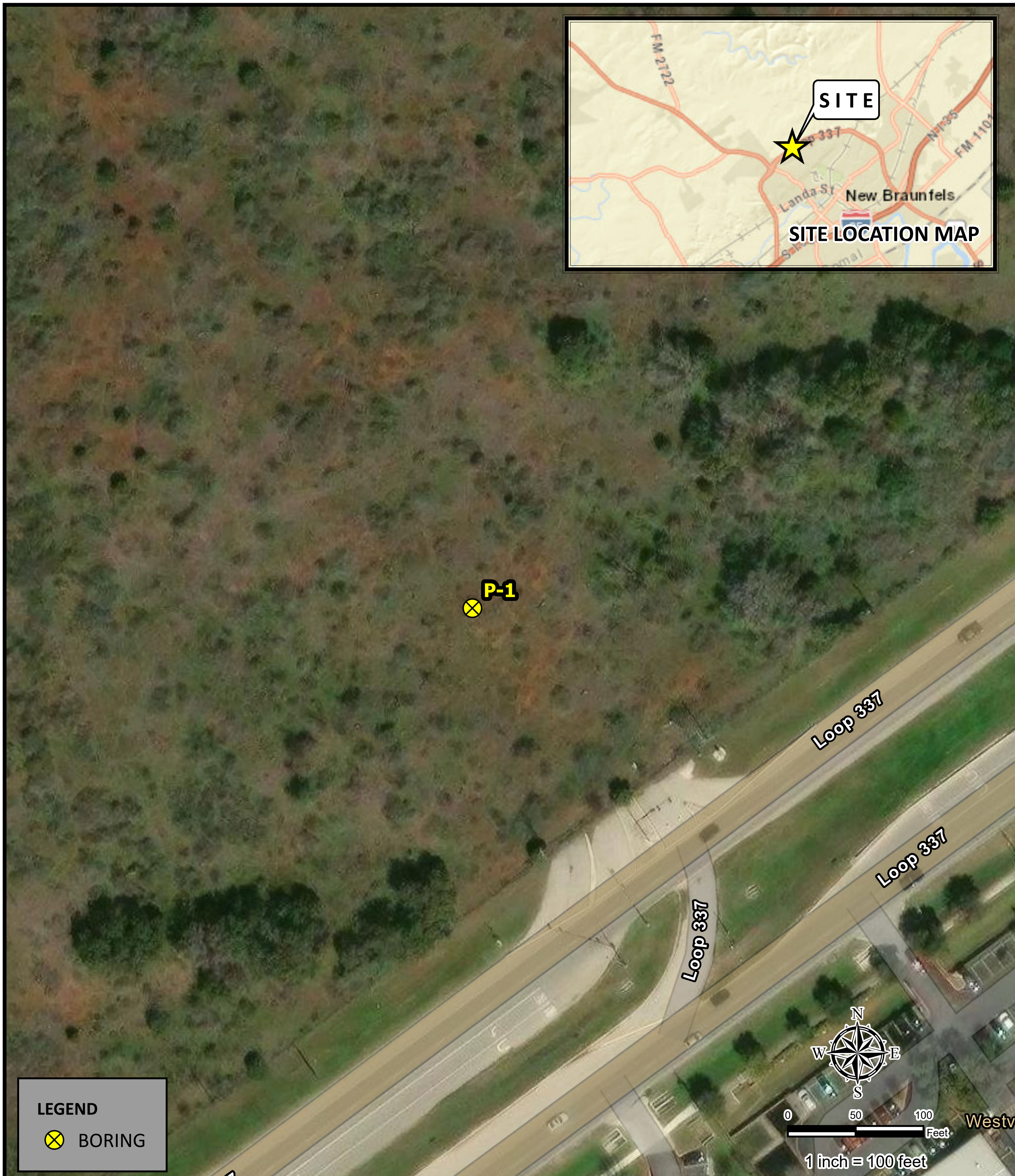
BUDGETING FOR CONSTRUCTION TESTING

Appropriate budgets need to be developed for the required construction testing and observation activities. At the appropriate time before construction, we advise that RKI and the project designers meet and jointly develop the testing budgets, as well as review the testing specifications as it pertains to this project.

Once the construction testing budget and scope of work are finalized, we encourage a preconstruction meeting with the selected contractor to review the scope of work to make sure it is consistent with the construction means and methods proposed by the contractor. RKI looks forward to the opportunity to provide continued support on this project and would welcome the opportunity to meet with the Project Team to develop both the scope and budget for these services.

* * * * * * * * * * * * * * * * * *

ATTACHMENTS



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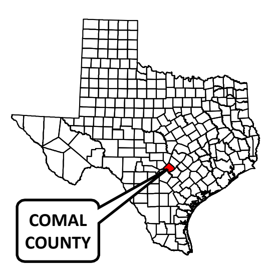
Hybrid Reference Layer: Sources: Esri, TomTom, Garmin, FAO, NOAA, USGS, © OpenStreetMap contributors, and the GIS User Community
World Imagery: Vantor

BORING LOCATION MAP

Roadway D at Veramendi Precinct 9
Veramendi Master Planned Development
New Braunfels, Texas



PROJECT No.:	ANA26-003-00
ISSUE DATE:	1/19/2026
DRAWN BY:	BM
CHECKED BY:	SS
REVIEWED BY:	IM

FIGURE

1

NOTE: This Drawing is Provided for Illustration Only, May Not be to Scale and is Not Suitable for Design or Construction Purposes

LOG OF BORING NO. P-1

Roadway D at Veramendi Precinct 9
Veramendi Master Planned Development
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hand Auger

LOCATION: N 29.72323; W 98.14781

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	SHEAR STRENGTH, TONS/FT^2 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 / PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT 10 20 30 40 50 60 70 80	PLASTICITY INDEX	% -200
			LEAN CLAY, Dark Brown				28	
			LIMESTONE, Tan and Gray					
			Boring Terminated					

Depth scale: 5, 10, 15

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	1.5 ft	**DEPTH TO WATER:**	Dry	**PROJ. No.:**	ANA26-003-00
DATE DRILLED:	1/20/2026	**DATE MEASURED:**	1/20/2026	**FIGURE:**	2

KEY TO TERMS AND SYMBOLS

MATERIAL TYPES

SOIL TERMS

- CALCAREOUS
- CALICHE
- CLAY
- CLAYEY
- GRAVEL
- GRAVELLY
- PEAT
- SAND
- SANDY
- SILT
- SILTY
- FILL

ROCK TERMS

- CHALK
- CLAYSTONE
- CLAY-SHALE
- CONGLOMERATE
- DOLOMITE
- IGNEOUS
- LIMESTONE
- MARL
- METAMORPHIC
- SANDSTONE
- SHALE
- SILTSTONE

OTHER

- ASPHALT
- BASE
- CONCRETE/CEMENT
- BRICKS / PAVERS
- WASTE
- NO INFORMATION

WELL CONSTRUCTION AND PLUGGING MATERIALS

- BLANK PIPE
- SCREEN
- BENTONITE
- CEMENT GROUT
- BENTONITE & CUTTINGS
- CONCRETE/CEMENT
- CUTTINGS
- GRAVEL
- SAND
- VOLCLAY

SAMPLE TYPES

- AIR ROTARY
- GRAB SAMPLE
- CORE
- GEOPROBE SAMPLER
- ROTOSONIC -DAMAGED
- MUD ROTARY
- NO RECOVERY
- NX CORE
- PITCHER
- ROTOSONIC -INTACT
- SHELBY TUBE
- SPLIT BARREL
- SPLIT SPOON
- TEXAS CONE PENETROMETER
- DISTURBED

STRENGTH TEST TYPES

- POCKET PENETROMETER
- TORVANE
- UNCONFINED COMPRESSION
- TRIAXIAL COMPRESSION UNCONSOLIDATED-UNDRAINED
- TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED

NOTE: VALUES SYMBOLIZED ON BORING LOGS REPRESENT SHEAR STRENGTHS UNLESS OTHERWISE NOTED

PROJECT NO. ANA26-003-00

RABAKISTNER

REVISED 04/2012

FIGURE 3a

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

Terms used in this report to describe soils with regard to their consistency or conditions are in general accordance with the discussion presented in Article 45 of SOILS MECHANICS IN ENGINEERING PRACTICE, Terzaghi and Peck, John Wiley & Sons, Inc., 1967, using the most reliable information available from the field and laboratory investigations. Terms used for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in American Society for Testing and Materials D2487-06 and D2488-00, Volume 04.08, Soil and Rock; Dimension Stone; Geosynthetics; 2005.

The depths shown on the boring logs are not exact, and have been estimated to the nearest half-foot. Depth measurements may be presented in a manner that implies greater precision in depth measurement, i.e 6.71 meters. The reader should understand and interpret this information only within the stated half-foot tolerance on depth measurements.

RELATIVE DENSITY

Penetration Resistance Blows per ft	Relative Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium Dense
30 - 50	Dense
> 50	Very Dense

COHESIVE STRENGTH

Resistance Blows per ft	Consistency	Cohesion TSF
0 - 2	Very Soft	0 - 0.125
2 - 4	Soft	0.125 - 0.25
4 - 8	Firm	0.25 - 0.5
8 - 15	Stiff	0.5 - 1.0
15 - 30	Very Stiff	1.0 - 2.0
> 30	Hard	> 2.0

PLASTICITY

Plasticity Index	Degree of Plasticity
0 - 5	None
5 - 10	Low
10 - 20	Moderate
20 - 40	Plastic
> 40	Highly Plastic

ABBREVIATIONS

B = Benzene
T = Toluene
E = Ethylbenzene
X = Total Xylenes
BTEX = Total BTEX
TPH = Total Petroleum Hydrocarbons
ND = Not Detected
NA = Not Analyzed
NR = Not Recorded/No Recovery
OVA = Organic Vapor Analyzer
ppm = Parts Per Million

Qam, Qas, Qal = Quaternary Alluvium
Qat = Low Terrace Deposits
Qbc = Beaumont Formation
Qt = Fluviatile Terrace Deposits
Qao = Seymour Formation
Qle = Leona Formation
Q-Tu = Uvalde Gravel
Ewi = Wilcox Formation
Emi = Midway Group
Mc = Catahoula Formation
EI = Laredo Formation
Kknm = Navarro Group and Marlbrook Marl
Kpg = Pecan Gap Chalk
Kau = Austin Chalk

Kef = Eagle Ford Shale
Kbu = Buda Limestone
Kdr = Del Rio Clay
Kft = Fort Terrett Member
Kgt = Georgetown Formation
Kep = Person Formation
Kek = Kainer Formation
Kes = Escondido Formation
Kew = Walnut Formation
Kgr = Glen Rose Formation
Kgru = Upper Glen Rose Formation
Kgrl = Lower Glen Rose Formation
Kh = Hensell Sand

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil type.
Interlayered	Soil sample composed of alternating layers of different soil type.
Intermixed	Soil sample composed of pockets of different soil type and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of carbonate.
Carbonate	Having more than 50% carbonate content.

SAMPLING METHODS

RELATIVELY UNDISTURBED SAMPLING

Cohesive soil samples are to be collected using three-inch thin-walled tubes in general accordance with the Standard Practice for Thin-Walled Tube Sampling of Soils (ASTM D1587) and granular soil samples are to be collected using two-inch split-barrel samplers in general accordance with the Standard Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). Cohesive soil samples may be extruded on-site when appropriate handling and storage techniques maintain sample integrity and moisture content.

STANDARD PENETRATION TEST (SPT)

A 2-in.-OD, 1-3/8-in.-ID split spoon sampler is driven 1.5 ft into undisturbed soil with a 140-pound hammer free falling 30 in. After the sampler is seated 6 in. into undisturbed soil, the number of blows required to drive the sampler the last 12 in. is the Standard Penetration Resistance or "N" value, which is recorded as blows per foot as described below.

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows Per Foot	Description
25	25 blows drove sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows drove sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows drove sampler 3 inches during initial 6-inch seating interval.

NOTE: To avoid damage to sampling tools, driving is limited to 50 blows during or after seating interval.

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Roadway D at Veramendi Precinct 9
Veramendi Master Planned Development
New Braunfels, Texas

FILE NAME: GINT ANA26-003-00.GPJ 2/2/2026

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
P-1	0.0 to 1.0		9	45	17	28	CL				
	1.0 to 1.5		1								

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial

PROJECT NO. ANA26-003-00

RABA KISTNER
a kiwa company

Project Number: ANA26-003-00
Test Date: January 20, 2026

DCP TEST DATA

B-1

Roadway D at Veramendi Precinct 9
Veramendi Master Planned Development
New Braunfels, Texas

Type of Ham.	No. of Blows	Penetration Incre. (mm)	Penetration Cumm. (in)	CBR (%)	M_R (ksi)	q_{ult} (ksf)
1	1	25	1	8	12	2.17
1	3	40	2.6	16	24	3.44
1	3	40	4.1	16	24	3.44
1	3	27	5.2	25	37.5	4.63
1	3	28	6.3	24	36	4.51
1	4	28	7.4	33	49.5	5.57
1	6	27	8.5	54	81	7.72
1	10	35	9.8	72	108	9.35
1	10	25	10.8	105	157.5	12.01
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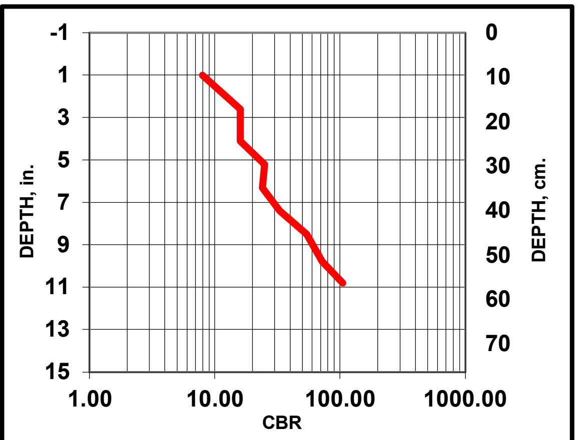
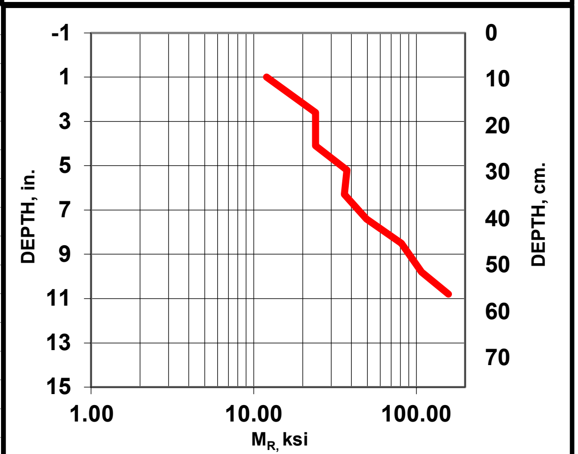
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





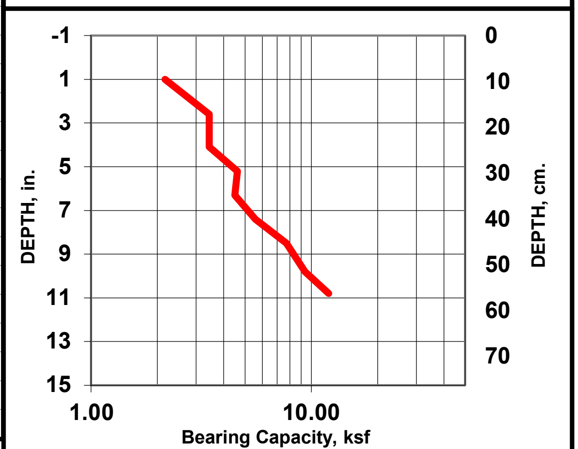
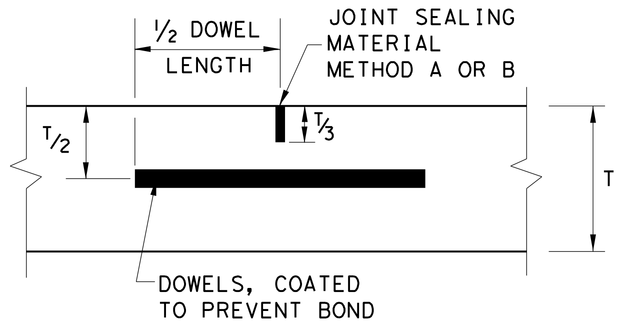
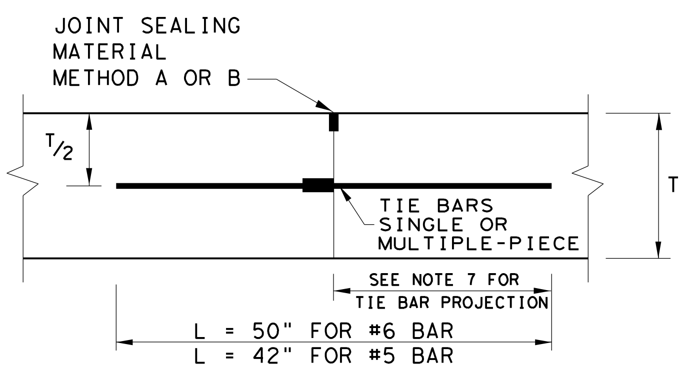


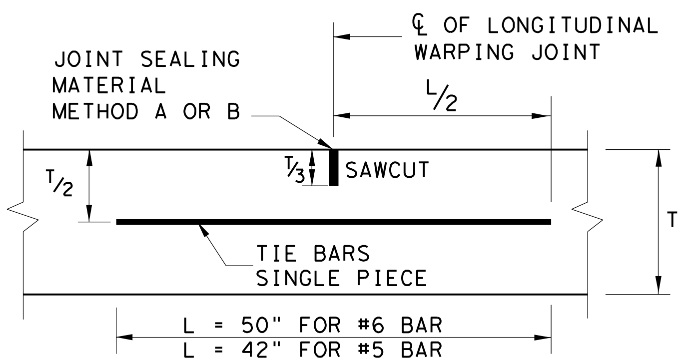
Figure 5



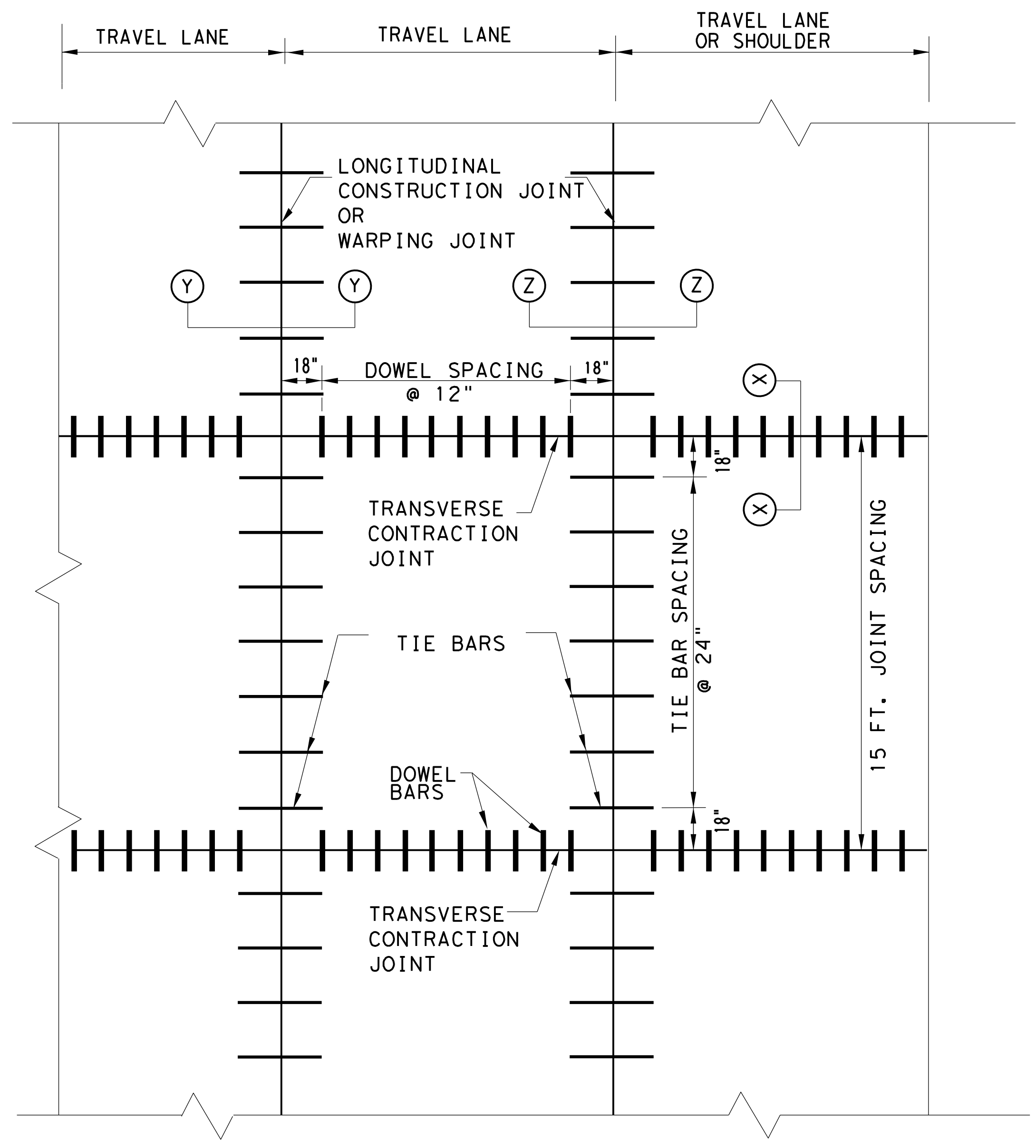
TRANSVERSE CONTRACTION JOINT
SECTION X-X



LONGITUDINAL CONSTRUCTION JOINT
SECTION Y-Y



LONGITUDINAL WARPING JOINT
SECTION Z-Z



TYPICAL PAVEMENT LAYOUT
PLAN VIEW (NOT TO SCALE)

GENERAL NOTES

1. DETAILS FOR PAVEMENT WIDTH, PAVEMENT THICKNESS AND THE CROWN CROSS-SLOPE SHALL BE SHOWN ELSEWHERE IN THE PLANS. FOR PAVEMENTS WIDER THAN 100 FT. WITHOUT A FREE LONGITUDINAL JOINT, ADDITIONAL DETAILS MAY BE SHOWN ELSEWHERE IN THE PLANS.
2. FOR FURTHER INFORMATION REGARDING THE PLACEMENT OF CONCRETE AND LOAD TRANSFER DEVICES REFER TO THE GOVERNING SPECIFICATION FOR "CONCRETE PAVEMENT".
3. THE SPACING BETWEEN TRANSVERSE CONTRACTION JOINTS SHALL BE 15 FT. UNLESS OTHERWISE SHOWN IN THE PLANS.
4. TRANSVERSE CONSTRUCTION JOINTS MAY BE FORMED BY USE OF METAL OR WOOD FORMS EQUAL IN DEPTH TO THE DEPTH OF PAVEMENT, OR BY METHODS APPROVED BY THE ENGINEER.
5. USE HAND-OPERATED IMMERSION VIBRATORS TO CONSOLIDATE THE CONCRETE ADJACENT TO ALL THE FORMED JOINTS.
6. PAVEMENT WIDTHS OF MORE THAN 15 FT. SHALL HAVE A LONGITUDINAL JOINT (SECTION Z-Z OR SECTION Y-Y). THESE JOINTS SHALL BE LOCATED WITHIN 6 IN. OF THE LANE LINE UNLESS THE JOINT LOCATION IS SHOWN ELSEWHERE ON THE PLANS.
7. THE MINIMUM PROJECTION OF TIE BARS INTO THE ADJACENT PLACEMENT IS 22.5 IN. FOR #6 BARS AND 18.5 IN. FOR #5 BARS.
8. WHEN TYING CONCRETE GUTTER AT A LONGITUDINAL JOINT, THE TIE BAR LENGTH OR POSITION MAY BE ADJUSTED. PROVIDE 3 IN. OF CONCRETE COVER FROM THE BACK OF GUTTER TO THE END OF TIE BAR.
9. REPLACE MISSING OR DAMAGED TIE BARS WITHOUT ADDITIONAL COMPENSATION BY DRILLING MIN. 10 IN. DEEP AND GROUTING TIE BARS WITH TYPE III, CLASS C EPOXY. MEET THE PULL-OUT TEST REQUIREMENTS IN ITEM 361.
10. WHEN A MONOLITHIIC CURB IS SPECIFIED, THE JOINT IN THE CURB SHALL COINCIDE WITH PAVEMENT JOINTS AND MAY BE FORMED BY ANY MEANS APPROVED BY THE ENGINEER.
11. DOWEL BAR PLACEMENT TOLERANCE SHALL BE +/- 1/4 IN. HORIZONTALLY AND VERTICALLY UNLESS OTHERWISE SPECIFIED.WHERE DOWEL BAR BASKETS ARE USED, REMOVE OR CUT THE SHIPPING WIRES.
12. THE DETAIL FOR JOINT SEALANT AND RESERVOIR IS SHOWN ON STANDARD SHEET "CONCRETE PAVING DETAILS, JOINT SEALS."

TABLE NO.1 DOWELS (SMOOTH BARS)

SLAB THICKNESS T (IN.)	BAR DIA. AND LENGTH	AVERAGE SPACING (IN.)
6 to 7.5	1" X 18"	12
8 to 10	1 1/4" X 18"	12
>= 10.5	1 1/2" X 18"	12

TABLE NO.2 TIE BARS (DEFORMED BARS)

SLAB THICKNESS T, (IN.)	BAR SIZE	AVERAGE SPACING (IN.)
6 to 7.5	#5	24
>= 8	#6	24

Figure 6a

ANA26-003-00

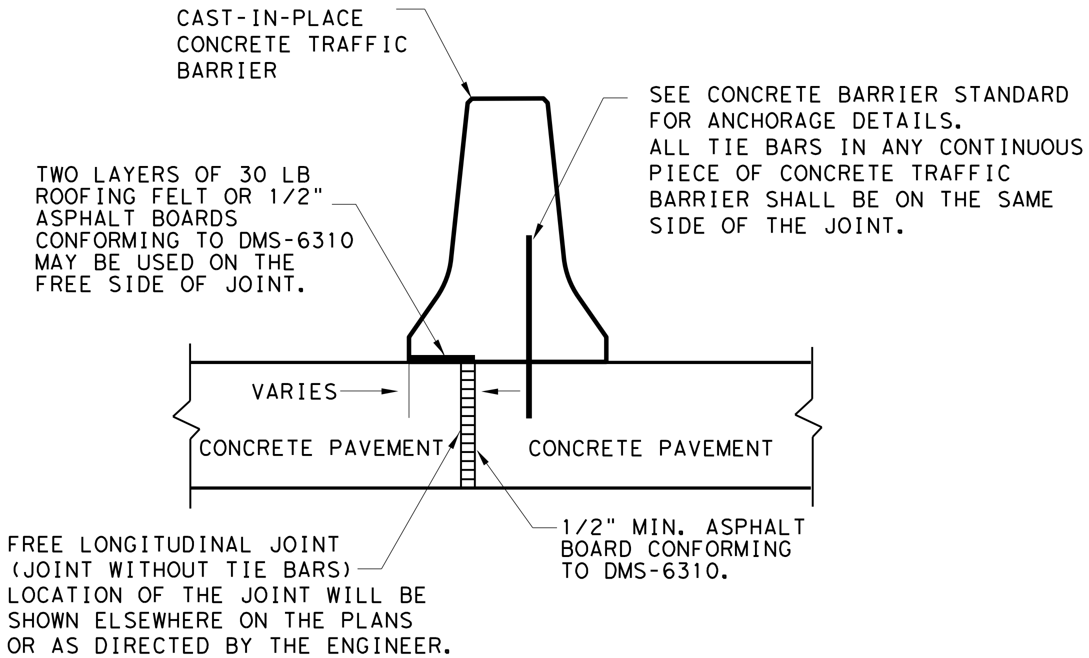
SHEET 1 OF 2

Texas Department of Transportation
Design Division Standard

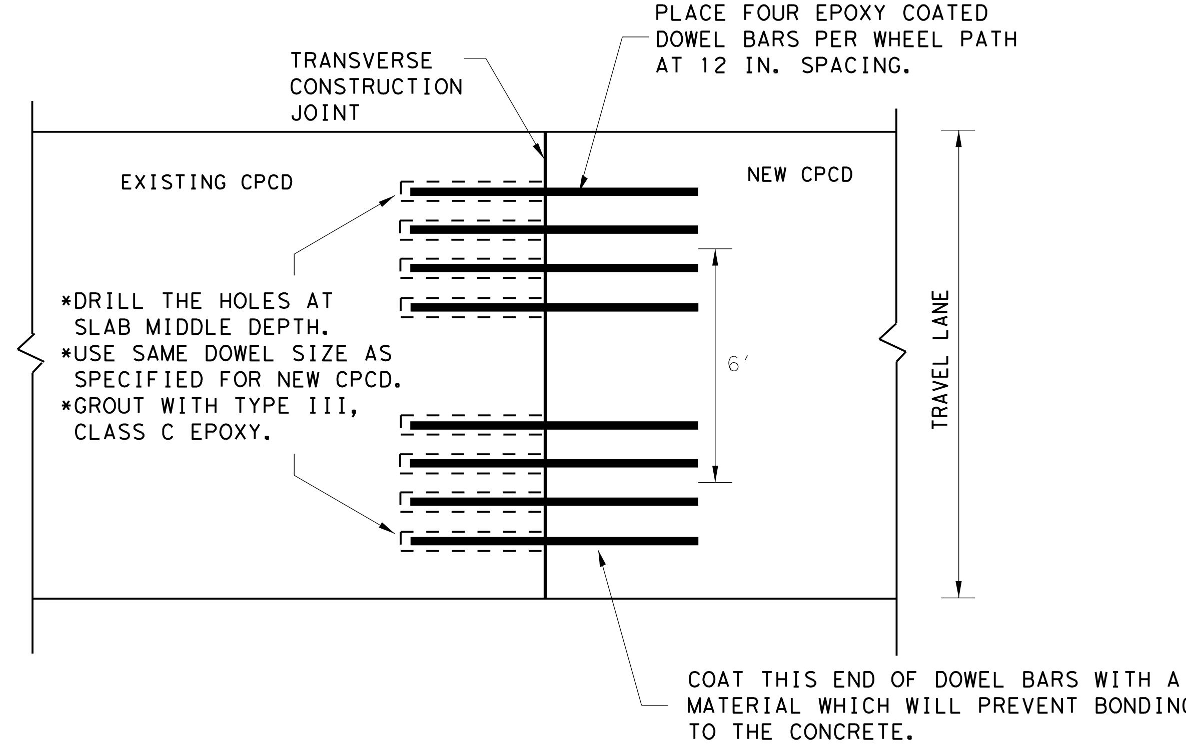
CONCRETE PAVEMENT CONTRACTION DESIGN
T-6 to 12 INCHES
CPCD-24

FILE: cpcd24.dgn	DN: CES	CK: KM	DW:	CK: AN
©TxDOT: Sept 2024	CONT	SECT	JOB	HIGHWAY
REVISIONS	DIST	COUNTY		SHEET NO.

DATE:
FILE:



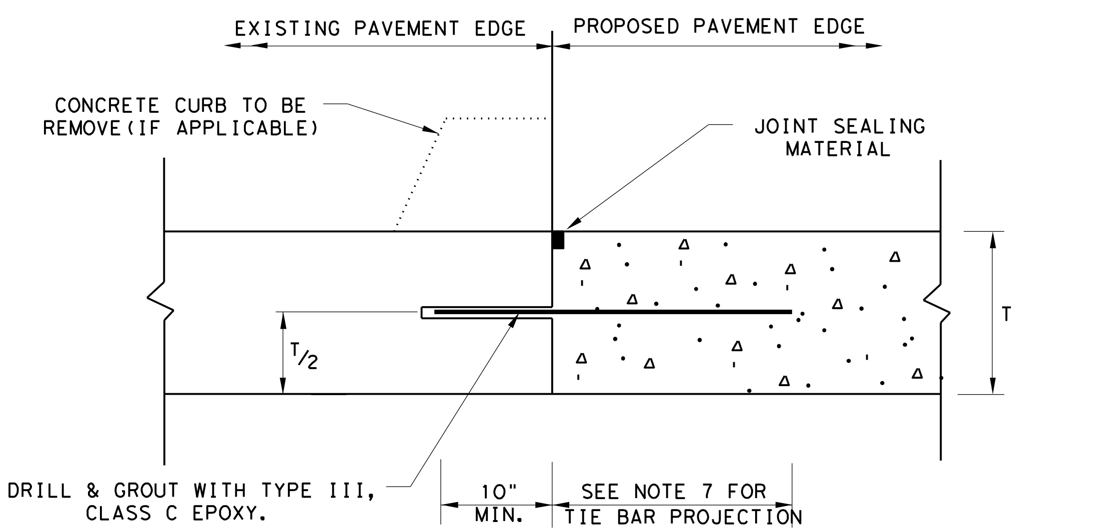
FREE LONGITUDINAL JOINT DETAIL



TRANSVERSE JOINT DETAIL

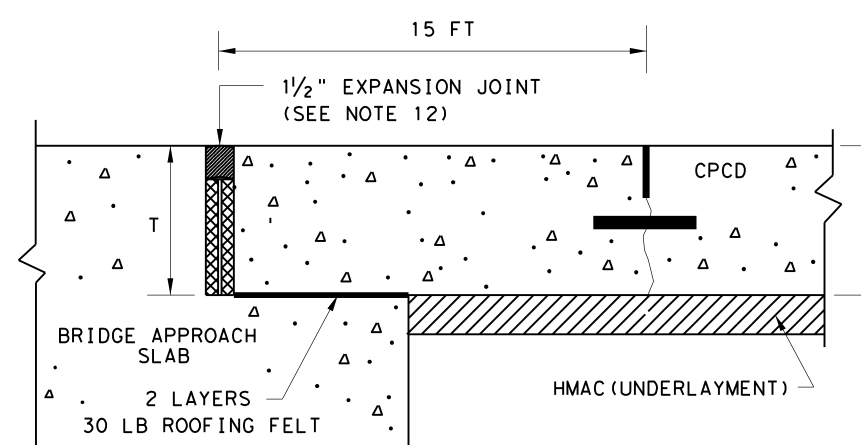
EXISTING CPCD TO NEW CPCD

PLAN VIEW (NOT TO SCALE)



1. USE A DRILL BIT WITH A DIAMETER THAT IS 1/8 IN. GREATER THAN THAT OF THE TIE BAR DIAMETER.
2. BEFORE CONCRETE PLACEMENT, PERFORM PULL-OUT TESTS ON EPOXY-GROUTED TIE BARS IN ACCORDANCE WITH ITEM 360.
3. SPACE TIE BARS AT 24" SPACING. USE #6 TIE BARS FOR 8" AND THICKER PAVEMENTS, USE #5 TIE BARS FOR LESS THAN 8" THICK PAVEMENTS.
4. THE TRANSVERSE JOINTS OF PROPOSED PAVEMENT SHALL COINCIDE WITH EXISTING PAVEMENT JOINTS UNLESS OTHERWISE SHOWN ON THE PLANS.

LONGITUDINAL WIDENING JOINT DETAIL



TRANSVERSE EXPANSION JOINT DETAIL

AT BRIDGE APPROACH

SHEET 2 OF 2

Texas Department of Transportation | Design Division Standard

CONCRETE PAVEMENT CONTRACTION DESIGN

T-6 to 12 INCHES

CPCD-24

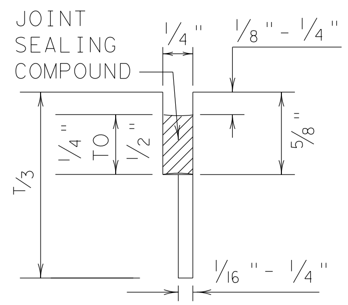
FILE: cpcd24.dgn	DN: CES	CK: KM	DW:	CK: AN
©TxDOT: Sept 2024	CONT	SECT	JOB	HIGHWAY
REVISIONS	DIST		COUNTY	SHEET NO.

Figure 6b

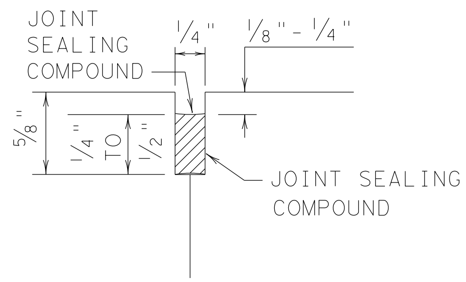
ANA26-003-00

DATE:
FILE:

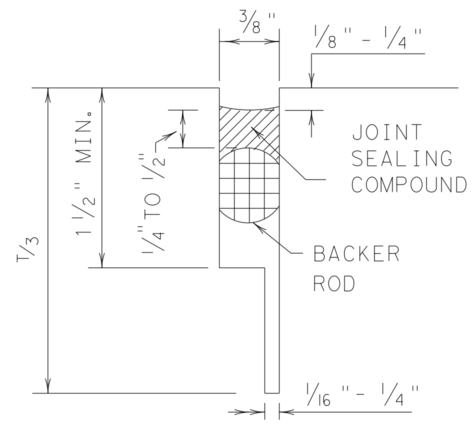
METHOD B: JOINT SEALING COMPOUND



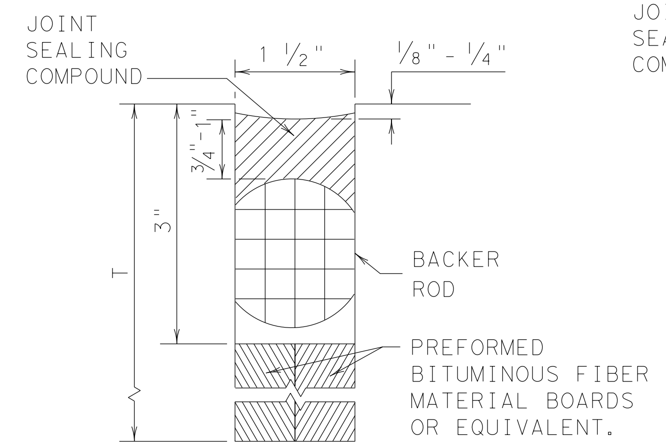
LONGITUDINAL SAWED CONTRACTION JOINT



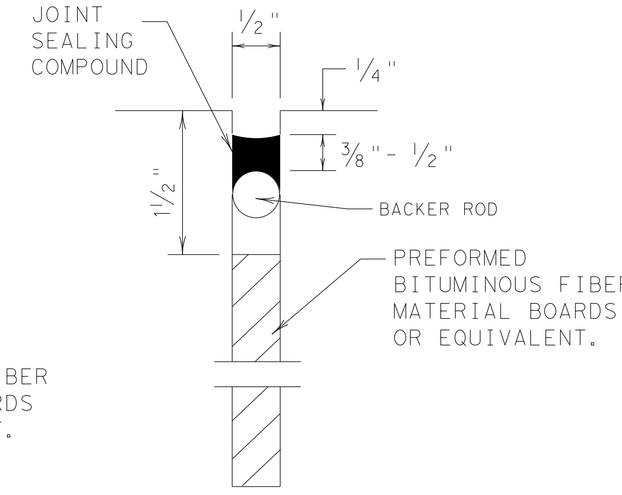
LONGITUDINAL OR TRANSVERSE CONSTRUCTION JOINT



TRANSVERSE SAWED CONTRACTION JOINT

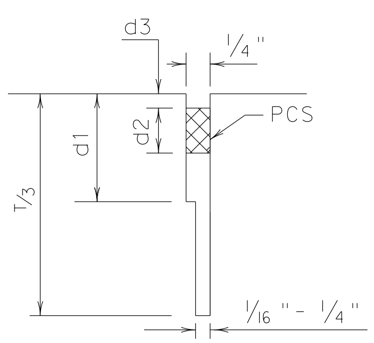


TRANSVERSE FORMED EXPANSION JOINT

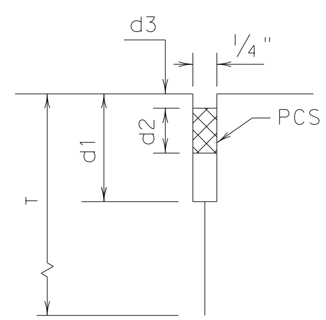


FORMED ISOLATION JOINT

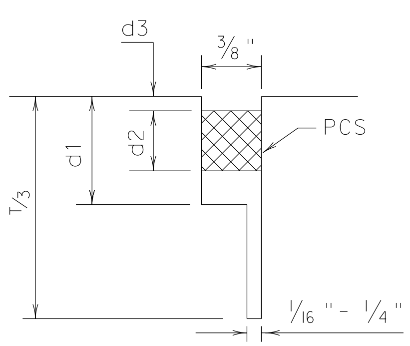
METHOD A: PREFORMED COMPRESSION SEALS (PCS)(DMS-6310 CLASS 6)



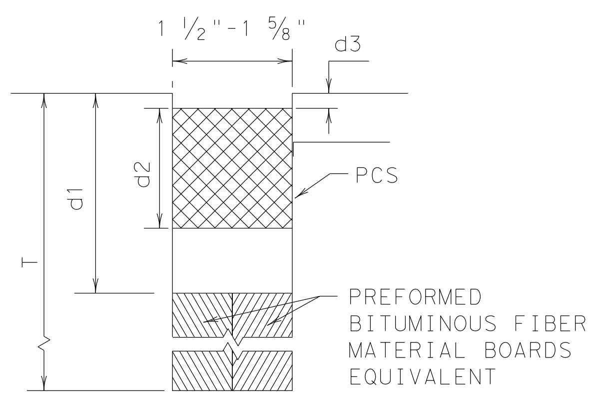
LONGITUDINAL SAWED CONTRACTION JOINT



LONGITUDINAL CONSTRUCTION JOINT



TRANSVERSE SAWED CONTRACTION JOINT



TRANSVERSE FORMED EXPANSION JOINT

Figure 6c

ANA26-003-00

GENERAL NOTES

1. UNLESS OTHERWISE SHOWN IN THE PLANS, EITHER METHOD "A" OR METHOD "B" MAY BE USED.
2. THE LOCATION OF JOINTS SHALL BE AS SHOWN ELSEWHERE IN THE PLANS.
3. THE JOINT RESERVOIR FOR SEALANT OR PCS SHALL BE SAWED UNLESS OTHERWISE SHOWN ON THE PLANS FOR THE LONGITUDINAL AND TRANSVERSE CONSTRUCTION JOINTS AND THE SAWED JOINTS.
4. DIMENSIONS d1, d2, AND d3 SHOWN IN METHOD A SHALL BE IN ACCORDANCE WITH THE PREFORMED COMPRESSION SEAL MANUFACTURER'S RECOMMENDATION.
5. REFER TO DMS-6310 "JOINT SEALANTS AND FILLERS" FOR THE CLASSIFICATIONS.
6. FOR SAWED LONGITUDINAL JOINT, LONGITUDINAL OR TRANSVERSE CONSTRUCTION JOINT, USE JOINT SEALANT CLASS 5 OR 8 UNLESS OTHERWISE SHOWN ON THE PLAN OR APPROVED.
7. FOR TRANSVERSE SAWED CONTRACTION, TRANSVERSE FORMED EXPANSION JOINT, AND ISOLATION JOINT USE JOINT SEALANT CLASS 5 OR 8 AT NEW JOINTS. USE JOINT SEALANT CLASS 4,5,7,OR 8 FOR MAINTAINING EXISTING JOINTS.
8. THE JOINTS SHALL BE CLEANED IN ACCORDANCE WITH THE ITEM 438 "CLEANING AND SEALING JOINTS" OR ITEM 713 "CLEANING AND SEALING JOINTS AND CRACKS (CONCRETE PAVEMENT)".
9. ISOLATION JOINTS ACCOMMODATE HORIZONTAL AND VERTICAL MOVEMENTS THAT OCCUR BETWEEN A PAVEMENT AND A STRUCTURE. ISOLATION JOINTS MAY BE USED FOR BRIDGE ABUTMENTS, INTERSECTIONS, CURB AND GUTTER, OLD AND NEW PAVEMENTS, OR AROUND DRAINAGE INLETS, MANHOLES, FOOTINGS AND LIGHTING STRUCTURES.

Texas Department of Transportation

Design Division Standard

CONCRETE PAVING DETAILS

JOINT SEALS

JS-14

FILE: js14.dgn	DN: TxDOT	DN: HC	DW: HC	CK: AN
© TxDOT: DECEMBER 2014	CONT	SECT	JOB	HIGHWAY
REVISIONS	DIST	COUNTY		SHEET NO.

DATE:
FILE:

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical- engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold- prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical- engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

GEOTECHNICAL BUSINESS COUNCIL
of the Geoprofessional Business Association

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@geoprofessional.org www.geoprofessional.org

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