April 27, 2022

Renovation Wrangler
102 E 26th Street
Bryan, Texas 77803

Attention: Ms. Katie Neason

Re: Report of Geotechnical Study for
Marco Polo Development
101 W 33rd Street
Bryan, Texas

Dear Ms. Neason:

Dudley Engineering LLC (DUDLEY) is pleased to submit to you the accompanying report that documents the results of a geotechnical study performed for the proposed Marco Polo development. The proposed development will be located at 101 W 33rd Street in Bryan, Texas. The geotechnical study was performed in accordance with DUDLEY's cost estimate dated February 26, 2022.

This letter transmits one (1) electronic copy of the report entitled "Report of Geotechnical Study for Marco Polo Development; 101 W 33rd Street; Bryan, Texas." The accompanying report summarizes the results of the subsurface investigation and laboratory testing program. In addition, foundation recommendations and design parameters are presented for the proposed buildings associated with the development based on the results of the geotechnical study.

DUDLEY sincerely appreciates the opportunity to work with Renovation Wranglers on this project. Please do not hesitate to contact us at (979) 777-0720 if you have any questions or if we can provide any additional assistance. We look forward to continuing our working relationship with you in the future.

Sincerely,
**Dudley Engineering, LLC**

Anna Dudley

Anna Dudley, P.E.
President

Enclosures: Geotechnical Report
Via E-mail: [katieneason@me.com]

# REPORT OF GEOTECHNICAL STUDY
# MARCO POLO DEVELOPMENT
# 101 W 33RD STREET
# BRYAN, TEXAS

*Prepared For*

Renovation Wrangler
102 E 26th Street
Bryan, Texas 77803

*Prepared By*


DUDLEY
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College Station, TX 77845
Texas Engineering Firm Registration F-18677


**DUDLEY Project No.: 22-00109**

April 27, 2022

STATE OF TEXAS
G. TAYLOR STINSON
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LICENSED
PROFESSIONAL ENGINEER

04/27/2021


G. Taylor Stinson, PE, MS
Principal

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# 1.0 INTRODUCTION

## 1.1 OVERVIEW

This report was prepared by Dudley Engineering LLC (DUDLEY) for Renovation Wranglers to document the results of a geotechnical study performed for the proposed Marco Polo development in Bryan, Texas. The geotechnical study was performed in accordance with DUDLEY's cost estimate dated February 26, 2022. The subsurface investigation was initiated on April 20, 2022 and was completed on the same date. The laboratory testing program was initiated shortly after the completion of drilling operations and was completed on April 23, 2022. A description of the subsurface information compiled during the field and laboratory phases of the project and an outline of DUDLEY's interpretation of the information is presented in this report for your review and consideration.

## 1.2 PROJECT DESCRIPTION

The proposed development will be located at 101 W 33rd Street as illustrated on Figure 1 in Attachment A of this report. The project will consist of two (2) structures. The first structure will have a total slab area of approximately 4,300 square feet and will be three stories in height. The second structure will have a total slab area of approximately 8,600 square feet and will also be three stories in height. The building superstructures are anticipated to consist of wood framing, with board and batten siding and composite siding. In addition, some metal framing may be required for the project.

Grading plans for the proposed development are not currently available. Nevertheless, we anticipate that changes to existing grade will be relatively minimal and on the order of 1-foot or less due to the developed nature of the surrounding area. DUDLEY should be notified if this assumption is incorrect because it may result in changes to the recommendations presented in this report.

## 1.3 SCOPE OF SERVICES

The scope of services associated with the current geotechnical study included the following:

- ***Task 1 – Subsurface Investigation:*** Secure information on subsurface conditions at the project site by drilling four (4) exploratory borings.

- ***Task 2 – Laboratory Testing Program:*** Perform laboratory tests on select soil samples recovered from the borings to aide in characterizing the subsurface materials.

- ***Task 3 – Engineering Analysis and Report Preparation:*** Evaluate the information developed from the subsurface investigation and laboratory testing program so that geotechnical recommendations and design parameters can be furnished for the proposed carwash facility.

# 2.0 SUBSURFACE INVESTIGATION

## 2.1 BORING DESIGNATION AND LOCATION

Four (4) borings were drilled for the project. The borings were designated as B-1 through B-4 as illustrated on Figure 2 in Attachment A of this report. The borings illustrated on Figure 2 were established by the drilling crew using a recreational hand-held global positioning system (GPS) device. The ground surface elevation at each boring was not specifically measure during drilling operations.

## 2.2 DRILLING AND SAMPLING

The borings were drilled with a CME 550 drilling rig. The borings were advanced dry with flight augers so that water levels could be monitored during and immediately after the completion of drilling operations. Soil samples were collected in accordance with ASTM D1586 – *Standard Test Methods for Standard Penetration Tests (SPT) and Split-Barrel Sampling of* Soil. Furthermore, SPT sampling utilized an automatic hammer, which generally has a higher energy transfer efficiency than traditional SPT equipment, i.e. safety hammers. The energy transfer ratio of the automatic hammer was not specifically evaluated as part of the current investigation; however, it is generally assumed to be 1.33 times more than that recorded using a standard safety hammer.

The borings were advanced 20 feet below the existing ground surface. Therefore, 60 linear feet of drilling was associated with the project. Representative soil samples were obtained at 1.5-foot intervals within the upper 10 feet of the stratigraphy. Below a depth of 10 feet, samples were collected at 5-foot intervals to the 20-foot termination depth of drilling.

## 2.3 BORING LOGS

The subsurface materials encountered at the borings were continuously logged in the field by a trained representative of DUDLEY. Following removal from the samplers, the soils were visually classified in accordance with ASTM D2488 – *Standard Practice for Description and Identification of Soils (Visual-Manual Procedures)* and a color description was assigned using the Munsell Soil Color chart. Pocket penetrometer readings were also taken on cohesive and cohesive-granular soil samples recovered from the borings to estimate strength. This information is summarized in Attachment B on the boring logs. A key to the terms and symbols used on the boring logs is also presented in Attachment B immediately after the boring logs.

The boring logs represent our present evaluation of the subsurface materials encountered at the project site based on observations and classification of the materials in the laboratory. The lines designating the interfaces between different soil types/formations are approximate and may be more gradual or more distinct. Variations will naturally occur and should be expected across the project site and between boring locations.

## 2.4 SAMPLE CUSTODY

Samples obtained as part of the subsurface investigation are and remain the property of Renovation Wranglers. Unless other arrangements are requested by Renovation Wranglers and mutually accepted by DUDLEY in writing, DUDLEY will dispose of the samples ten (10) days after the date of this report. Samples consumed by laboratory testing procedures were discard immediately after testing.

# 3.0 LABORATORY TESTING PROGRAM

The laboratory testing program was orientated in obtaining additional information on select soil samples recovered from the borings so that the soils could be classified in accordance with ASTM D2487 – *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)*. The specific tests performed as part of the laboratory testing program, along with the number of each test, are summarized below in Table 1.

Table 1. Laboratory Classification Testing Procedures

| ASTM Designation | Test Description | Number of Test Performed |
|---|---|---|
| ASTM D1140 | Standard Test Methods for Determining the Amount of Material Finer than 75-µm (No. 200) Sieve in Soils by Washing | 8 |
| ASTM D2216 | Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock Mass | 8 |
| ASTM D4318 | Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils | 8 |

The results of the laboratory tests are illustrated on the boring logs found in Attachment B of this report. In addition, the results are summarized below in Table 2. Soils that were not specifically tested in the laboratory were classified in accordance with ASTM D2488 – *Standard Practice for Description and Identification of Soils (Visual-Manual Procedures)* and based on similarities with soil samples that were tested in the laboratory.

Table 2. Summary of Laboratory Classification Test Results

| Range in Material Finer than No. 200 Sieve (%) | Range in Moisture Content (%) | Range in Liquid Limit (LL) | Range in Plastic Limit (PL) | Range in Plasticity Index (PI) |
|---|---|---|---|---|
| 17.7 – 91.9 | 16.9 – 29.6 | 26 – 70 | 15 – 23 | 11 – 47 |

# 4.0 SITE CONDITIONS

## 4.1 SURFACE CONDITIONS

Based on observation made during drilling operations, the project site is currently occupied by various residential-type structures. Areas that are not specifically improved consist of short grasses and trees. There did not appear to be any obvious surface features, such as open bodies of water or drainage channels, within the proposed building footprints during drilling operation or in readily available historic aerial photographs dating back to 1960.

## 4.2 SUBSURFACE CONDITIONS

The subsurface conditions encountered at the borings is presented in detail on the boring logs found in Attachment B of this report. The boring logs should be reviewed for a detailed description of the stratigraphy encountered at each boring. In summary, the subsurface stratigraphy consisted of variable thicknesses of clay and sand. The clays were classified as either CL type soils (low plasticity or lean clays) or CH type soils (high plasticity or fat clays) under the USCS. The consistency or strength of the clays was estimated to range from firm to hard, with most of the clays exhibiting stiff to hard consistency. The sands were classified as SC type soils (clayey sands) or SM type soils (silty sands), with most of the sands classifying as SC type soils. The relative density or strength of the sands was estimated to range from medium dense to dense.

## 4.3 WATER LEVEL OBSERVATIONS

Perched water or groundwater was not encountered at borings B-2 and B-3 during or immediately after the completion of drilling operations. Alternatively, water was encountered at borings B-1 and B-4 during drilling operations. The water level ranged from 11 feet to 14 feet below the existing ground surface. Immediately after the completion of drilling borings B-1 and B-4, the water level ranged from 10 feet to 11 feet below the existing ground surface.

The boreholes were subsequently backfilled with soil cuttings, and as a result, long-term water level readings could not be obtained for the project. It should be noted that subsurface water levels might change and can vary with seasonal rainfall patterns, long-term climate fluctuations, and with the influence of local site conditions. Therefore, the absence or presence of water during the subsurface investigation does not mean that subsurface water will not be present or will be present at the same depth during construction or over the design life of the structures.

# 5.0 ANALYSIS

Key considerations in the design of structures in this geographical area include: (1) the strength and settlement characteristics of the foundation soils, (2) the volumetric stability or potential shrink/swell movements of the foundation soils, and (3) seismic loading conditions for the project area. Each of these considerations is addressed in more detail in the following subsections based on the information compiled during the subsurface investigation and laboratory testing program.

## 5.1 STRENGTH AND SETTLEMENT

In general, most of the soils encountered at the boring locations exhibited enough strength to support the loads typically associated with three-story wood-framed structures. However, it should be recognized that weak soils, such as the clays and sands encountered in the surficial portion of the stratigraphy at boring B-4, may be encountered during construction. This is especially true if construction operations are initiated during or shortly after significant rainfall events. If weak surficial soils are encountered during proof rolling observations, the soils should be removed from the building pad areas prior to the placement of fill or foundation elements. **However, under no circumstance shall more than 2 feet of existing soils be removed from a failed proof rolling area without first contacting DULDEY for further evaluation and direction.**

## 5.2 VOLUMETRIC STABILITY

### 5.2.1 Moisture and Movement Active Zone

The moisture active zone was estimated for the project site based on unsaturated soil mechanics and typical changes in climatic conditions for the Bryan, Texas area. Based on these considerations and the soils encountered at the boring locations, the moisture active zone was estimated to extend approximately 12 feet below the existing ground surface. This depth is also commonly referred to as the depth of equilibrium suction or zone of seasonal moisture change and is generally assumed to correspond to a depth where the separation between total soil suction is less than 0.2 pF. The estimated depth to equilibrium suction agrees well with approximate depths outlined in the literature for dry temperate to temperate climates, i.e., site with Thornthwaite moisture indexes (TMI) ranging from -25 to +10.

The design movement active zone is almost always shallower than the moisture active zone. Based on unsaturated soil mechanics and horizontal flow, the design movement active zone was estimated to extend approximately 6 feet to 8 feet below the existing ground surface. Assumptions related to the estimated depth of the movement active zone include the following: (1) measures are taken to protect against ponding of water at the ground surface and lateral flow of water from on and off site and (2) protections must be implemented against accidental subsurface leaks, such as the lining of pressurized utility lines and an associated subsurface drainage system above the poly sheeting or the installation of devices to continuously monitor leaks and shut off water supply as needed. **Failure to address these measures and/or protections could result in deep-seated swell below the estimated movement active zone and could result in volumetric movements greater than those estimated in the following subsections of this report.**

### 5.2.2 Shrink/Swell Potential

Calculations were performed to estimate the magnitude of total potential swell movements in the subsurface soils based upon Texas Department of Transportation (TxDOT) Test Method TEX-124-E (Updated January 2017). Under this methodology, the magnitude of swell movement is referred to as potential vertical rise (PVR). Based upon the soils encountered in the estimated movement active zone, PVR was computed to be approximately 3.0 inches or less for the dry-to-wet condition.

Calculations were also performed to estimate the magnitude of potential shrink/swell movements in the subsurface soils based upon the methodology outlined in the 3rd Edition of the Post-Tensioning Institute (PTI) publication entitled *Design of Post-Tensioned Slabs-on-Ground*. Under this methodology, potential unrestrained differential soil movements were estimated to be approximately 2.25 inches for the post-construction center-lift condition, i.e., wet-to-dry conditions, and 3.5 inches or less for the post-construction edge-lift condition, i.e., dry-to-wet condition. The estimated movements did not consider the presence of perimeter vertical moisture barriers.

### 5.2.3 Site Improvement Techniques

The excavation/replacement scheme is one (1) of the most effective site improvement techniques for reducing potential shrink/swell movements beneath a structure. However, preventative measures must be implemented to prevent water from infiltrating into the higher permeability select fill soils and migrating downward to clays present below the estimated movement active zone and zone of seasonal moisture change. This site improvement technique involves the excavation or removal of a significant depth of volumetrically unstable clays from the upper portions of the stratigraphy and the replacement of the existing soils with select fill soils prone to low magnitudes of shrink/swell movements. It may also consist of adding select fill soils to elevate the building pad. Potential reductions in volumetric movements through the excavation and replacement scheme are summarized below in Table 3. Furthermore, select fill material and compaction requirements are outlined in Section 9 of this report.

Table 3. Reductions in Volumetric Movements by Placing Compacted, Select Fill

| Select Fill Pad Thickness (feet) | Estimated PVR (inches) Note 1 & Note 3 | Estimated Unrestrained Differential Soil Movements (inches) Note 2 & Note 3 |
|---|---|---|
| 1-foot or less | 3.0 inches or less | 3.5 inches or less |
| 2 feet | 2.25 inches or less | 2.0 inches or less |
| 4 feet | 1.25 inches or less | 1.0-inch or less |
| 6 feet | 0.75-inch or less | 0.75-inch or less |

**Table 3 Note:**
1. Computed using TEX-124-E.
2. Computed using the 3rd Edition of the PTI publication. Does not considered reductions associated with the installation of vertical moisture barriers.
3. Deep-seated swell movements associated with poor drainage or breaks in utility lines have been excluded from the estimated unrestrained differential soil movements.

Table 4 on the following page provides reductions in the estimated unrestrained differential soil movement based on the installation of perimeter vertical moisture barriers. The installation of vertical moisture barriers along the perimeter of the building can also assist with reducing unrestrained differential soil movements. The primary effect of moisture barriers is to extend edge effects away from the foundation and to minimize fluctuations of water content directly below the structure. Moisture barriers will not eliminate volumetric movements due to shrinking or swelling of the foundation soils. However, volumetric movements will generally occur slower and in a more uniform fashion.

Table 4. Reductions in Volumetric Movements with Vertical Moisture Barriers

| **Select Fill Pad Thickness (feet)** | **Vertical Moisture Barrier Depth** Note 1 & Note 3 | **Estimated Post-Construction Unrestrained Differential Soil Movements (inches)** Note 2 & Note 3 |
|---|---|---|
| 1-foot or less | 2 feet | 3.0 inches or less |
| 1-foot or less | 4 feet | 1.75 inches or less |
| 2 feet | 4 feet | 1.25 inches or less |

**Table 4 Note:**
1. Depth below adjacent ground surface established following construction.
2. Computed using the 3rd Edition of the PTI publication.
3. Deep-seated swell movements associated with poor drainage or breaks in utility lines have been excluded from the estimated unrestrained differential soil movements.

The vertical moisture barriers usually consist of an excavated trench lined with any impermeable membranes such as polyethylene, concrete, or impervious semi-hardening slurries. Polyethylene membranes should be durable enough to resist puncture and tearing during construction. A minimum thickness of 30 mils is recommended. Concrete or impervious semi-hardening slurries should have a minimum thickness of 6 inches; however, larger thickness may be more practical from a construction standpoint.

## 5.3 SEISMIC LOADING CONDITIONS

Based on the soils encountered at the boring locations and our experience with soils generally encountered in the upper 100 feet of the stratigraphy in this geographic area, Site Class D is recommended for the project site. Table 5 on the following page summarizes basic seismic design parameters that were determined based on the Site Class, the project location, and the provisions outlined in ASCE/SEI 7-16 – *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*.

Table 5. Seismic Design Parameters

| Parameter | Description | Value |
|---|---|---|
| $S_S$ | $MCE_R$ ground motion (period = 0.2s) | 0.066 g |
| $S_1$ | $MCE_R$ ground motion (period = 0.1s) | 0.040 g |
| $S_{DS}$ | Numeric seismic design value at 0.2s SA | 0.057 g |
| $S_{D1}$ | Numeric seismic design value at 0.1s SA | 0.040 g |

# 6.0 FOUNDATION RECOMMENDATIONS

## 6.1 PRIMARY BUILDING FOUNDATIONS

Based on the results of the subsurface investigation and laboratory testing program, it is DUDLEY's opinion that a subgrade supported foundation system may be considered for foundation support of the proposed structures. However, the foundation systems must be designed to resist differential volume change in the foundation soils and to prevent structural damage to the supported structures as outlined in the recently adopted 2021 International Building Code by the City of Bryan. The risk associated with a subgrade supported foundation system is outlined below in Table 6 based on ranges in unrestrained differential soil movement in the foundation soils.

Table 6. Subgrade Supported Foundation System Risk

| Range in Potential Unrestrained Differential Soil Movement | Perceived Level of Risk for Structures with Limited Movement Sensitive Finishes | Perceived Level of Risk for Structures with Extensive Movement Sensitive Finishes |
|---|---|---|
| ≤ 1.0-inch | Very Low | Low |
| Between 1.0-inch and 2.0 inches | Low | Low to moderate |
| 2.0 inches to 4.0 inches | Low to moderate | Moderate to high |
| > 4 inches | Moderate to high | High to very high |

Based on the existing conditions encountered at the borings, the risk associated with the project site would be considered **low to moderate** if limited movement sensitive finishes are associated with the proposed structures. Alternatively, the risk would be considered **moderate to high** if extensive movement sensitive finishes are associated with the structures. Reductions in the perceived level of risk may be achieved by considering the prevalence of movement sensitive finishes along the interior and exterior of the structures and by implementing one or more of the site improvement techniques previously outlined in Section 5 to achieve the desired level of performance.

The levels of risk previously outlined in Table 6 assume that positive drainage and vegetation control will be established around the perimeter of the buildings as outlined in Section 9. In addition, it assumes that the shape factor for each building does not exceed 32 and that the simplified shape factor (combined overlapping rectangle perimeter$^2$/area of overlapping rectangles) does not exceed 24. If either of these factors are exceeded, the designer should considered one (1) or more of the following: (1) modification to the foundation footprint to reduce the shape factor, (2) strengthened foundation systems (additional stiffening ribs and deepened ribs in areas of high torsion or non-prestressed reinforcement), or (3) geotechnical approaches (such as moisture barriers, excavation/replacement, moisture conditioning, or moisture injection) to reduce potential unrestrained differential soil movements to approximately 1.0-inch or less.

## 6.2 ISOLATED EXTERIOR COLUMN LOCATIONS

Columns are anticipated along the exterior of the buildings to support the porches/decks associated with the buildings. The columns are currently planned to be isolated from the primary foundation systems and directly abutting flatwork. Furthermore, maximum column loads will be on the order of approximately 45,000 pounds.

Drilled piers may be utilized to support the isolated exterior columns. However, the drilled piers must be designed to limit uplift movements typically associated with soil-to-pier adhesion. The placement of flatwork adjacent to the isolated pier foundation elements will also assist with minimizing uplift movements on the pier shafts due to soil-to-pier adhesion. Finally, temporary casing or slurry displacement techniques should be anticipated during pier drilling operations to prevent groundwater seepage and associated sidewall sloughing.

An alternative means of providing foundation supported to the isolated columns would be to extend the stiffened, subgrade supported foundation system out beyond the building perimeter. The isolated column locations could then be supported by grade beams of widened footings. This is the recommended approach by DUDLEY because it will result in the same foundation system and it will minimize issues associated with groundwater and sloughing soils.

# 7.0 FOUNDATION DESIGN PARAMETERS

## 7.1 GENERAL DESIGN PARAMETERS

Shallow foundation elements shall be designed to resist potential axial, uplift, and lateral loading conditions. Specific shallow foundation element design parameters for these loading conditions are provided below in Table 7 for shallow foundation elements founded in either existing on-site soils passing proof rolling observation or compacted, select fill soils. The actual bearing soils will depend on the thickness of select fill placed beneath the building and the required depth of the grade beams as determined in Section 7.2.

Table 7. Shallow Foundation Design Parameters

| | |
|---|---|
| **Minimum Founding Depth** Note 1 | Exterior foundation elements: 18 inches below adjacent ground surface |
| | Interior foundation elements: 18 inches below proposed finished floor elevation |
| **Minimum Width** | 12 inches |
| **Allowable Unit Base Resistance** | 2,700 psf (maximum loading, FS = 2.0) |
| | 1,800 psf (sustained loading, FS = 3.0) |
| **Estimated Footing Movement based on Sustained Loading** | Maximum settlement: 1.0-inch or less |
| | Differential settlement: 0.75-inch or less |
| **Lateral Sliding Resistance** | Coefficient of friction – 0.36 (FS = 1.5) Note 2 |
| | Adhesion – 400 psf (FS = 2.0) Note 3 |
| **Uplift Resistance** | See Note 4 |
| **Modulus of Subgrade Reaction for 1-ft by 1-ft Plate** | 175 psi/in (slab supported by compacted, select fill) |
| | 75 psi/in (slab supported by existing on-site soils) |

**Table 7 Note:**

1. Deeper founding depths may be required based on the design parameters furnished in Section 7.2.
2. For beams and footings bearing directly on compacted, select fill.
3. For beams and footings bearing directly on existing, on-site clays. Lateral sliding resistance shall not exceed one-half of the dead load.
4. The weight of the reinforced concrete footing (150 pcf) and the dead load acting on the footing may be considered when evaluating uplift resistance.

## 7.2 SHRINK/SWELL DESIGN PARAMETERS

As outlined in Section 1808.6 – *Design for expansive soils* of the 2021 International Building Code, moments, shears, and deflections for use in designing slab-on-ground, mat, or raft foundation supported by expansive soils shall be determined in accordance with WRI TF 700-R-07, PTI DC 10.5, or another rational design methodology. The following subsections provide geotechnical design parameters that can be utilized by the Structural Engineer for the WRI and PTI design methods.

### 7.2.1 WRI Design Parameters

Design information related to the WRI design method for subgrade supported foundations is provided below in Table 8. The design parameters were formulated based on a climatic rating ($C_w$) of 20, which is representative of drought durations on the order of 2.5 months. Table 8 provides design parameters for existing conditions and variable thickness of select fill that may be placed as part of grading operations and/or site improvements techniques orientated toward the excavation and replacement scheme.

Table 8. WRI Design Parameters

| Select Fill Building Pad Thickness | Effective Plasticity Index (PI) | Soil Climatic Rating (1 – C) |
|---|---|---|
| 1-foot or less | 45 | 0.30 |
| 2 feet | 35 | 0.20 |
| 4 feet | 25 | 0.10 |
| 6 feet | 22 | 0.07 |

**Table 8 Notes:**

1. The recommended design parameters do not consider the potential effects of non-climatic factors. These conditions include, but are not limited to, the location of trees and planters around the structure, poor drainage conditions, breaks in utility lines, etc.
2. The WRI/CRSI design procedure was formulated to limit deflections to L/480. The Structural Engineer should consider deeper beam depths and/or closer beam spacings than those computed using the WRI/CRSI procedure if stricter deflection criterion is required.

## 7.2.2 PTI Design Parameters

### *7.2.2.1 Excavation and Replacement*

Design information related to the PTI design method for subgrade supported foundations is provided below in Table 9 for existing conditions and variable thicknesses of compacted, select fill.

Table 9. PTI Design Parameters

| Select Fill Pad Thickness | $e_{m\text{-center}}$ (feet) | $e_{m\text{-edge}}$ (feet) | $y_{m\text{-center}}$ (in) | $y_{m\text{-edge}}$ (in) |
|---|---|---|---|---|
| 1-foot or less | 6.4 | 3.5 | 2.25 | 3.5 |
| 2 feet | 7.2 | 3.8 | 1.25 | 2.0 |
| 4 feet | 7.8 | 4.1 | 0.75 | 1.0 |
| 6 feet | 8.3 | 4.2 | 0.5 | 0.75 |

**Table 9 Notes:**

1. The recommended design parameters do not consider the potential effects of non-climatic factors. These conditions include, but are not limited to, the location of trees and planters around the structure, poor drainage conditions, breaks in utility lines, etc.

### *7.2.2.2 Vertical Moisture Barriers*

Design information related to the PTI design method for subgrade supported foundations is provided below in Table 10 for perimeter vertical moisture barriers that extend below the adjacent ground surface following the completion of construction.

Table 10. PTI Design Parameters for Perimeter Vertical Moisture Barriers

| Select Fill Pad Thickness | Vertical Moisture Barrier Depth (ft) Note 1 | $e_{m\text{-center}}$ (feet) | $e_{m\text{-edge}}$ (feet) | $y_{m\text{-center}}$ (in) | $y_{m\text{-edge}}$ (in) |
|---|---|---|---|---|---|
| 1-foot or less | 2 feet | 6.2 | 2.9 | 1.75 | 3.0 |
| 1-foot or less | 4 feet | 3.8 | 2.0 | 1.25 | 1.75 |
| 2 feet | 4 feet | 4.8 | 2.0 | 1.0 | 1.25 |

**Table 10 Notes:**

1. Depth below adjacent ground surface established following the completion of construction.

#### *7.2.2.3 Initial Tendon Stressing*

A coefficient of friction of 0.75 is recommended for initial tendon stressing when the slab is cast directly on a polyethylene sheet. A coefficient of friction of 1.0 is recommended for initial tendon stressing when the slab is cast on a sand layer without a polyethylene sheet. Reference Table 7 for recommended coefficient of friction or adhesion when evaluating sliding due to environmental forces such as wind.

## 7.3 ARCHITECTURAL AND STRUCTURAL ELEMENT DETAILING

The superstructure and architectural elements of the proposed buildings shall be designed to accommodate the potential shrink/swell movements or consolidation of the foundation soils. Jointing of interior dry walls above door and window openings and the use of slip joints between dry wall panels should be considered. If movement-sensitive floor coverings, such as ceramic tile, marble, or wood, must be placed in the structure, we recommend that strong consideration be given to the use of geotextile reinforcement layers and/or underlayment layers between the floor coverings and the slab. Also, the tile should be frequently jointed to minimize the manifestation of distress cracking associated with slab movement. The use of flexible plumbing connections for water and sewer piping can help reduce, but not eliminate, potential leakage frequently associated with slab movements. Similarly, the employment of "through-slab" sleeves for rigid electrical conduit can help to minimize distress to the electrical system. Furthermore, all drainage piping and general plumbing piping systems associated with the buildings or in proximity to the buildings should be leak tested following installation. Water produced from any leaks in drainage or pressure piping following construction could produce localized swelling movements in the foundation soils. The swelling movements may be of a greater magnitude than is typically associated with seasonal moisture variations as previously discussed in this report. These increased swelling movements could result in distress to foundation elements and the building superstructures.

# 8.0 DRILLED PIER DESIGN PARAMETERS AT ISOLATED COLUMN LOCATIONS

## 8.1 GENERAL

Isolated drilled piers may be required to support exterior columns associated with the porches/decks if the stiffened, subgrade supported foundation system cannot be extended beneath these structural elements. Drilled piers foundation elements shall be designed in accordance with applicable building code requirements to resist all potential axial, uplift, and lateral loading conditions. Design parameters for these loading conditions are provided in the following subsections.

## 8.2 AXIAL LOADING CONDITIONS

Design parameters for axial loads, i.e. downward loads, acting on the drilled pier foundation elements are provided below in Table 11.

Table 11. Drilled Pier Axial Load Design Parameters

| | |
|---|---|
| **Minimum Founding Depth** Note 1 | 16 feet below existing ground surface |
| **Minimum Shaft Diameter** | 12 inches to facilitate cleaning and inspection |
| **Allowable Unit Base Resistance** Note 2 | 9,000 psf (maximum loading conditions, FS = 2.0) |
| | 6,000 psf (sustained loading conditions, FS = 3.0) |
| **Allowable Unit Side Resistance** | 300 psf below a depth of 8 feet |
| **Ratio of Pier Bell Diameter to Shaft Diameter** Note 3 | 1:1 (straight shaft) |
| **Pier Settlement** Note 4 | Maximum settlement: 1.0-inch or less |
| | Differential settlement: 0.75-inch or less |

**Table 11 Note:**
1. Ground surface present at borings during drilling.
2. Assumes drilled piers will have a minimum spacing of three (3) pier end diameters, center to center. For spacings of one (1) pier end diameter, multiply allowable resistance values by 0.5. Interpolation can be used for spacings between one (1) and three (3) piers diameters.
3. Assumes that reinforcing steel and concrete will be placed in the drilled pier excavations shortly after drilling. **Temporary casing or slurry displacement installation should be anticipated.**
4. Based on a maximum shaft diameter of 3 feet and sustained loading conditions.

## 8.3 UPLIFT LOADING CONDITIONS

Uplift loading conditions associated with environmental forces, such as wind, may be resisted by considering the dead load of the drilled piers and the dead load acting on the piers. In addition, an allowable unit side resistance of 200 psf (FS = 3.0) may be considered below a depth of 8 feet relative to the final ground surface. The construction of a bell or underream at the base of the pier shafts is not recommended since granular soils and water were encountered at some of the boring locations.

DUDLEY also evaluated potential uplift movements of the drilled piers due to soil-to-pier adhesion acting on the shafts. This evaluation considered pier movement in an elastic medium based on soil-pier slip. Based on the evaluation, potential uplift movements in the piers were estimated to be approximately **0.75-inch or less.**

Tensile forces generated on the upper 8 feet of the pier shafts due to soil-to-pier adhesion were estimated to be approximately 1,000 psf and should also be accounted for when designing the longitudinal steel for the drilled pier foundation elements

## 8.4 LATERAL LOADING CONDITIONS

Table 12 below provides LPILE design parameters that can be used to evaluate lateral loading conditions acting on the drilled piers. The design parameters presented in Table 12 assume that the piers will have a minimum clear spacing of six (6) pier diameters. If closer spacings are required, P-multipliers should be applied to the static soil modulus. At a spacing of three (3) pier diameters, the adjusted soil modulus can be estimated by multiplying the static soil modulus by 0.6. For trailing piers in a line at three (3) diameters spacing, a P-multiplier of 0.4 should be used. Linear interpolation can be used for pier spacings between three (3) and six (6) diameters. Piers spaced closer than three (3) pier diameters should not be relied upon for lateral resistance.

Table 12. Drilled Pier LPILE Design Parameters

| Stratum | Depth | Stratum Description | Average Undrained Shear Strength | Effective Unit Weight | Static Soil Modulus | Soil Strain E50 |
|---|---|---|---|---|---|---|
| I | 0 – 15 | Stiff Clay | 1,000 psf | 120 pcf | 500 pci | 0.007 |

# 9.0 CONSTRUCTION CONSIDERATIONS

## 9.1 SITE PREPARATION

### 9.1.1 Demolition

Demolition shall consist of the destruction and removal of all non-vegetative matter encountered above, on, or below the ground surface within the construction limits. This shall include, but not be limited to, buildings, abandoned utilities, all material derived from the demolition of concrete items such as base courses, curbs, curb and gutters, sidewalks, floors, steps, driveways, drainage structures of all sorts, guard fences, and other miscellaneous items such as foundations or walls of any sort, iron or steel items, and asphaltic items such as pavement and base courses. Materials shall be salvaged as indicated or specified, or disposed of in accordance with applicable federal, state, and local regulations and/or ordinances.

### 9.1.2 Stripping and Clearing

Any vegetation existing within the building areas prior to construction shall be removed. In addition, any remaining organic matter and topsoil, as well as any weak, or wet soils, shall be stripped and removed from the building areas. The removal of the vegetation should include all roots, including the major root systems associated with large trees, both currently existing as well as previously existing on the site. The removal of the major root systems should include any desiccated soils present within the root bulbs of the trees. If the existing vegetation and organic materials are not removed from the proposed buildings, it is possible that the existing vegetation will interfere with the proposed construction and could potentially adversely impact the future performance of the proposed structures.

### 9.1.3 Proof Rolling

Prior to placing any fill soils, proof rolling should be performed with a 15-ton pneumatic roller or equivalent vehicle to identify weak surficial soil formations. Any weak surficial soils identified during proof rolling should be removed and replaced with acceptable fill. For the purposes of this report, weak soils are defined as soil exhibiting rutting greater than 2 inches or elastic deformations greater than 1-inch.

## 9.2 BUILDING PAD DESIGN

### 9.2.1 Excavation and Replacement

Prior to placing any select fill soils as part of the excavation and replacement scheme, the exposed subgrade soils shall be scarified to a depth of 6 inches. The moisture content of the scarified soils shall be adjusted to the optimum moisture content (OMC) + 2 percentage points to the OMC + 5 percentage points, inclusive, and the soils shall be compacted to at least 95 percent of the maximum dry density determined in accordance with ASTM D698 (Standard Proctor). This requirement may be waived by the Engineer of Record based on the amount of select fill placed beneath the structures.

### 9.2.2 Preventative Measures

If the excavation and replacement scheme is implemented or if select fill soils are placed to elevate the building pads, the select fill building pads should be of a uniform thickness to minimize potential differential movements in the foundation systems. DUDLEY also recommend that the limits of the select fill pad should not extend beyond the face of the foundation elements. Otherwise, the more moisture permeable soils of the select fill pad, which extend beyond the face of the foundation system, could serve as preferential pathways for moisture migrating from outside the structure areas to collect within the select fill pad (also referred to as the "bathtub effect"). This collected moisture could infiltrate into the clays still present within the stratigraphy following construction of the select fill pad and could result in increased magnitudes of swelling above those predicted in this report.

If the owner, designer, or contractor selects to extend the select fill pad beyond the foundation perimeters, a low permeability clay cap or approved equivalent is highly recommended to help minimize moisture infiltration into the select fill soil pad. The low permeability, clay "cap" should have a plasticity index (PI) between 20 and 35, inclusive, and shall be at least 1-foot in thickness. Compaction of the clay cap should be above the OMC and at least 92 percent of the maximum dry density as determined by ASTM D698 in non-structural areas and 95 percent in structural areas. Alternatively, perimeter grade beams may be extended below the select fill pad to minimize stormwater infiltration.

There are certain situations were foundation elements such as drilled piers, continuous footings, spread footing, etc. may extend beyond the perimeter of the building. In this case, the select fill building pad must extend beyond the limits of these foundation elements, i.e., general 3 to 5 feet. The clay "cap" previously referenced above and/or flatwork should be provided beyond the extended select fill building pad in these situations in order to minimize the potential for the development of the "bathtub effect".

## 9.3 SELECT FILL MATERIAL AND PLACEMENT REQUIREMENTS

Select fill used to replace weak surficial soils, to elevate the building pad above the existing ground surface to achieve drainage requirements, or as part of the excavation and replacement scheme should meet the material and compaction requirements outlined on the following page in Table 13. Compaction characteristics of the select fill shall be verified by in-place density tests. The tests should be performed on each 6-inch-thick lift at an average rate of one (1) test for every 2,000 square feet of plan area for the building pad. A minimum of three (3) tests should be performed for each distinct lift of fill.

Table 13. Select Fill Requirements

| Unified Soil Classification System (USCS) | Plasticity Index (PI) | Compaction Standard | Dry Unit Weight Note 1 | Moisture Content Note 2 |
|---|---|---|---|---|
| SC or CL | 7 to 20, inclusive | ASTM D698 | ≥ 95% $D_A$ | $W_{OPT}$ – 2% to $W_{OPT}$ + 3.0%, inclusive |

**Table 13 Notes:**
1. Maximum dry unit weight ($D_A$) determined in accordance with ASTM D698.
2. Optimum moisture content ($W_{OPT}$) determined in accordance with ASTM D698.

## 9.4 SURFACE GRADING AND DRAINAGE

Grading across the site and around the perimeter of the buildings is one of the most important factors in minimizing infiltration of surface water into the foundation soils. It is extremely important, particularly in areas where expansive soils are present, that water drains away from the foundations and not be allowed to pond against or near the foundations. Adequate slope of the ground surface is critical. The ground surface immediately adjacent to the building foundations shall be sloped away from the buildings at a slope of not less than 5 percent and, preferably more, for a minimum distance of 10 feet. In addition, small irregularities in the ground surface should be avoided over this 10-foot distance to prevent micro-ponding and subsequent surface water infiltration into the foundation soils. A slope of 2 percent is also recommended beyond this 10-foot distance. Impervious surfaces, such as flatwork or paving, within 10 feet of the building perimeter, should also be sloped not less than 2 percent. The minimum slopes are perpendicular to the perimeter of the foundation and not parallel to it. Slopes that are parallel to the foundation perimeter will distribute water along the foundation instead of removing it and result in surface water infiltration into the foundation soils. Finally, the slopes established on the site grading plan should consider maximum settlements outlined for the building foundation and any backfill placed adjacent to the foundation.

If physical obstructions or lot lines prohibit the 10-foot minimum horizontal distance, a 5 percent slope shall be established to an approved alternative method for diverting water away from the foundation. An approved alternative method would consist of a subsurface drainage system or swale. The subsurface drainage system or swale shall be sloped not less than 2 percent and must continue to divert water away from the foundation. The subsurface drainage system would generally consist of rigid perforated pipe, granular backfill, and a geotextile fabric or poly-liner. Furthermore, the subsurface drainage system would discharge into a sump, and area drain, or a suitable gravity outlet. If a sump is used, it must be equipped with a pump to drain water flowing into the sump. The pump should preferably discharge to an area-wide drainage system located well away from the foundation.

The roof drainage system, i.e., gutters and downspouts, serves to collect water from precipitation to carry it away from the foundation. The downspouts should be tight lined to extend at least 5 feet and, preferably 10 feet, beyond the perimeter of the foundation. This generally consist of connecting the downspouts to piping that will carry water to a sloping final grade at least 5 feet from the foundation or to an underground catchment system at least 10 feet from the foundation. This will reduce the chances of providing a supplemental source of water to the foundation soils and subsequent swelling movements.

## 9.5 VEGETATION CONTROL AND CLEARING PRACTICES

The effect of vegetation on expansive soil movement is dictated by the depth and extent of the root zone and the cracks in the soil that are generated by the growing roots. Trees and large vegetation near the foundation, either removed or planted during construction, cause most foundation problems requiring repair. Trees and large vegetation removed during construction tend to cause heave due to rehydration or increases in soil-moisture. This change in moisture generally occurs over a 5-year period, with approximately 50 percent of the moisture increase occurring over the first year of vegetation removal.

Trees planted within half of their mature height from the edge of the foundation have caused differential foundation movement because the root systems remove large quantities of water from the soil. If trees and large vegetation are planted near the foundation and if sufficient water is not supplied, the foundation soils may shrink resulting in subsidence in the foundation. Significant subsidence distress will usually not occur for 10 to 20 years as the tree matures. During dry periods, enough water should be supplied to trees to minimize shrinkage of expansive soils. Irrigation water should also be applied well away from the foundation to attract the tree roots in that direction. New trees and large vegetation should be planted away from the foundation. The rule of thumbs is that a tree should be at least 1 to 1.5 times its mature height away from the foundation. If trees are planted well away from the foundation in irrigated areas, the chances of subsequent foundation damage will be minimized.

On expansive soils, the main landscaping goal should be to minimize fluctuations in soil water content. Proper surface drainage, plant choices, sprinkling practices, and long-term maintenance are all important. Landscaping practices will have a significant influence on the wetting of the foundation soils. Xeriscape landscaping or landscaping requiring little of no irrigation should be considered within the first 5 to 10 feet of the foundation perimeter. This will eliminate the need for supplemental water from irrigation. Furthermore, sprinkler systems should be directed away from the foundation and should not spray water within 5 feet of the foundation. Landscaping practices must also be careful to maintain positive drainage away from the foundation. Watering should be limited to the minimum needed to maintain the landscaping. Furthermore, landscaping should not trap water against the foundation. Metal edging or other damming devices within 5 feet of the foundation should be avoided.

## 9.6 SHALLOW FOUNDATION RECOMMENDATIONS

DUDLEY strongly recommend the prompt placement of concrete into the footing excavations immediately following completion of digging, cleaning, placement of reinforcing steel, and inspection of the excavation. Precautions should be taken during placement of the reinforcement and concrete to prevent any loose excavated soil from entering the excavation. Any clods of earth that slump into the footing excavation during concrete placement should be promptly removed. DUDLEY should also be contacted if the shallow foundation excavations become impacted by rainfall events that result in weak layers at the base of the excavations.

## 9.7 DRILLED PIER RECOMMENDATIONS

The drilled pier excavations should be checked to ensure that the shaft size, and founding depths specified on the plans have been achieved. Verification of the construction process and the dimensional characteristics of the piers should be performed as part of the project quality assurance (QA) program. The drilled pier excavations should be inspected to ensure that all loose material greater than 3 inches dimension and all standing water over 6 inches depth have been removed prior to placement of the concrete. Precautions should be taken during placement of the reinforcement and concrete to prevent any loose excavated soil from entering the excavation. Prompt placement of concrete into the pier excavation, as soon as the drilling is completed and the excavation cleaned and inspected, is strongly recommended. Under no circumstances should a pier be drilled, that cannot be filled with concrete before the end of the workday or prior to the occurrence of a significant rainfall event. There is always a possibility that groundwater will enter the open pier excavations at the project site and will cause excessive sloughing of the pier excavation sidewalls. Therefore, the contractor should be prepared to use casing or slurry displacement to ensure the integrity of the excavation and to permit pouring of pier concrete in a dry condition.

## 9.8 UTILITY TRENCH PROVISIONS

Provisions should be made to discourage the possibility that utility trenches will serve as pathways for water to migrate from areas outside of the structure area to beneath the structure following completion of construction. We recommend that the bottom of the utility trenches be sloped in a downward direction away from the structure. We also recommend that anti-seep collars be employed along the length of all utility trenches and at the face of the structure to serve as a barrier to moisture migration along the granular soils in the trenches to the interior portions of the structure.

# 10.0 BASIS OF RECOMMENDATIONS

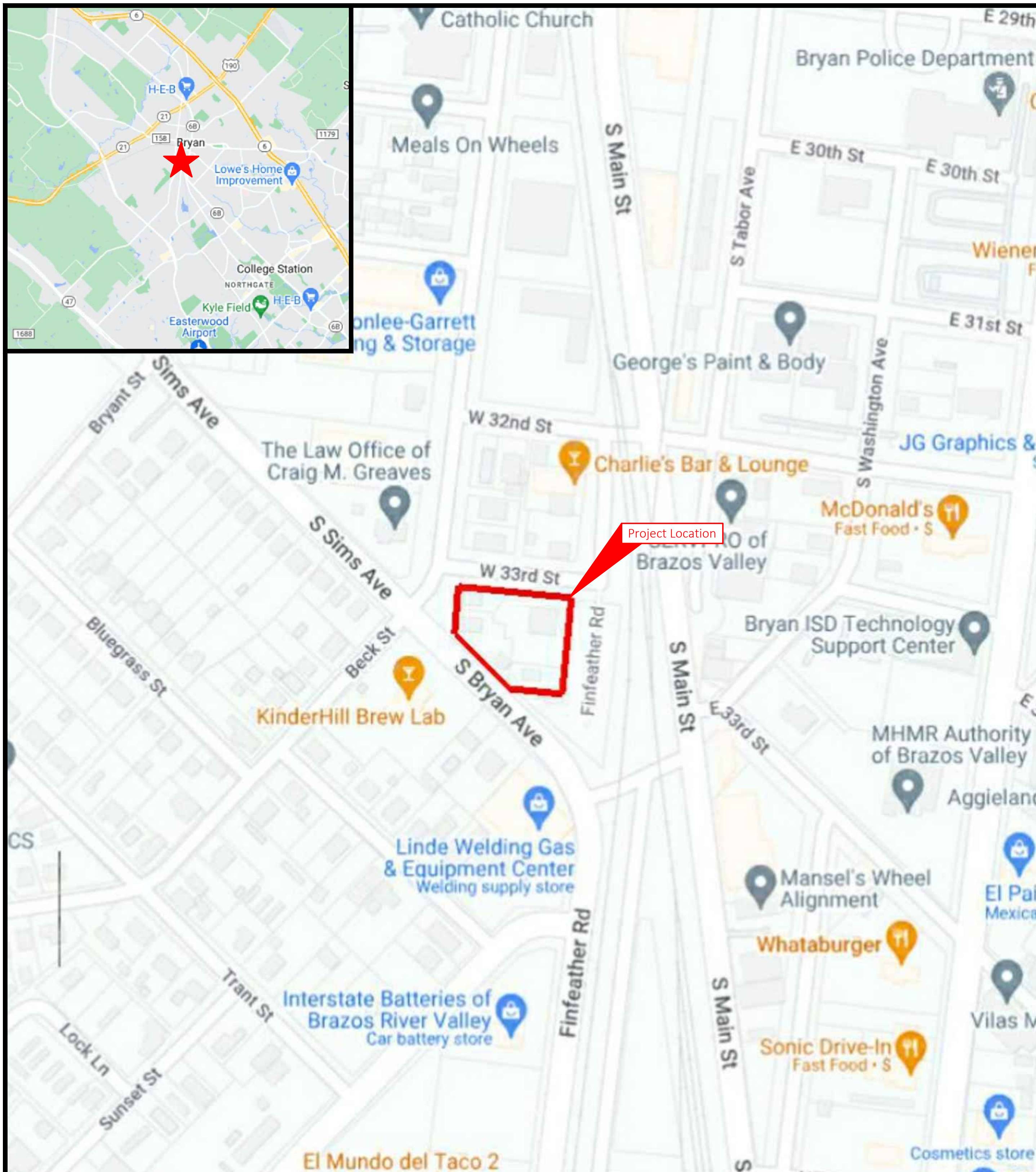
The subsurface information at the site was developed from the subsurface investigation and laboratory testing program and was based upon four (4) widely spaced borings across the project site. The borings were in enough detail and scope to form a reasonable basis for the conceptual planning and design of the foundation system for the proposed Marco Polo buildings described in this report. Recommendations contained in this report were developed based upon a generalization of the subsurface conditions encountered at the boring locations across the site and the assumption that the generalized conditions are continuous throughout the areas under consideration. However, regardless of the thoroughness of a subsurface exploration, there is always a possibility that subsurface conditions encountered over a given area will be different from those present at specific, isolated boring locations. As a result, actual site conditions may be better or worse than the conditions indicated at the boring locations.

DUDLEY warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional engineering practice in the field of geotechnical engineering in this geographic area. No other warranty is implied or expressed.

The information presented in this report was presented for the specific site and the specific structure described in the report. The information should not be employed for the design of other structures or for other projects in the general area of this project without written consent of DUDLEY.

# ATTACHMENT A

**Figures**




Notes:

Scale:

1" = 300' (In Feet)

0 75' 150' 300' 600'

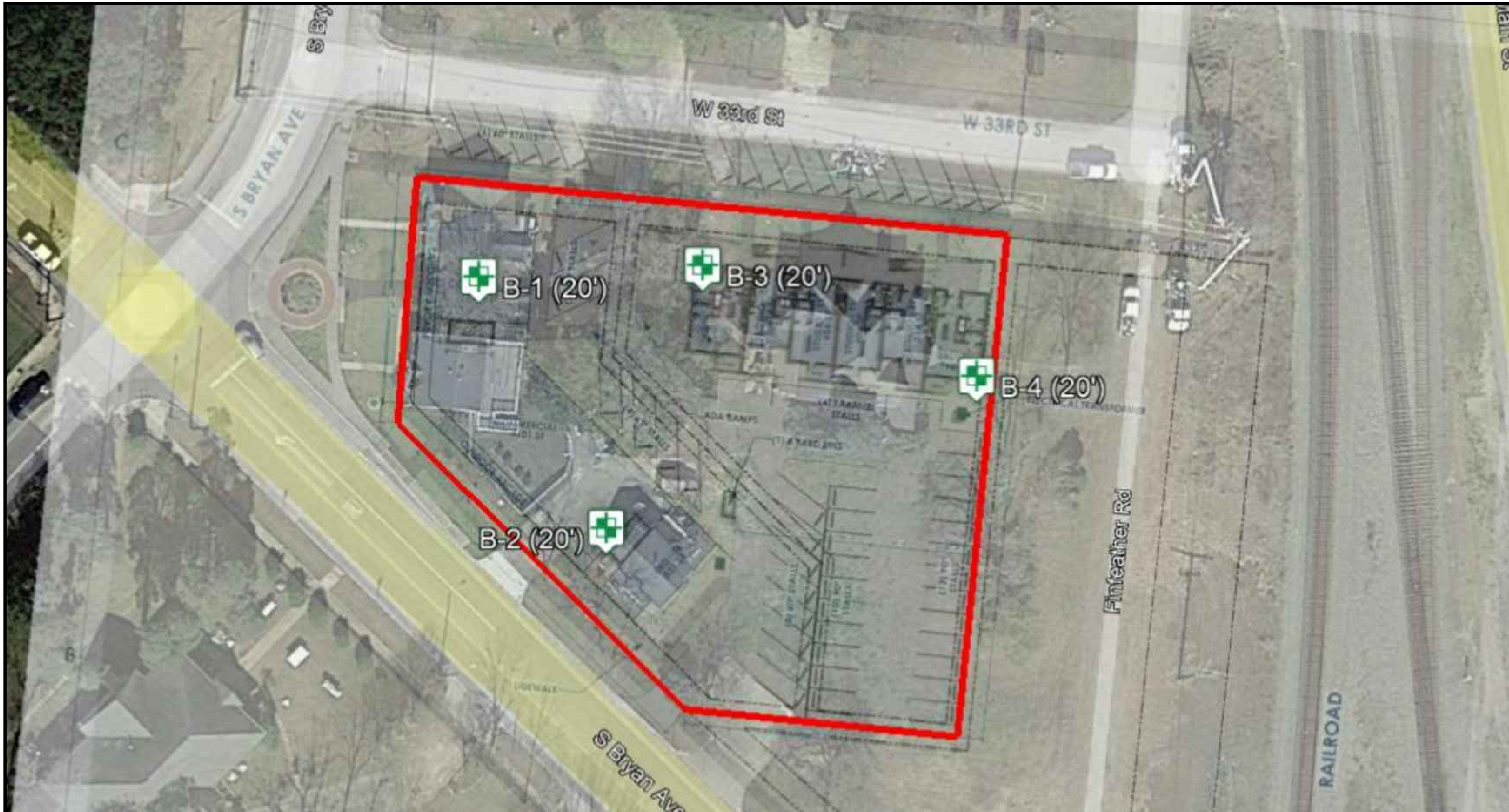
N
W E
S

# Vicinity Map

## Marco Polo Development

Bryan, Texas

| Source: | Drawn By: | Date: | Project No.: | |
|---|---|---|---|---|
| Google Maps (2022) | GTS | 04/27/2022 | 22-00109 | Figure 1 |




Key:

B-1 (20') Boring Location, Designation, & Depth

Scale:

1" = 40' (In Feet)

0 10' 20' 40' 80'

N W E S

Notes:

Plan of Borings
Marco Polo Development
Bryan, Texas

Source: Google Earth (2022), Aerial Date: 12/29/2019

Drawn By: GTS

Date: 04/27/2022

Project No.: 22-00109

Figure 2

# ATTACHMENT B

**Boring Logs**
Log of Borings B-1 through B-4
Boring Log Key

# LOG OF BORING B-1

DUDLEY

Marco Polo Development
101 W 33rd Street
Bryan, Texas

DUDLEY Project No.: 22-00097



Drill Date: 04/20/2022

Location: Reference Figure 2 - Plan of Borings

Surface Elevation: Unknown

| Depth (ft) | USCS Abbreviation | Water Level | MATERIAL DESCRIPTION | Pocket Penetrometer Reading (tsf) | Uncorrected SPT N-Value (blows/ft) | Moisture Content (%) | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) | Passing #40 Sieve (%) | Passing #200 Sieve (%) | Additional Laboratory Tests & Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | Loose, grayish-brown, clayey SAND, dry | | | | | | | | | |
| | | | Stiff to very stiff, gray, fat CLAY, with sand, slightly moist | 4.0 | 3 | | | | | | | |
| | CH | | | 4.0 | 10 | 22.7 | 64 | 22 | 42 | | 72.9 | |
| | | | -light brownish-gray, with decreasing sand content, below 4' | | | | | | | | | |
| 5 | CH | | | 4.0 | 12 | 24.7 | 56 | 21 | 35 | | 91.9 | |
| | | | Medium dense, light brownish-gray, clayey SAND, dry | 4.0 | 19 | | | | | | | |
| | | | | 2.5 | 17 | | | | | | | |
| 10 | | ▼ (after drilling, 11') | | | | | | | | | | |
| | | ∇ (during drilling, 14') | -gray and wet below 13' | | | | | | | | | |
| | | | | | 20 | | | | | | | |
| 15 | | | | | | | | | | | | |
| | | | Very stiff to hard, gray to light brownish-gray, fat CLAY, slightly moist | 4.5+ | 18 | | | | | | | |
| 20 | | | Boring terminated at 20' | | | | | | | | | |
| 25 | | | | | | | | | | | | |

Sample Key: Split-Spoon (SPT); Shelby Tube; Disturbed; Texas Cone (TCP); Rock Core; No Recovery

Water Level Symbols: ∇ During Drilling; ▼ After Drilling

Water Level Observations Summary: Water at 14' during drilling. Water at 11' after drilling.

Notes:
1. Borehole drilled with CME 550.
2. Drilling Method - 3.5" Ø Continuous Flight Augers.
3. Automatic SPT hammer.

# DUDLEY

# LOG OF BORING B-2

Marco Polo Development
101 W 33rd Street
Bryan, Texas

DUDLEY Project No.: 22-00097



Drill Date: 04/20/2022

Location: Reference Figure 2 - Plan of Borings

Surface Elevation: Unknown

| Depth (ft) | Sample | USCS Abbreviation | USCS Symbol | Water Level | MATERIAL DESCRIPTION | Pocket Penetrometer Reading (tsf) | Uncorrected SPT N-Value (blows/ft) | Moisture Content (%) | Liquid Limit LL | Plastic Limit PL | Plasticity Index PI | Passing #40 Sieve (%) | Passing #200 Sieve (%) | Additional Laboratory Tests & Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | Loose, grayish-brown, clayey SAND, dry | | | | | | | | | |
| | | | | | Stiff to very stiff, dark gray, fat CLAY, with sand, slightly moist | 2.5 | 3 | | | | | | | |
| | | CH | | | | 2.5 | 5 | 29.7 | 70 | 23 | 47 | | 84.0 | |
| | | | | | -light brownish-gray below 4' | | | | | | | | | |
| 5 | | CH | | | | 4.5+ | 8 | 29.6 | 65 | 22 | 43 | | 88.5 | |
| | | | | | Very stiff to hard, light brownish-gray, sandy, lean CLAY, slightly moist | 4.5+ | 19 | | | | | | | |
| | | | | | Dense, light gray, clayey SAND, dry | 4.0 | 33 | | | | | | | |
| 10 | | | | | | | | | | | | | | |
| | | | | | Very stiff to hard, light gray to light brownish-gray, fat CLAY, slightly moist | 4.5+ | 19 | | | | | | | |
| 15 | | | | | | | | | | | | | | |
| | | | | | | 4.5+ | 16 | | | | | | | |
| 20 | | | | | Boring terminated at 20' | | | | | | | | | |
| 25 | | | | | | | | | | | | | | |

Field Strength Data: Pocket Penetrometer Reading (tsf) 1.0 2.0 3.0 4.0; Uncorrected SPT N-Value (blows/ft) 20 40 60 80

Natural Moisture Content & Atterberg Limits: Plastic Limit, Moisture Content, Liquid Limit; 20 40 60 80

Sample Key: Split-Spoon (SPT), Shelby Tube, Disturbed, Texas Cone (TCP), Rock Core, No Recovery

Water Level Symbols: During Drilling, After Drilling

Water Level Observations Summary: Borehole dry during and after drilling.

Notes:
1. Borehole drilled with CME 550.
2. Drilling Method - 3.5" Ø Continuous Flight Augers.
3. Automatic SPT hammer.

# LOG OF BORING B-3

DUDLEY

Marco Polo Development
101 W 33rd Street
Bryan, Texas

DUDLEY Project No.: 22-00097



Drill Date: 04/20/2022

Location: Reference Figure 2 - Plan of Borings

Surface Elevation: Unknown

| Depth (ft) | USCS Abbreviation | MATERIAL DESCRIPTION | Pocket Penetrometer Reading (tsf) | Uncorrected SPT N-Value (blows/ft) | Moisture Content (%) | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) | Passing #40 Sieve (%) | Passing #200 Sieve (%) | Additional Laboratory Tests & Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | Loose, brown, clayey SAND, dry | | | | | | | | | |
| | CH | Stiff to very stiff, dark gray, sandy, fat CLAY, slightly moist | 3.0 | 2 | 20.7 | 53 | 20 | 33 | | 67.5 | |
| | | | 4.5+ | 8 | | | | | | | |
| 5 | | Very stiff to hard, light brownish-gray, sandy, lean CLAY, dry | 4.5+ | 12 | | | | | | | |
| | | Medium dense to dense, light brownish-gray, clayey SAND, slightly moist | | 28 | | | | | | | |
| | SC | | | 35 | 17.5 | 26 | 15 | 11 | | 17.7 | |
| 10 | | | | | | | | | | | |
| | | Very stiff to hard, light brownish-gray, fat CLAY, slightly moist | 4.5+ | 15 | | | | | | | |
| 15 | | | | | | | | | | | |
| | | | 4.5 | 16 | | | | | | | |
| 20 | | Boring terminated at 20' | | | | | | | | | |
| 25 | | | | | | | | | | | |

Field Strength Data: Pocket Penetrometer Reading (tsf) 1.0 2.0 3.0 4.0; Uncorrected SPT N-Value (blows/ft) 20 40 60 80

Natural Moisture Content & Atterberg Limits: Plastic Limit, Moisture Content, Liquid Limit — 20 40 60 80

Sample Key: Split-Spoon (SPT), Shelby Tube, Disturbed, Texas Cone (TCP), Rock Core, No Recovery

Water Level Symbols: During Drilling, After Drilling

Water Level Observations Summary: Borehole dry during and after drilling.

Notes:
1. Borehole drilled with CME 550.
2. Drilling Method - 3.5" Ø Continuous Flight Augers.
3. Automatic SPT hammer.

# LOG OF BORING B-4

Marco Polo Development
101 W 33rd Street
Bryan, Texas

DUDLEY Project No.: 22-00097



Drill Date: 04/20/2022

Location: Reference Figure 2 - Plan of Borings

Surface Elevation: Unknown

| Depth (ft) | USCS Abbreviation | MATERIAL DESCRIPTION | Pocket Penetrometer Reading (tsf) | Uncorrected SPT N-Value (blows/ft) | Moisture Content (%) | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) | Passing #40 Sieve (%) | Passing #200 Sieve (%) | Additional Laboratory Tests & Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | Loose, dark gray, silty SAND, dry | | | | | | | | | |
| | | Firm to stiff, dark gray, fat CLAY, with sand, moist | 1.5 | 2 | | | | | | | |
| | CL | Stiff to very stiff, dark gray, sandy, lean CLAY, slightly moist | 4.5+ | 9 | 20.3 | 40 | 17 | 23 | | 52.9 | |
| 5 | SC | Medium dense, gray to light brownish-gray, clayey SAND, slightly moist | 4.5+ | 11 | 16.9 | 34 | 16 | 18 | | 43.1 | |
| | | | 3.5 | 10 | | | | | | | |
| | | Stiff to very stiff, light brownish-gray, fat CLAY, with orange staining, slightly moist | 3.0 | 5 | | | | | | | |
| 10 | | | | | | | | | | | |
| | | -grayish-brown below 13' | 4.5+ | 12 | | | | | | | |
| 15 | | | | | | | | | | | |
| | | Medium dense to dense, light gray to light brownish-gray, clayey SAND, wet | 4.0 | 26 | | | | | | | |
| 20 | | Boring terminated at 20' | | | | | | | | | |
| 25 | | | | | | | | | | | |

Field Strength Data: Pocket Penetrometer Reading (tsf) 1.0, 2.0, 3.0, 4.0; Uncorrected SPT N-Value (blows/ft) 20, 40, 60, 80

Natural Moisture Content & Atterberg Limits: Plastic Limit, Moisture Content, Liquid Limit; 20, 40, 60, 80

Sample Key: Split-Spoon (SPT), Shelby Tube, Disturbed, Texas Cone (TCP), Rock Core, No Recovery

Water Level Symbols: During Drilling, After Drilling

Water Level Observations Summary: Water at 11' during drilling. Water at 10' after drilling.

Notes:
1. Borehole drilled with CME 550.
2. Drilling Method - 3.5" Ø Continuous Flight Augers.
3. Automatic SPT hammer.

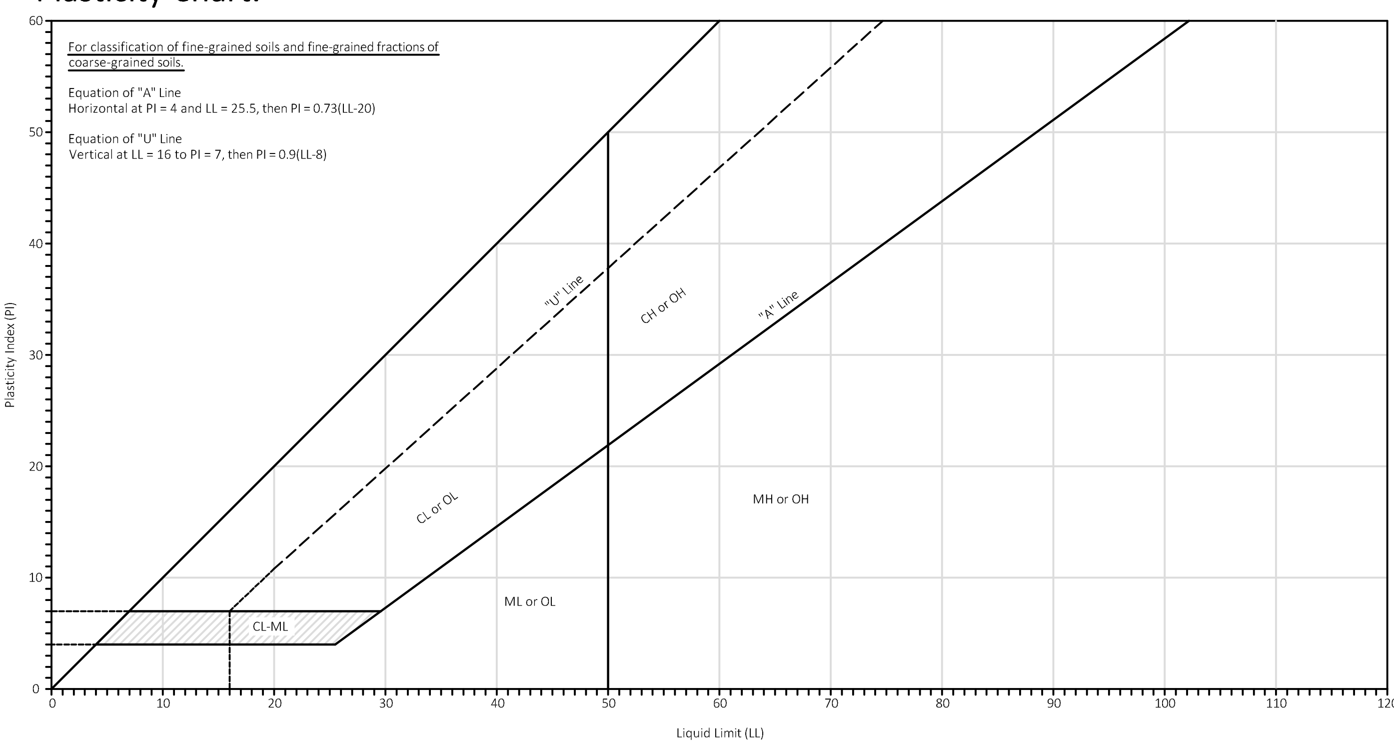
# BORING LOG KEY (SHEET 1 OF 2)

## Unified Soil Classification System (USCS):

| Major Classification Group | Sub-Group | | Group Hatch | Group Symbol | Group Name | Laboratory Classification Criteria |
|---|---|---|---|---|---|---|
| Coarse-Grained Soils (> 50% retained on the #200 sieve) | Gravels (> 50% of the coarse fraction is retained on the #4 sieve) | Clean gravels | | GW | Well-graded gravel | $C_u \geq 4$ and $1 \leq C_c \leq 3$ |
| | | | | GP | Poorly-graded gravel | $C_u < 4$ and/or $1 < C_c$ or $C_c > 3$ |
| | | Gravels with fines | | GM | Silty gravel | Fine classify as ML or MH |
| | | | | GC | Clayey gravel | Fine classify as CL of CH |
| | Sands (≥ 50% of the coarse fraction passes the #4 sieve) | Clean sands | | SW | Well-graded sand | $C_u \geq 6$ and $1 \leq C_c \leq 3$ |
| | | | | SP | Poorly graded sand | $C_u < 6$ and/or $1 < C_c$ or $C_c > 3$ |
| | | Clean gravels | | SM | Silty sand | Fine classify as ML or MH |
| | | | | SC | Clayey sand | Fine classify as CL of CH |
| Fine-Grained Soils (≥ 50% passes the #200 sieve) | Silts and clays with liquid limit < 50% | | | CL | Low plasticity or lean clay | PI > 7 and plots on or above "A" line (Must fall outside CL-ML hatched area) |
| | | | | ML | Low plasticity silt | PI < 4 and plots below "A" line (Must fall outside CL-ML hatched area) |
| | | | | OL | Low plasticity organic silt or clay | LL ratio < 0.75 |
| | Silts and clays with liquid limit ≥ 50% | | | CH | High plasticity or fat clay | PI plots on or above "A" line |
| | | | | MH | High plasticity elastic silt | PI plots below "A" line |
| | | | | OH | High plasticity organic silt, organic clay | LL ratio < 0.75 |
| Highly organic soils | | | | Pt | Peat | |

Prefix: G = Gravel, S = Sand, M = Silt, C = Clay, O = Organic
Suffix: W = Well-Graded, P = Poorly-Graded, M = Silty, L = Clay with LL<50%, H = Clay with LL≥50%
Ref. ASTM D2487 for dual symbol and modifier requirements.

## Plasticity Chart:



# BORING LOG KEY (SHEET 2 OF 2)

## Gradation:

| Particle Description | Particle Size (mm) | Retaining U.S. Sieve Size |
|---|---|---|
| Boulder | 75 - 300 | 12" |
| Cobble | 75 - 300 | 3" |
| Coarse Gravel | 19 - 75 | 3/4" |
| Fine Sand | 4.75 - 19 | No. 4 |
| Coarse Sand | 2.0 - 4.75 | No. 10 |
| Medium Sand | 0.42 - 2.0 | No. 40 |
| Fine Sand | 0.075 - 0.42 | No. 200 |
| Silt | 0.002 - 0.075 | Hydrometer Analysis Required |
| Clay | < 0.002 | |

## Volumetric Stability:

| Liquid Limit (LL) | Swell Potential Classification |
|---|---|
| < 20 | None/Very Low |
| 20 - 35 | Low |
| 35 - 50 | Moderate |
| 50 - 70 | High |
| 70 - 90 | Very High |
| > 90 | Extremely High |

## Cohesionless Soil Strength:

| Sands and Gravels | | | | |
|---|---|---|---|---|
| Relative Density (Description) | $N_{SPT}$ (blows/ft) | $N_{TCP}$ (blows/ft) | Friction Angle (degrees) | Relative Density (%) |
| Very Loose | 0 - 4 | 0 - 8 | < 30 | 0 - 20 |
| Loose | 4 - 10 | 8 - 20 | 30 - 32 | 20 - 40 |
| Medium Dense | 10 - 30 | 20 - 60 | 32 - 37 | 40 - 70 |
| Dense | 30 - 50 | 60 - 100 | 37 - 42 | 70 - 90 |
| Very Dense | > 50 | > 100 | > 42 | 90 - 100 |

## Cohesive Soil Strength:

| Silts and Clays | | | |
|---|---|---|---|
| Consistency | $N_{SPT}$ (blows/ft) | $N_{TCP}$ (blows/ft) | Unconfined Compressive Strength (tsf) |
| Very Soft | 0 - 2 | 0 - 3 | < 0.25 |
| Soft | 2 - 4 | 3 - 6 | 0.25 - 0.5 |
| Firm | 4 - 8 | 6 - 11 | 0.5 - 1.0 |
| Stiff | 8 - 16 | 11 - 22 | 1.0 - 2.0 |
| Very Stiff | 16 - 32 | 22 - 45 | 2.0 - 4.0 |
| Hard | > 32 | > 45 | > 4.0 |

## Dynamic Cone Penetrometer:

| Penetration Range (mm/blow) | Penetration Range (in/blow) | Approximate California Bearing Ratio (CBR) | Approximate Mid-Range k-value (psi/in) |
|---|---|---|---|
| < 4 | < 0.16 | > 70 | > 450 |
| 4 - 5 | 0.16 - 0.20 | 50 - 70 | 400 - 450 |
| 5 - 8 | 0.20 - 0.31 | 30 - 50 | 350 - 400 |
| 8 - 14 | 0.31 - 0.55 | 15 - 30 | 250 - 350 |
| 14 - 19 | 0.55 - 0.75 | 10 - 15 | 200 - 250 |
| 19 - 25 | 0.75 - 0.98 | 7 - 10 | 150 - 200 |
| 25 - 30 | 0.98 - 1.18 | 3 - 7 | 80 - 150 |
| 30 - 35 | 1.18 - 1.38 | 1 - 3 | < 80 |
| > 35 | > 1.38 | < 1 | < 50 |

## Sulfate Induced Heave Risk:

| Risk | Soluble Sulfate Concentration (PPM) |
|---|---|
| Low | < 3,000 |
| Moderate | 3,000 - 5,000 |
| Moderate to High | 5,000 - 8,000 |
| High to Unacceptable | > 8,000 |
| Unacceptable | > 11,000 |

## Rock Strength:

| Hardness | $N_{SPT}$ (blows/ft) | $N_{TCP}$ (in./100 blows) | Unconfined Compressive Strength (psi) |
|---|---|---|---|
| Very Soft | 1 - 30 | > 6 in./100 | < 4,000 |
| Soft | 30 - 50 | 4 in. - 6 in./100 | 4,000 - 8,000 |
| Hard | 50 - 100 | 1 in. - 5 in./100 | 8,000 - 16,000 |
| Very Hard | 1 - 30 | 0 in. - 2 in./100 | 16,000 - 32,000 |
| Extremely Hard | No Penetration | No Penetration | > 32,000 |

## Rock Quality:

| Rock Description | Rock Quality Designation (RQD) |
|---|---|
| Very Poor | < 0.25 |
| Poor | 0.25 - 0.50 |
| Fair | 0.50 - 0.75 |
| Good | 0.75 - 0.90 |
| Excellent | > 0.90 |

## Moisture:

| Description | Meaning |
|---|---|
| Dry | No water evident in sample. Moisture content less than plastic limit. |
| Moist | Sample feels damp. Moisture content at or slightly above plastic limit. |
| Very Moist | Water visible on sample. Moisture content between plastic limit and liquid limit. |
| Wet | Sample contains free water. |