GEOTECHNICAL INVESTIGATION
RESIDENTIAL DEVELOPMENT

1975 CAMBRIANNA DRIVE
SAN JOSE, CALIFORNIA

JANUARY 28, 2022
PROJECT PA21.1017.00

SUBMITTED TO:

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1 INTRODUCTION

This report presents the results of our geotechnical study for the proposed residential development at 1975 Cambrianna Drive in San Jose, California. The project site is the eastern approximately 2.7-acre portion of a larger property identified with Assessor Parcel Number (APN) 984-45-006. The project site is referenced as the "project," "site," or "project area" in this report. The approximate location of the project site is shown on the Vicinity Map included with Figures 1 and 2 of this report. Figure 1 shows a layout of the site's existing conditions. Figure 2 shows a layout of the site's proposed development.

This report presents our findings, conclusions, and geotechnical recommendations for design and construction of the project. These findings, conclusions, and recommendations are based on information collected and reviewed during this investigation. The conclusions and recommendations in this report should not be extrapolated to other areas or used for other projects without our review.

1.1 Project Description

The project area is currently vacant. The proposed residential development will include single-family and duet units consisting of two-story structures, and some lots may include an auxiliary dwelling unit (ADU). Associated site improvements will include underground utilities, landscaping, exterior flatwork, driveways, and on-site streets. No swimming pools or basements are planned. Retaining walls, if required, are expected for landscaping purposes and up to about 3 feet in height.

An approximately 4,000-square-foot area in the northwestern portion of the site is designated for stormwater treatment and retention. Details of the stormwater treatment and retention system have not been finalized; and based on our experience on past Robson Homes projects, the system is expected to consist of underground storage vaults with invert elevation at about 10 to 15 feet below ground surface.

Preliminary grading information is not available when we prepared this report. Because the site is essentially flat-lying, site grading is anticipated to involve cuts and fills of about 1 to 4 feet thick to construct the building pads and to achieve design grades. Deeper excavations will be necessary for underground utilities and stormwater retention vaults.

The above project descriptions are based on information provided to us. If the actual project differs from those described above, Geo-Logic Associates (GLA) should be contacted to review our findings, conclusions, and recommendations and present any necessary modifications to address the different project development schemes.

1.2 Information Provided

For this investigation, Robson Homes provided us with the following:

1. A drawing titled "Site Plan," prepared by Civil Engineering Associates, dated 7/22/2021.

2. An exhibit showing the 1975 Cambrianna Avenue property with the project area identified.

1.3 Purpose and Scope of Services

The purpose of this geotechnical study was to explore subsurface conditions at the project site and to provide geotechnical recommendations for design and construction of the proposed improvements. The following work was performed.

1. Performed a site reconnaissance to observe site surface conditions and to mark locations of our exploration.

2. Reviewed available geologic and geotechnical information pertinent to the site.

3. Notified Underground Service Alert (USA) for underground utility clearance and coordinated our drilling with Robson Homes.

4. Subcontracted with a private underground locator to check the proposed exploration locations for presence of underground utilities.

5. Explored subsurface conditions by means of six exploratory drill holes to depths between approximately 20 and 45 feet below ground surface.

6. Collected a bulk sample of the near-surface soil.

7. Performed laboratory tests on selected soil samples from the drill holes and on the bulk sample to measure pertinent engineering properties of the samples.

8. Performed engineering analysis on the field and laboratory data.

9. Prepared this geotechnical investigation report.

2 SITE INVESTIGATION

This study consists of a site reconnaissance and a subsurface exploration program. The site reconnaissance was to observe existing site surface conditions. The subsurface exploration program was to explore earth conditions at the project site. The observed surface and subsurface site conditions are discussed in Section 3 of this report.

2.1 Subsurface Exploration

Our subsurface exploration program involved drilling of six exploratory drill holes (DH-1 through DH-6) on August 9, 2021. The exploratory drill holes were located in the field by referencing to existing site features and pacing; therefore, their locations are approximate. The approximate locations of the drill holes are shown on Figures 1 and 2. The drill holes were backfilled with cement grout after completion of drilling.

The six exploratory drill holes were advanced using a track-mounted CME 55 drilling rig equipped with 8-inch diameter hollow-stem augers. The depth of exploration ranged between approximately 20 and 45 feet below ground surface (bgs). In the field, our personnel visually classified the materials encountered and maintained a log of each drill hole.

Soil samples were obtained using a 2-inch outside diameter (O.D.; 1.4-inch inside diameter, I.D.) split-barrel sampler (also called a Standard Penetration Test sampler) and a 3-inch O.D. (2½-inch I.D.) split-barrel sampler. Soil samples were obtained by driving the sampler up to 18 inches into the earth material using a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler was recorded for each 6-inch penetration interval. The number of blows required to drive the sampler the last 12 inches, or the penetration interval indicated on the log when harder material was encountered, is shown as blows per foot (blow count) on the drill hole logs. The CME 55 rig is equipped with an automatic trip hammer.

In the field, our personnel visually classified the materials encountered and maintained a log of each drill hole. Visual classification of soils encountered in our drill holes was made in general accordance with the Unified Soil Classification System (ASTM D 2487 and D 2488). The results of our laboratory tests were used to refine our field classifications. Two Keys to Soil Classification, one for fine grained soils and one for coarse grained soils, are included in Appendix A, together with the logs of these drill holes.

2.2 Laboratory Testing

Geotechnical laboratory testing was conducted on selected soil samples collected from our drill holes. These tests included moisture content, dry density, Atterberg limits, sieve analysis, and percentage passing a No. 200 sieve. An R-value test was performed on the bulk sample collected from the site. The laboratory test results are presented on the drill hole logs at the corresponding sample depths. Graphic presentations of the results of the Atterberg limits, sieve analysis, and R-value tests are presented on separate sheets in Appendix B

In addition to geotechnical testing, two selected soil samples were sent to CERCO Analytical for corrosivity analysis. A brief report from CERCO Analytical with the corrosivity test results is included in Appendix B.

3 FINDINGS

3.1 Surface Conditions

The project site is approximately the eastern 2.7 acres of a larger parcel currently owned by the Cambrian School District at 1975 Cambrianna Drive, San Jose, California. The project site is bordered on the east by houses and Taper Avenue, on the south by Cambrianna Drive, on the west by existing buildings and parking lot, and on the north by houses and Browning Avenue.

The project site is occupied by a lawn except in the southern portion where it is a gravel parking lot. There are three existing trees. Ground surface across the site is essentially flat-lying, with a gentle down gradient from south to north.

3.2 Subsurface Conditions

Subsurface soils encountered in our drill holes can be generalized as alluvial soils. In DH-1, a pavement section consisting of roughly 2 inches of base rock over roughly 1.5 inches of asphalt concrete over roughly 3 inches of base rock was encountered at the surface. Below the pavement section is a layer of stiff sandy silty clay with low plasticity to about 4 feet below ground surface (bgs). This silty clay is underlain by hard sandy lean clay with gravel to about 8 feet bgs, dense clayey sand with gravel to about 13.5 feet bgs, and dense to very dense well graded sand with clay and gravel to the maximum explored depth of about 20 feet bgs.

In DH-2, a pavement section similar to that in DH-1 was encountered at the ground surface. Below the pavement section are stiff to very stiff sandy silty clay with gravel of low plasticity to about 4 feet bgs, very stiff to hard sandy lean clay with gravel to about 8.5 feet bgs, dense clayey gravel with sand to about 13.5 feet bgs, and dense to very dense well graded sand with clay and gravel to the maximum explored depth of about 19.5 feet bgs.

In DH-3, a layer of stiff to very stiff sandy silty clay with low plasticity was encountered to about 4 feet bgs. This silty clay is underlain by very dense clayey sand with gravel to about 8 feet bgs, very dense poorly graded gravel with clay and sand to about 13.5 feet bgs, and well graded sand with clay and gravel to the maximum explored depth of about 20 feet bgs.

In DH-4, a layer of firm sandy silty clay with low plasticity was encountered to about 4 feet bgs. This silty clay is underlain by very stiff to hard lean clay to about 8 feet bgs, very dense well graded sand with clay and gravel to about 19 feet bgs, dense to very dense clayey sand with gravel to about 29 feet bgs, and dense to very dense poorly graded sand with clay and gravel to the maximum explored depth of about 45 feet bgs.

In DH-5, a layer of very stiff to hard sandy silty clay with low plasticity was encountered to about 4 feet bgs. This silty clay is underlain by dense clayey sand with gravel to about 14 feet bgs, very dense poorly graded sand with clay and gravel to about 17 feet bgs, and very dense poorly graded gravel with clay and sand to the maximum explored depth of about 20 feet bgs.

In DH-6, a layer of hard sandy silty clay with low plasticity was encountered to about 4 feet bgs. This silty clay is underlain by dense clayey sand with variable amounts of gravel to about 14 feet bgs, very dense poorly graded sand with clay and gravel to about 18 feet bgs, and very dense poorly graded gravel with clay and sand to the maximum explored depth of about 20 feet bgs.

3.3 Groundwater

Groundwater was not encountered in any of our six drill holes for this study, the deepest of which extended to a depth of about 45 feet bgs.

Historical high groundwater at the project site was estimated at about 48 feet based on our review of Plate 1.2, "Depth to historically highest ground water, historical liquefaction sites, and locations of boreholes, San Jose West 7.5-minute Quadrangle, California," Seismic Hazard Zone Report 058, prepared by California Geological Survey, Department of Conservation, 2002.

It should be noted that fluctuations in the groundwater level may occur due to seasonal variations in rainfall and temperature, pumping from wells, regional groundwater recharge program, irrigation, or other factors that were not evident at the time of our investigation.

3.4 Variations in Subsurface Conditions

Our interpretations of soil and groundwater conditions, as described in this report, are based on information obtained from drill holes and laboratory testing for this study. Our conclusions and recommendations are based on these interpretations. Please realize the site has undergone different phases of development and grading. Therefore, it is likely that undisclosed variations in subsurface conditions exist at the site, particularly old foundations, abandoned utilities and localized areas of deep and loose fill.

Careful observations should be made during construction to verify our interpretations. Should variations from our interpretations be found, we should be notified to evaluate whether any revisions should be made to our recommendations.

4 SEISMIC CONSIDERATIONS

4.1 Earthquake Faulting

The Greater San Francisco Bay Area is seismically dominated by the active San Andreas Fault system, the tectonic boundary between the northward moving Pacific Plate (west of the fault) and the North American Plate (east of the fault). This movement is distributed across a complex system of generally strike-slip, right-lateral, and subparallel faults.

Potential sources of significant earthquake ground shaking at the site include several active and potentially active faults in the San Francisco Bay area, as well as faults farther afield. The faults were first compiled on the State's Fault Activity Map (Jennings, 1974; Jennings and Bryant, 2010). This map has now been integrated into the US Geological Survey's Quaternary Fault and Fold Database and made available as a .kmz "drape" over Google Earth terrain files.

The distance to a seismic source (fault) is defined by the NGA relationships as the closest distance to the seismogenic zone, be it in the subsurface or at the surface; distances may therefore differ from distances measured on the ground surface. The distances shown on the table below are for reference only, as they are horizontal distances from the site to the surface trace of the seismic source, and not necessarily the closest distance to a (dipping) seismogenic zone. These distances were measured using the US Geological Survey's Quaternary Fault and Fold Database, with major faults listed in approximate order of distance from the site; not all sources are listed in the summary table below.

Fault Name	Approximate Distance	Orientation from Site
Monte Vista-Shannon	3 km	Southwest
San Andreas	11 km	Southwest
Sargent	14 km	South/Southeast
Hayward (southeast extension)	16 km	Northeast
Calaveras (central section)	19 km	Northeast
San Gregorio	35½ km	Southwest

4.2 Ground Accelerations

According to the 2019 California Building Code (CBC) and American Society of Civil Engineers (ASCE) Standard 7-16, the spectral response acceleration at any period can be taken as the lesser of the spectral response accelerations from the probabilistic and deterministic ground motion approaches. The U.S. Seismic Design Maps tool available at the Structural Engineers Association of California (SEAOC) website was used for this purpose to retrieve seismic design parameter values for design of buildings at the subject site. Two levels of ground motions are considered in the Application: Risk-targeted Maximum Considered Earthquake (MCE_R) and Design Earthquake (DE), with both probabilistic and deterministic values defined in terms of maximum-direction rather than geometric-mean, horizontal spectral acceleration (S_a). The probabilistic MCE_R

spectral response accelerations are represented by a 5 percent damped acceleration response spectrum having a 1 percent probability of collapse within a 50-year period and in the direction of the maximum horizontal response. The probabilistic Design Earthquake (DE) S_a value at any period can be taken as two-thirds of the MCE_R S_a value at the same period.

Using the U.S. Seismic Design Maps tool at the SEAOC website, a Site Class C, and the latitude and longitude of the site (latitude 37.265403º N, longitude -121.928864º W), the calculated geometric mean peak ground accelerations adjusted for site class effects (PGA_M) for the MCE_G (Geometric Mean Maximum Considered Earthquake) is 0.945g. A Site Class C was selected based on regional USGS data.

4.3 Seismicity

The Working Group on California Earthquake Probabilities' (WGCEP) estimates of the probabilities of major earthquakes are now in their sixth iteration, with the greatest changes in approach being the inclusion of multifold rupture scenarios, in the progressive consideration of more potential seismic sources, the possibility of earthquakes on unrecognized faults, and the inclusion of the notion of fault "readiness". Current estimates (WGCEP, 2014) for the San Francisco region indicate a 72% probability of a large (magnitude 6.7 or greater) earthquake in the San Francisco Bay area as a whole over the 30-year period beginning in 2014; this overall probability is greater than the previous (WGCEP, 2007) probability of 63%, due mainly to the inclusion of multi-fault rupture scenarios. The estimate for the Calaveras fault alone is 14.4% (revised up from the 7% presented by WGCEP, 2007); for the (northern) San Andreas fault alone, 27.4% (revised upward from the WGCEP (2007) value of 21%); and for the Hayward fault, 45.3% (revised upward from the WGCEP (2007) value of 31%).

4.4 Liquefaction

Soil liquefaction is a phenomenon in which saturated granular soils, and certain fine-grained soils, lose their strength due to the build-up of excess pore water pressure during cyclic loading, such as that induced by earthquakes. Soils most susceptible to liquefaction are saturated, clean, loose, fine-grained sands and non-plastic silts. Certain gravels, plastic silts, and clays are also susceptible to liquefaction. The primary factors affecting soil liquefaction include: 1) intensity and duration of seismic shaking; 2) soil type; 3) relative density of granular soils; 4) moisture content and plasticity of fine-grained soils; 5) overburden pressure; and 6) depth to ground water.

The project site is not located in a California Geologic Survey (CGS) Earthquake Zones of Required Investigation for liquefaction nor in a Santa Clara County liquefaction hazard zone (County of Santa Clara, October 26, 2012).

The granular soils encountered in our borings consist predominantly of dense to very dense sands and gravels. Groundwater was not encountered in our deepest boring which was 45 feet in depth. Historical high groundwater was about 48 feet bgs. Therefore, in our opinion, the potential for liquefaction of the granular soils encountered in our drill holes is low because of the dense to very dense relative density of the soils and the deep groundwater level.

4.5 Seismic Design Parameters

Design of the proposed structures should comply with design for structures located in seismically active areas. Structures should be designed in accordance with the requirements of governing jurisdictions and applicable building codes. GLA evaluated ASCE 7-16 seismic design parameters for the site using the SEAOC U.S. Design Maps application. The table below lists the seismic design parameters for the site. Note that, because the Mapped Spectral Acceleration at 1.0-second Period (S_1) value for the site is larger than 0.2 g, a site response analysis may be required, in accordance with Section 11.4.8 of ASCE 7-16.

Seismic Design Parameter	Value
Site Class	C
Site Coefficient, F_a	1.2
Site Coefficient, F_v	1.4
Mapped Spectral Acceleration at 0.2-second Period, S_s	1.912g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.68g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	2.295g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.952g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.53g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.635g
Long-period Transition Period, T_L	12 sec.

5 CONCLUSIONS AND DISCUSSION

Based on our geotechnical evaluation, it is our opinion the project site may be developed as discussed in this report, provided our geotechnical recommendations are incorporated in the design and construction of the project. Our opinions, conclusions, and recommendations are based on our understanding of the proposed development, data review, properties of soils encountered in subsurface exploration, laboratory test results, and engineering analyses. Geotechnical considerations for this project are discussed below.

5.1 Ground Rupture

The project site is not located in an Alquist-Priolo Earthquake Fault Zone. Because no active or potentially active faults are known to cross the site, it is reasonable to conclude the risk of fault rupture through the project site is low.

5.2 Seismic Shaking

The project site is located in an area of high seismicity. Based on general knowledge of the site seismicity, it should be anticipated that, during their useful life, the proposed structures will be subject to at least one severe earthquake (magnitude 7 to 8+) that could cause considerable ground shaking at the site. It is also anticipated that the site will periodically experience small to moderate magnitude earthquakes.

5.3 Expansion Potential of Surficial Soils

The surficial soils encountered in our drill holes consist generally of silty clay with low plasticity, which generally corresponds to a low expansion potential. Therefore, soil expansion should not be a concern at this site.

6 GEOTECHNICAL RECOMMENDATIONS

6.1 Earthwork

6.1.1 Site Preparation, Clearing and Stripping

Prior to grading, construction areas should be cleared of obstructions, deleterious materials, abandoned or designated utility lines, existing pavements, designated trees, and other below grade obstacles encountered during the clearing operation. Tree stumps should be grubbed. Roots with diameter of about 1 inch or larger or length of about 3 feet or longer should be removed. Depressions, excavations, and holes that extend below the planned finish grades should be cleaned and backfilled with engineered fill compacted to the requirements given under the section of "Engineered Fill Placement and Compaction."

After clearing, the site should be stripped to sufficient depth to remove vegetation and organic-laden topsoil. Stripped material may be stockpiled for use in landscape areas if approved by the project landscape architect; otherwise, it should be removed from the site. For planning purposes, an estimated stripping depth of 3 to 6 inches may be assumed in unpaved areas. The actual stripping depth should be determined in the field by the Geotechnical Engineer at the time of construction.

6.1.2 Excavation, Temporary Construction Slopes, and Shoring

Excavations for this project are expected to include demolition excavations, over-excavation to rework the upper soils, cuts to achieve design grades, trenching to construct new underground utilities, excavations for construction of the underground stormwater vaults, and foundation excavations. The site soils are generally low plasticity clays and granular soils (sands and gravels) with variable amounts of fines. Granular soils may have little or no cohesion and excavations in these materials will require more extensive bracing or laying back because the granular soils are prone to sudden collapse. Excavations and temporary construction slopes should be constructed in accordance with the current CAL-OSHA safety standards and local jurisdiction. The stability and safety of excavations, braced or unbraced, is the responsibility of the contractor. Care should be exercised when excavating in the proximity of existing structures and improvements. For excavations with no groundwater or seepage, the on-site clayey soils may be considered as Type B soils and the granular soils may be considered as Type C soil per OSHA 29 CFR Part 1926, Appendix A to Subpart P.

Contractors are responsible for the design, installation, maintenance, and removal of temporary shoring and bracing systems. The presence of existing structures, pavements, and underground utilities must be incorporated in the design of the shoring and bracing systems.

Trench excavations adjacent to existing or proposed foundations should be above an imaginary plane having an inclination of 1½:1 (horizontal to vertical) extending down from the bottom edge of the foundations.

6.1.3 Over-excavation and Re-compaction of Soils

After site clearing and stripping, the upper 1 foot of soil below stripped ground surface should be over-excavated. The soil surface exposed by over-excavation should be properly prepared as recommended below under "Subgrade Preparation." After the subgrade soil has been prepared, the excavation may be raised to design grade with engineered fill.

6.1.4 Subgrade Preparation

In areas to receive engineered fills, foundations, concrete slabs-on-grade, and pavements, the subgrade soils should be scarified to a depth of 8 inches, moisture-conditioned, and compacted in accordance with the recommendations given in the "Engineered Fill Placement and Compaction" section below. In building and concrete slab-on-grade areas, subgrade preparation should extend a minimum of 5 feet horizontally beyond the limits of the proposed structures and any adjoining flatwork, unless it is restricted by existing improvements. In pavement areas, subgrade preparation should extend a minimum of 3 feet beyond the back of the curbs or pavements.

Prepared soil subgrades should be non-yielding when proof-rolled by a fully loaded water truck or similar weight equipment. Moisture conditioning of subgrade soils should consist of adding water if the soils are too dry and allowing the soils to dry if the soils are too wet. After the subgrades are properly prepared, the areas may be raised to design grades by placement of engineered fill.

Wet soils should be anticipated during and after rainy months. Where encountered, unstable, wet or soft soil will require processing before compaction can be achieved. If construction schedule does not allow for air-drying, other means such as lime or cement treatment of the soil or excavation and replacement with suitable material may be considered. Geotextile fabrics may also be used to help stabilize the subgrade. The method to be used should be determined at the time of construction based on the actual site conditions. We recommend obtaining unit prices for subgrade stabilization during the construction bid process.

6.1.5 Materials for Fill

In general, on-site soils with an organic content of less than 3 percent by weight, free of deleterious materials or hazardous substances, and meeting the gradation requirements below may be used as engineered fill except where special material (such as capillary break material) is recommended.

Engineered fill material should not contain rocks or lumps larger than 3 inches in greatest dimension, should not contain more than 15 percent of the material larger than 1½ inches, and should contain at least 20 percent passing the No. 200 sieve. In addition to these requirements, import fill should have a low expansion potential as indicated by Plasticity Index of 15 or less (per

ASTM D4318), or Expansion Index of less than 20 (per ASTM D4829).

All fills should be approved by the Geotechnical Engineer before delivery to the site. At least 5 working days prior to importing to the site, a representative sample of the proposed import soil should be delivered to our laboratory for evaluation. Import fills should be tested and approved for residential use per the California Department of Toxic Substances Control (DTSC) guidelines.

6.1.6 Engineered Fill Placement and Compaction

Engineered fill should be placed in horizontal lifts each not exceeding 8 inches in thickness, moisture conditioned to the required moisture content, and mechanically compacted to the recommendations below. Relative compaction or compaction is defined as the in-place dry density of the compacted soil divided by the laboratory maximum dry density as determined by ASTM Test Method D1557, latest edition, expressed as a percentage. Moisture conditioning of soils should consist of adding water to the soils if they are too dry and allowing the soils to dry if they are too wet.

Engineered fills consisting of on-site or imported soils should be compacted to at least 90 percent relative compaction with moisture content between about 1 and 3 percent above the laboratory optimum value. In pavement areas, the upper 8 inches of subgrade soil should be compacted to a minimum of 95 percent relative compaction. Aggregate base in vehicle pavement areas should be compacted at slightly above the optimum moisture content to a minimum of 95 percent relative compaction.

6.1.7 Trench Backfill

Backfilling of utility trenches in public right-of-way areas should comply with the City of San Jose Standard Specifications and Details.

Backfilling of utility trenches in private areas may consist of bedding material extending from the bottom of the trench to about 1 foot above the top of pipe, and on-site or imported backfill material above the bedding to the proposed finish subgrade. Bedding may consist of free-draining sand (less than 5% passing a No. 200 sieve), lean concrete, or sand cement slurry. Sand, if used as bedding, should be compacted to at least 90 percent relative compaction. Backfill material may consist of on-site or imported soil, and should be compacted per recommendations in the “Engineered Fill Placement and Compaction” section above.

The backfill material should be placed in lifts each not exceeding 6 inches in uncompacted thickness. Thicker lifts may be used if the contractor can demonstrate that the recommended level of compaction can be achieved with the compaction equipment and procedures used. Compaction should be performed by mechanical means only. Water jetting or flooding to attain compaction of backfill should not be permitted.

6.1.8 Considerations for Soil Moisture and Seepage Control

Subgrade soil and engineered fill should be compacted at moisture content meeting our recommendations. Consideration should be given to reducing the potential for water infiltration from the exterior to under the buildings through utility lines crossing the building perimeter. In utility lines crossing beneath perimeter foundations, permeable backfill should be terminated at least 1 foot outside of the perimeter foundation. Impermeable material, such as concrete or clay soil, should be used for the entire trench depth to act as a seepage cutoff.

Where concrete slabs or pavements abut against landscaped areas, the base rock layer and subgrade soil should be protected against saturation. Water if allowed to seep into the subgrade soil or pavement section could reduce the service life of the improvements. Methods that may be considered to reduce infiltration of water include: 1) subdrains installed behind curbs and slabs in landscape areas; 2) vertical cut-offs, such as a deepened curb section, or equivalent, extending at least 2 inches into the subgrade soil; and 3) use of a drip or controlled irrigation system for landscape watering.

6.1.9 Wet Weather Construction

If site grading and construction is to be performed during the winter rainy months, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms can cause delay to construction and damage to previously completed work by saturating compacted pads or subgrades, or flooding excavations.

Earthwork during rainy months will require extra effort and caution by the contractors. The contractors are responsible for protecting their work to avoid damage by rainwater. Standing pools of water should be pumped out immediately. Construction during wet weather conditions should be addressed in the project construction bid documents and/or specifications. We recommend the contractors submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

6.2 Foundations

6.2.1 General

The proposed residential structures may be supported on conventional continuous and/or isolated spread footing foundations or post-tensioned slab foundations. General recommendations for design of these foundations are presented below. The Geotechnical Engineer should review the foundation plans and details before construction and observe the foundation excavations during construction to determine if the foundation excavations extend into suitable bearing material. Prior to placement of concrete, foundation excavations should be cleaned of loose soils. If unsuitable soils are encountered in the foundation excavations, the soils should be removed as recommended by our Geotechnical Engineer and replaced with approved material such as compacted engineered fill or lean concrete.

Foundation excavations should not be allowed to dry before placement of concrete. If visible cracks appear in the foundation excavations, the excavations should be thoroughly moisture conditioned beginning at least 2 days prior to placement of concrete to close all cracks. It is also important that the base of the foundation excavations not be allowed to become excessively wet, resulting in soft soils. Water should not be allowed to pond in the bottom of the excavations. Areas that become water damaged should be over-excavated to a firm base. The foundation excavations should be monitored by our representative for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials.

6.2.2 Conventional Continuous and/or Isolated Spread Footing Foundations

Footings, continuous and isolated, may be used to support the proposed residential structures and site retaining walls. Footings should bear on undisturbed native soil and/or properly compacted engineered fill. Preparation of soil subgrade, moisture conditioning, and compaction of soil and engineered fill should be as recommended in the "Earthwork" section of this report.

Footings may be designed for a net allowable bearing pressure of 3,000 pounds per square foot due to dead plus live loads, with a one-third increase when including transient loads such as wind or seismic. The footing bottom should extend at least 18 inches below pad grade or lowest adjacent finish grade, whichever provides a deeper embedment. Footings should be at least 12 inches wide. Footings should be reinforced as determined by the project Structural Engineer.

Resistance to lateral loads may be developed from a combination of friction between the bottom of foundations and the supporting subgrade, and by passive resistance acting against the vertical sides of the foundations. Footings bearing on native soil or engineered fill may be designed using an ultimate friction coefficient of 0.3 between the foundations and supporting subgrade, and an ultimate passive resistance of 300 pounds per cubic foot (pcf, equivalent fluid weight) acting against the embedded sides of the foundations. The passive pressure can be assumed to act starting at the top of the lowest adjacent grade in paved areas. In unpaved areas, the passive pressure can be assumed to act starting at a depth of 1 foot below grade. It should be noted that the passive resistance value discussed above is only applicable where the concrete is placed directly against undisturbed soil or engineered fills. Voids created by the use of forms should be backfilled with property compacted engineered fill or with concrete.

Total post-construction settlement of the foundations is anticipated to be up to about 1 inch, with up to about ½ inch of differential settlement over a distance of about 30 feet.

To maintain the desired support, the bottom of footings adjacent to utility trenches or buried structures should be below an imaginary plane having an inclination of 1.5 horizontal to 1 vertical, extending upward from the bottom edge of the adjacent utility trenches or structures. If the footings are closer than the recommended distance, the project Geotechnical Engineer should be consulted for recommendations.

6.2.3 Post-tensioned Slabs

In lieu of footings, the proposed residential structures may be constructed on post-tensioned (PT) slab foundations bearing on properly moisture-conditioned and compacted soil subgrades. Preparation of soil subgrade, moisture conditioning, and compaction of soil and engineered fill should be as recommended in the "Earthwork" section of this report.

The following parameters may be used with the 2004 PTI "Design of Post-Tensioned Slabs-on-Ground, Third Edition" manual for design of the PT slabs.

Parameters	PT Slabs Constructed on Properly Prepared Subgrade Soil
e_m (center lift)	9 feet
e_m (edge lift)	5.2 feet
y_m (center lift)	0.25 inch
y_m (edge lift)	0.5 inch

Allowable soil bearing pressure = 1,500 psf for dead plus live loads, with a one-third increase when including transient loads, such as wind or seismic

A deepened edge, minimum 6 inches wide, should be constructed along the perimeter of the PT slabs. The deepened edge should extend to at least 18 inches below the bottom of the PT slabs. The deepened edge can help reduce moisture infiltration to under the PT slabs.

Where interior building grades are higher than the exterior grades, the perimeter foundation elements should be designed to resist the lateral soil pressure and surcharge loads acting on the foundations. The bottom of the perimeter foundations should extend at least 18 inches below the lowest finish grades, excluding landscaping soils which are typically not compacted and should not be considered for structural support.

We understand the PT slabs will be constructed on 1 to 2 inches of sand over a 15-mil visqueen vapor barrier over compacted subgrade soil. Sand has been used for protection of the vapor barrier during construction and to allow dissipation of concrete mix water during curing. The use of sand, or equivalent material, should be determined by the project structural engineer or architect. A lower water-cement ratio (0.45 to 0.50) will help reduce the permeability of the concrete and, hence, vapor transmission through the slabs.

Settlements are expected to be primarily elastic. Post construction total and differential settlements of the PT slabs are anticipated to be less than 1 and ½ inch, respectively.

6.2.4 Drilled Pier Foundations

Drilled, cast-in-place, reinforced concrete piers may be considered for support of proposed pole type structures. Piers should be designed to derive their vertical supporting capacity from "skin friction" between the pier shafts and the surrounding earth materials. Piers should have a diameter of 12 inches or greater. Center to center spacing of the piers should be a minimum of 3 pier diameters. Reinforcement in the piers should be determined by the structural engineer.

For dead plus live vertical loads, a net allowable adhesion value of 450 pounds per square foot may be assumed along the pier shafts. This value may be increased by one-third when including transient loads, such as wind or seismic. The upper 1 foot of soil should be ignored in the calculation of vertical load capacity. End bearing capacity should be ignored.

Resistance to lateral loads may be calculated based on passive soil pressure acting against the piers. For dead plus live loads, the ultimate passive resistance in soil or engineered fill may be calculated using an equivalent fluid weight of 300 pounds per cubic foot acting on 1.5 times the pier diameter, for level ground surface in front of the piers in the direction of load application. The upper 1 foot of soil should be ignored in the calculation of passive pressure. It should be noted that passive resistance is only applicable where the concrete is placed directly against undisturbed soil or engineered fill.

The presence of granular soils should be considered in the design and construction of the foundation piers because granular soils are prone to caving if the holes are not cased. Steel casing should be provided to keep the pier holes open. If piers extend below groundwater level, concrete should be placed by the "tremie" method to replace the water in the pier holes.

6.3 Concrete Slabs-on-Grade

6.3.1 Interior Building Slabs-on-grade

If the buildings are supported on conventional footings, the interior building floors are anticipated to be concrete slabs-on-grade. Interior building concrete slabs-on-grade should be constructed on properly prepared subgrade soil as recommended in the "Earthwork" section of this report. Once the slab subgrade soil has been moisture conditioned and compacted, the soil should not be allowed to dry prior to concrete placement. If the subgrade soil is too dry, the moisture content of the soil should be restored to the recommended value prior to placement of concrete. The project structural engineer should design the slab thickness, reinforcing, and control joint spacing.

Slabs that will be covered with moisture sensitive floor coverings or where vapor transmission through the slab is undesirable should be underlain by at least 4 inches of capillary break material such as free draining, ¾-inch by No. 4 clean crushed rock. A visqueen layer should be placed over the capillary break material. The visqueen should be a high-quality polymer at least 15 mils thick that is resistant to puncture during slab construction. Laps between sheets and openings should be taped. Typically, the membrane and the slab are separated by 2 inches of sand but this should be determined by the structural engineer and architect.

A lower water-cement ratio (0.45 to 0.50) will also help reduce the permeability of the floor slab.

It should be understood that the recommended plastic membrane is not intended to waterproof the concrete slab floor. If waterproofing is desired, the project designers and/or a flooring expert should be contacted.

6.3.2 Exterior Slabs-on-grade

Exterior concrete slabs-on-grade for this project will be limited to driveways and exterior flatwork. These slabs should be constructed on properly moisture conditioned and compacted subgrade soil as recommended in the "Earthwork" section of this report. Soil subgrades MUST be maintained in a moist condition prior to placement of concrete for the concrete slabs. Design of reinforcement, joint spacing, etc. is the responsibility of the design engineer.

Exterior concrete slabs-on-grade should be cast free from adjacent foundations or other non-heaving edge restraints. This may be accomplished by using a strip of 1/2-inch asphalt-impregnated felt divider material between the slab edges and the adjacent structure. Frequent construction or control joints should be provided in all concrete slabs where cracking is objectionable. Continuous reinforcing or dowels at the construction and control joints will also aid in reducing uneven slab movements.

6.4 Retaining Walls

Retaining walls for this project are anticipated to be landscaping walls with exposed height up to about 3 feet. Retaining walls should be designed to resist lateral earth pressure and surcharge forces acting on the walls. Lateral pressures will depend on the degree of movement the walls are allowed (or desired), the type of backfill, the magnitude of external loads, and subsurface drainage provisions. For static loading conditions, the walls may be designed using at-rest or active soil pressure. At-rest soil pressure should be used for walls where movement at the top of walls is restrained or undesirable. Wall movements could cause settlement of backfill and structures supported on the backfill. Active soil pressure may be used for retaining walls where the top of walls is free to deflect and resulting movement of the backfill is acceptable. The at-rest and active soil pressures given below are for level backfill surface and include both drained and undrained backfill conditions.

Condition	Lateral Soil Pressure (Equivalent Fluid Weight) for Level Backfill	
	Drained Backfill	Undrained Backfill
Active	45 pcf	80 pcf
At-rest	55 pcf	90 pcf

Note: To develop active soil pressures, wall movements of about 0.005H to 0.01H may be necessary for cohesive soils, with up to 0.005H for cohesionless soils.

Pressures due to static external loads should be added to the soil pressures recommended above in the wall design. For uniform vertical load at the ground surface, the additional lateral pressure on the walls should be calculated as a uniform pressure equal to the magnitude of the vertical load multiplied by a factor. For level backfill slope, the factor is 0.38 for active soil condition and

0.5 for at-rest soil condition. For other slope inclinations and other types of surcharge loads, such as vehicle loads, point loads, strip loads, consult our office for specific recommendations.

Foundations for retaining walls may consist of footings or drilled piers designed using the recommendations in the "Foundations" section of this report.

To achieve a drained backfill condition, a subsurface drain should be installed behind each wall extending from the wall bottom to about 1 foot below finished grade. The drain should consist of a 12-inch minimum wide blanket of drainage material consisting of either Class 2 Permeable material (Caltrans Standard Specifications, Section 68) or clean, 1/2 to 3/4-inch maximum size crushed rock or gravel. If crushed rock or gravel is used, it should be encapsulated in a geotextile filter fabric, such as Mirafi 140N or equivalent. Filter fabric is optional if Class 2 Permeable material is used. The top 1 foot below finish grade should be backfilled with compacted clayey soil to reduce infiltration of surface water.

A 4-inch minimum diameter, perforated, schedule 40 PVC (or equivalent) pipe should be installed (with perforations facing down) along the base of each wall on a 2-inch thick bed of drain rock, regardless whether drain rock or pre-fabricated drainage panel is used. The pipes should be sloped to drain by gravity to a proper collection system and be discharged at a proper outlet as designed by the project Civil Engineer.

Backfill against retaining walls should be compacted as discussed in the "Earthwork" Section of this report. Over-compaction should be avoided because increased compaction effort can result in lateral pressures significantly higher than those recommended above. Backfill placed within 3 feet of the walls should be compacted with hand-operated equipment.

6.5 Vehicle Pavements

Vehicle pavements for this project will be an interior street, primarily serving automobiles and light pickup trucks, with occasional heavy vehicles, such as delivery and garbage trucks. If the pavements are constructed prior to completion of construction, the pavements will be subject to construction traffic including heavy delivery and concrete trucks.

An R-value of 54 was measured on a bulk sample of soil collected from the site. For design purposes, an R-value of 35 was used to calculate the pavement sections tabulated below using the Caltrans pavement section design procedures.

DESIGN TRAFFIC INDEX	HOT MIX ASPHALT (inches)	CLASS 2 AGGREGATE BASE (inches)	TOTAL (inches)
5.0	3.0	5.0	8.0
5.5	3.0	6.0	9.0
6.0	3.5	6.5	10.0
6.5	3.5	8.0	11.5

DESIGN TRAFFIC INDEX	HOT MIX ASPHALT (inches)	CLASS 2 AGGREGATE BASE (inches)	TOTAL (inches)
7.0	4.0	8.5	12.5

Pavement sections should be constructed on soil subgrades that have been prepared as outlined in the "Earthwork" section of this report. The upper 8 inches of soil subgrade in pavement areas should be compacted to a minimum of 95 percent relative compaction. The full section of aggregate base and aggregate subbase should be compacted to a minimum of 95 percent relative compaction. Evaluation of relative compaction should be based on ASTM D1557, latest edition. The Class 2 Aggregate Base material should conform to Section 26 of the Caltrans Standard Specifications and the Class 2 Aggregate Subbase material should conform to Section 25 of the Caltrans Standard Specifications.

6.6 Surface and Subsurface Drainage

Engineering design of grading and drainage at the site is the responsibility of the project Civil Engineer. We suggest the following for consideration by the project Civil Engineer, as appropriate.

Sufficient surface drainage should be provided to direct water away from buildings, foundations, concrete slabs-on-grade and pavements, and towards suitable collection and discharge facilities. Ponding of surface water should be avoided by establishing positive drainage away from all improvements.

6.7 Stormwater Treatment System

A stormwater treatment and retention system is proposed in the northwestern portion of the project site. Details regarding this system are not available at the time this report was prepared. Based on our experience with past Robson Homes projects, we have anticipated the system would involve underground stormwater vaults and bioretention basins. The stormwater vaults may extend about 10 feet below ground surface. The bioretention basins typically would compose of an 18-inch thick layer of bio-treatment soil mix underlain by a 12-inch thick layer of Caltrans Class 2 Permeable material. We recommend the following guidelines be incorporated in the planning and design of the bioretention system.

- Underground vaults, bioretention basins, pipes, etc. should be constructed above an imaginary plane extending down at an inclination of 1.5:1 (horizontal:vertical) from the bottom edge or corner of nearby foundations. This may require deepening of the nearby foundations.

- Bioretention basins should be constructed above an imaginary plane extending down at an inclination of 1.5:1 (horizontal:vertical) from the bottom edge of nearby exterior flatwork or pavements. If this minimum set back is not met, the following should be considered.

- Line the sides of the bioretention basins with an impermeable barrier to reduce lateral migration of water.

- Install one or more layers of geogrids in the soil adjacent to the bioretention basin materials for added lateral support. If the vertical distance between the bottom of the bioretention basins and the adjacent finish grade (H) is 5 feet or less, one layer of geogrid at least 6 feet wide should be installed at mid-height (H/2). If H is greater than 5 feet, additional layers of geogrids should be installed at not more than 2 feet vertical spacing. The length and elevation of multi geogrid layers should be determined by the Geotechnical Engineer after review of the basin design.

- Construct concrete curbs for pavements. The concrete curbs should extend below the bottom of the bioretention basins and should be designed to resist the lateral soil pressure recommended in this report.

7 PLAN REVIEW, EARTHWORK AND FOUNDATION OBSERVATION

Post-report geotechnical services by Geo-Logic Associates (GLA), typically consisting of pre-construction design consultations and reviews and construction observation and testing services, are necessary for GLA to confirm the recommendations contained in this report. This report is based on limited sampling and investigation, and by those constraints may not have discovered local anomalies or other varying conditions that may exist on the project site. Therefore, this report is only preliminary until GLA can confirm that actual conditions in the ground conform to those anticipated in the report. Accordingly, as an integral part of this report, GLA recommends post-report, construction related geotechnical services to assist the project team during design and construction of the project. GLA requires that it perform these services if it is to remain as the project Geotechnical Engineer-of-record.

During design, GLA can provide consultation and supplemental recommendations to assist the project team in design and value engineering, especially if the project design has been modified after completion of our report. It is impossible for us to anticipate every design scenario and use of construction materials during preparation of our report. Therefore, retaining GLA to provide post-report consultation will help address design changes, answer questions and evaluate alternatives proposed by the project designers and contractors.

Prior to issuing project plans and specifications for construction bidding purposes, GLA should review the grading, drainage and foundation plans and the project specifications to determine if the intent of our recommendations has been incorporated in these documents. We have found that such a review process will help reduce the likelihood of misinterpretation of our recommendations which may cause construction delay and additional cost.

Construction phase services can include, among other things, the observation and testing during site clearing, stripping, excavation, mass grading, subgrade preparation, fill placement and compaction, backfill compaction, foundation construction and pavement construction activities.

Geo-Logic Associates would be pleased to provide cost proposals for follow-up geotechnical services. Post-report geotechnical services may include additional field and laboratory services.

8 LIMITATIONS

In preparing the findings and professional opinions presented in this report, Geo-Logic Associates (GLA) has endeavored to follow generally accepted principles and practices of the engineering geologic and geotechnical engineering professions in the area and at the time our services were performed. No warranty, express or implied, is provided.

The conclusions and recommendations contained in this report are based, in part, on information that has been provided to us. In the event that the general development concept or general location and type of structures are modified, our conclusions and recommendations shall not be considered valid unless we are retained to review such changes and to make any necessary additions or changes to our recommendations. To remain as the project Geotechnical Engineer-of-record, GLA must be retained to provide geotechnical services as discussed under the Post-report Geotechnical Services section of this report.

Subsurface exploration is necessarily confined to selected locations and conditions may, and often do, vary between these locations. Should conditions different from those described in this report be encountered during project development, GLA should be consulted to review the conditions and determine whether our recommendations are still valid. Additional exploration, testing, and analysis may be required for such evaluation.

Should persons concerned with this project observe geotechnical features or conditions at the site or surrounding areas which are different from those described in this report, those observations should be reported immediately to GLA for evaluation.

It is important that the information in this report be made known to the design professionals involved with the project, that our recommendations be incorporated into project drawings and documents, and that the recommendations be carried out during construction by the contractor and subcontractors. It is not the responsibility of GLA to notify the design professionals and the project contractors and subcontractors.

The findings, conclusions, and recommendations in this report are applicable only to the specific project development on this specific site. These data should not be used for other projects, sites, or purposes unless they are reviewed by GLA or a qualified geotechnical professional.

Report prepared by,

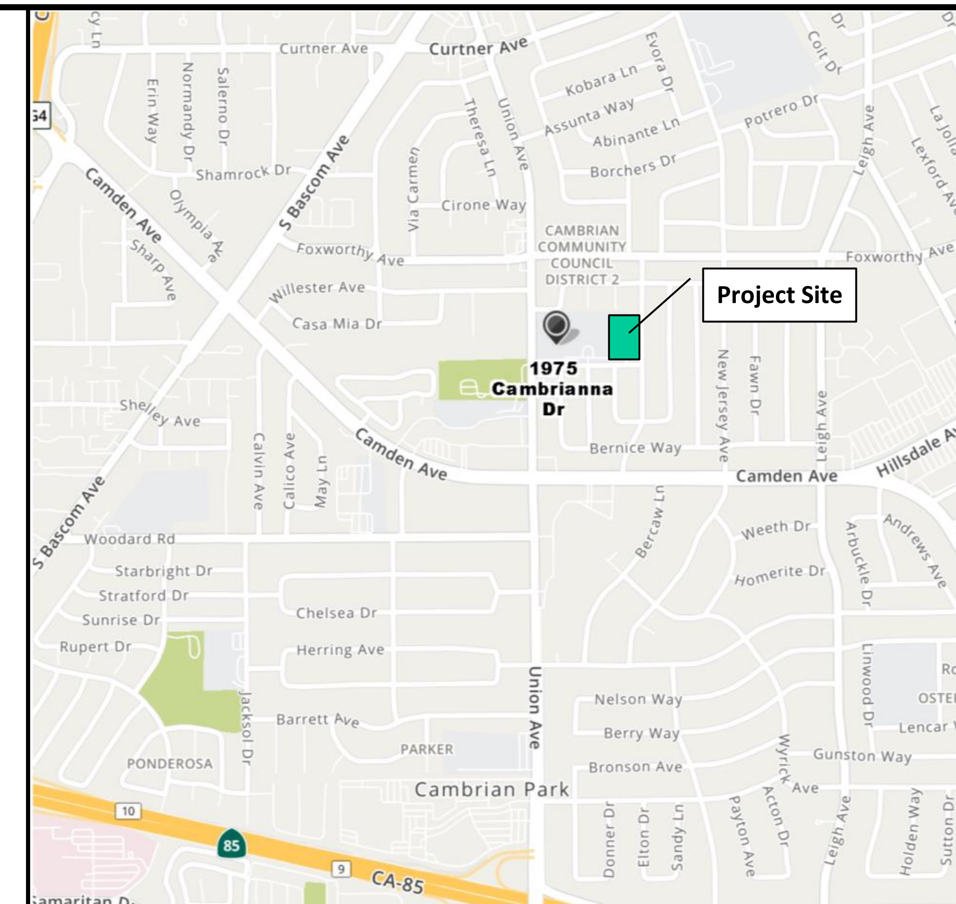
Geo-Logic Associates

REGISTERED PROFESSIONAL ENGINEER
CHALERM S. LIANG
GE 2031
GEOTECHNICAL
STATE OF CALIFORNIA

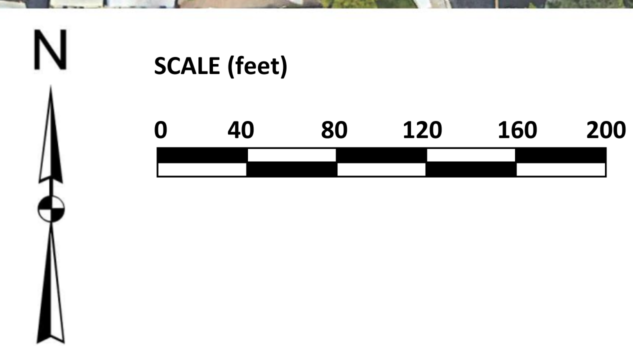
Chalerm (Beeson) Liang
GE 2031

Fs/csl

Copy: Robson Homes, Mary Gourlay (electronic copy)



VICINITY MAP (NO SCALE)



Legend

DH-6 Number & approximate location of exploratory drill hole

Base

Google Earth, 1975 Cambrianna Drive, San Jose, California (8/11/2021)

Geo-Logic Associates
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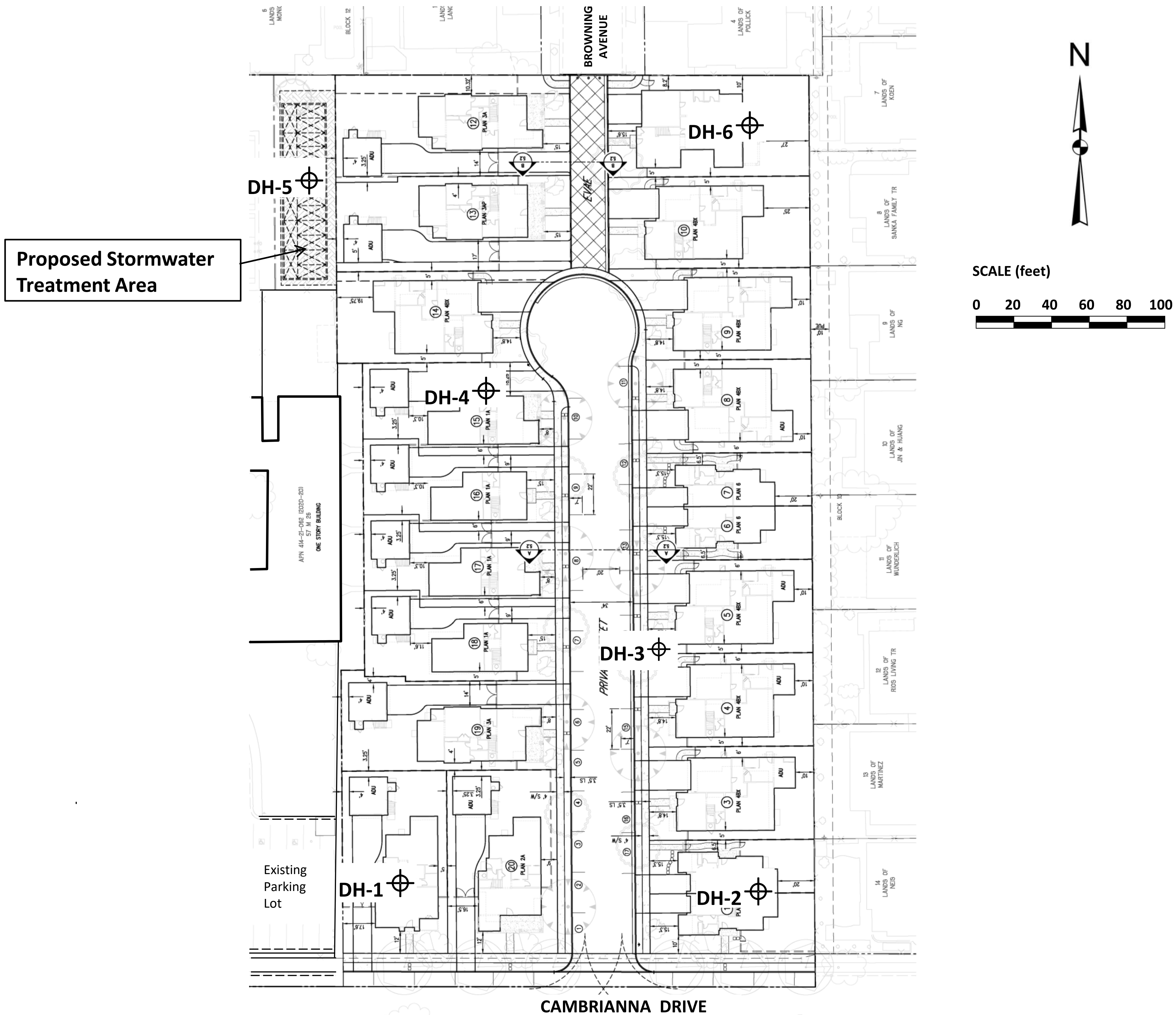
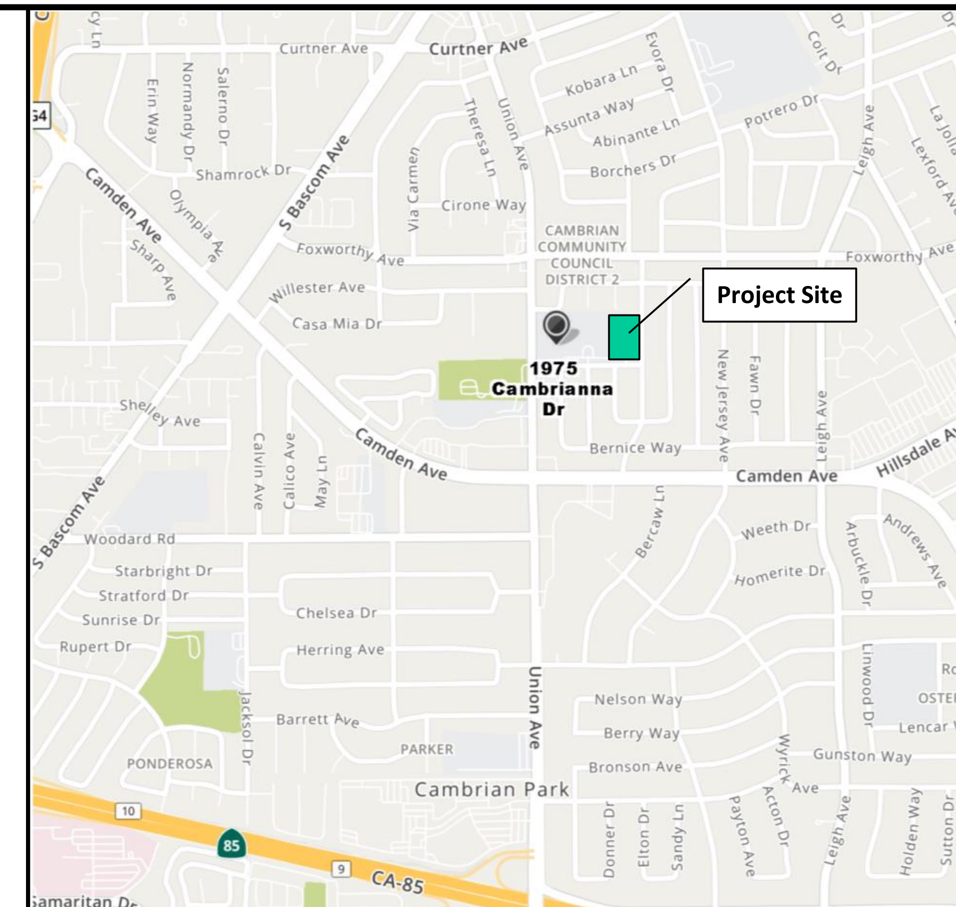
Drafted By:
Date: January 2022
Checked By:
Revision:

**SITE PLAN (EXISTING CONDITIONS)
PROPOSED RESIDENTIAL DEVELOPMENT
1975 CAMBRIANNA DRIVE
SAN JOSE, CALIFORNIA**

FIGURE 1

PROJECT PA21.1017





VICINITY MAP (NO SCALE)

Legend

DH-6 — Number & approximate location of exploratory drill hole

Base

Site Plan, 1975 Cambrianna Drive, San Jose, California, Sheet 3.0, prepared by Civil Engineering Associates, dated 7/22/2021

Geo-Logic ASSOCIATES

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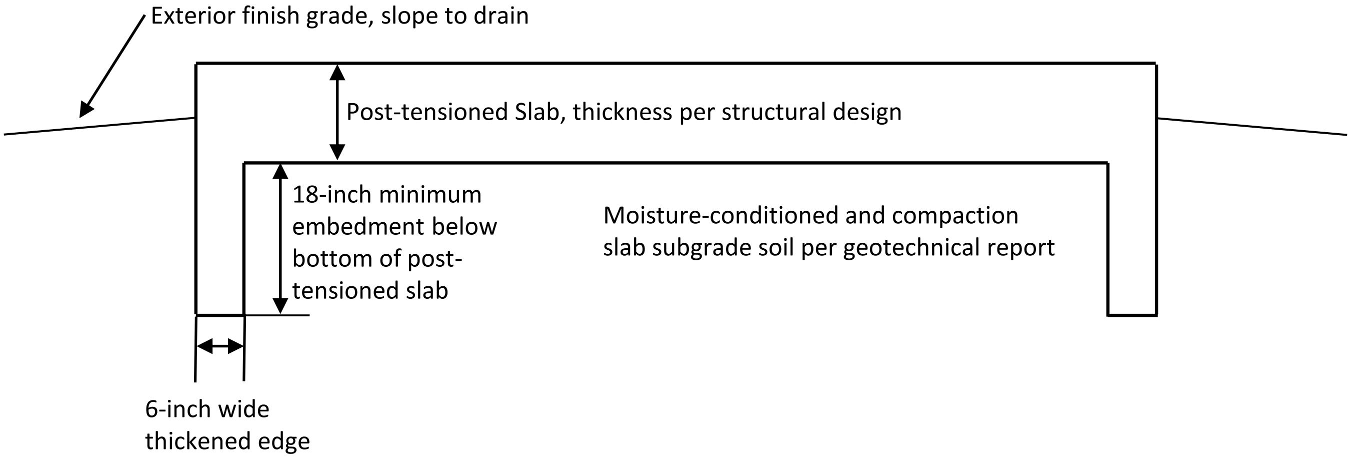
Date:	January 2022
Checked By:	
Revision:	

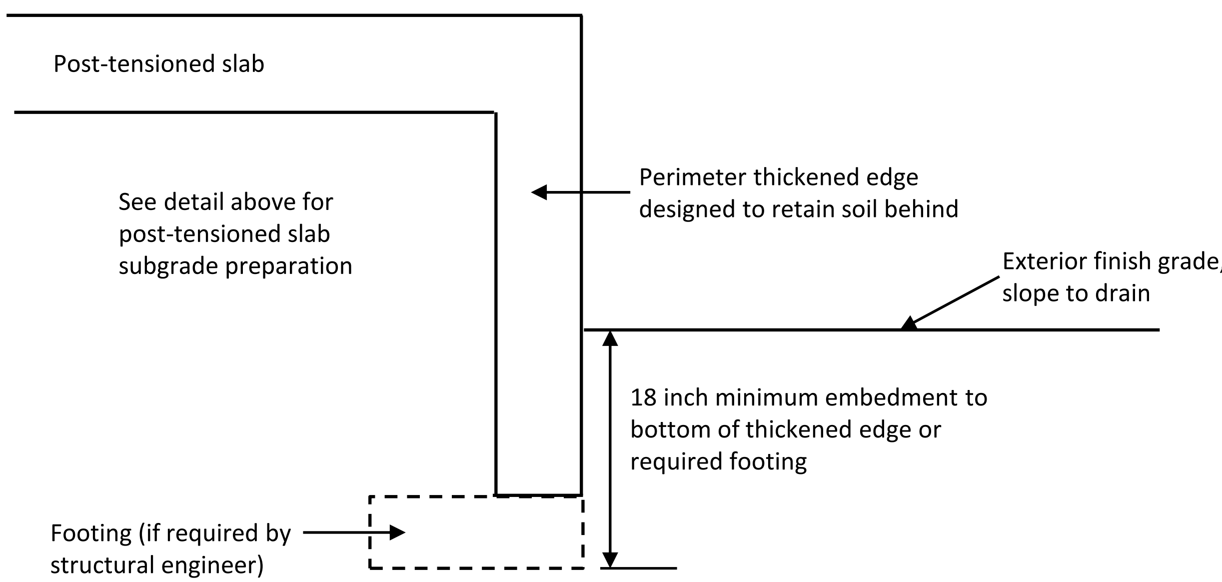
SITE PLAN (PROPOSED DEVELOPMENT)
PROPOSED RESIDENTIAL DEVELOPMENT
1975 CAMBRIANNA DRIVE
SAN JOSE, CALIFORNIA

FIGURE 2

PROJECT PA21.1017



Subgrade Preparation and Thickened Edge for Post-tensioned Slab Foundations



Foundation Embedment for Post-tensioned Slabs with Differential Grades

Note:
1. Refer to geotechnical report for detailed recommendations.

Schematic Only – Not to Scale

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Compiled by:	Date:
Reviewed by:	Revision:

POST-TENSIONED SLAB
TYPICAL SECTION
1975 CAMBRIANNA DRIVE
SAN JOSE, CALIFORNIA

FIGURE
3

PROJECT
PA21.1017

APPENDIX A

KEYS TO SOIL CLASSIFICATION

AND

DRILL HOLE LOGS

KEY TO SOIL CLASSIFICATION - FINE GRAINED SOILS

(50% OR MORE IS SMALLER THAN NO. 200 SIEVE SIZE)

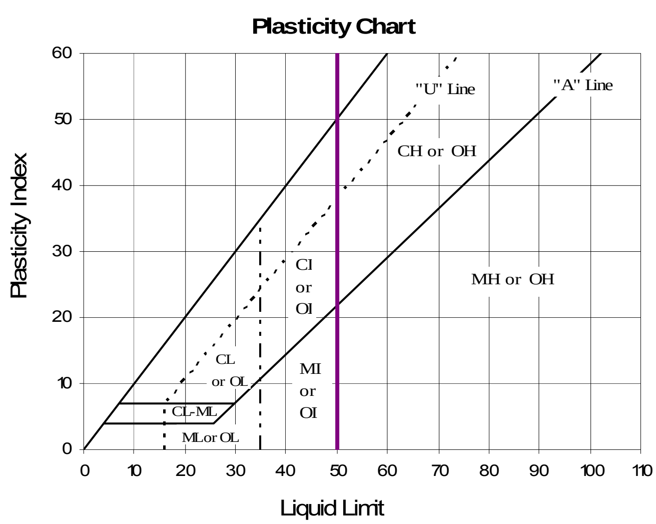
(modified from ASTM D2487 to include fine grained soils with intermediate plasticity)

MAJOR DIVISIONS			GROUP SYMBOLS	GROUP NAMES
SILTS AND CLAYS (Liquid Limit less than 35) Low Plasticity	Inorganic	PI < 4 or plots below "A" line	ML	Silt, Silt with Sand or Gravel, Sandy or Gravelly Silt, Sandy or Gravelly Silt with Sand or Gravel
	Inorganic	PI > 7 or plots on or above "A" line	CL	Lean Clay, Lean Clay with Sand or Gravel, Sandy or Gravelly Lean Clay, Sandy or Gravelly Lean Clay with Sand or Gravel
	Inorganic	PI between 4 and 7	CL-ML	Silty Clay, Silty Clay with Sand or Gravel, Sandy or Gravelly Silty Clay, Sandy or Gravelly Silty Clay with Sand or Gravel
	Organic	See footnote 3	OL	Organic Silt (below "A" Line) or Organic Clay (on or above "A" Line) (1,2)
SILTS AND CLAYS (35 ≤ Liquid Limit < 50) Intermediate Plasticity	Inorganic	PI < 4 or plots below "A" line	MI	Silt, Silt with Sand or Gravel, Sandy or Gravelly Silt, Sandy or Gravelly Silt with Sand or Gravel
	Inorganic	PI > 7 or plots on or above "A" line	CI	Clay, Clay with Sand or Gravel, Sandy or Gravelly Clay, Sandy or Gravelly Clay with Sand or Gravel
	Organic	See footnote 3	OI	Organic Silt (below "A" Line) or Organic Clay (on or above "A" Line) (1,2)
SILTS AND CLAYS (Liquid Limit 50 or greater) High Plasticity	Inorganic	PI plots below "A" line	MH	Elastic Silt, Elastic Silt with Sand or Gravel, Sandy or Gravelly Elastic Silt, Sandy or Gravelly Elastic Silt with Sand or Gravel
	Inorganic	PI plots on or above "A" line	CH	Fat Clay, Fat Clay with Sand or Gravel, Sandy or Gravelly Fat Clay, Sandy or Gravelly Fat Clay with Sand or Gravel
	Organic	See note 3 below	OH	Organic Silt (below "A" Line) or Organic Clay (on or above "A" Line) (1,2)

1. If soil contains 15% to 29% plus No. 200 material, include "with sand" or "with gravel" to group name, whichever is predominant.
2. If soil contains ≥30% plus No. 200 material, include "sandy" or "gravelly" to group name, whichever is predominant. If soil contains ≥15% of sand or gravel sized material, add "with sand" or "with gravel" to group name.
3. Ratio of liquid limit of oven dried sample to liquid limit of not dried sample is less than 0.75.

CONSISTENCY	UNCONFINED SHEAR STRENGTH (KSF)	STANDARD PENETRATION (BLOWS/FOOT)
VERY SOFT	< 0.25	< 2
SOFT	0.25 – 0.5	2 – 4
FIRM	0.5 – 1.0	5 – 8
STIFF	1.0 – 2.0	9 – 15
VERY STIFF	2.0 – 4.0	16 – 30
HARD	> 4.0	> 30

MOISTURE	CRITERIA
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soil is below the water table



GEO-LOGIC ASSOCIATES

KEY TO SOIL CLASSIFICATION – COARSE GRAINED SOILS
(MORE THAN 50% IS LARGER THAN NO. 200 SIEVE SIZE)
(modified from ASTM D2487 to include fines with intermediate plasticity)

MAJOR DIVISIONS			GROUP SYMBOLS	GROUP NAMES[1]
GRAVELS (more than 50% of coarse fraction is larger than No. 4 sieve size)	Gravels with less than 5% fines	Cu ≥ 4 and 1 ≤ Cc ≤ 3	GW	Well Graded Gravel, Well Graded Gravel with Sand
		Cu < 4 and/or 1 > Cc > 3	GP	Poorly Graded Gravel, Poorly Graded Gravel with Sand
	Gravels with 5% to 12% fines	ML, MI or MH fines	GW-GM	Well Graded Gravel with Silt, Well Graded Gravel with Silt and Sand
			GP-GM	Poorly Graded Gravel with Silt, Poorly Graded Gravel with Silt and Sand
		CL, CI or CH fines	GW-GC	Well Graded Gravel with Clay, Well Graded Gravel with Clay and Sand
			GP-GC	Poorly Graded Gravel with Clay, Poorly Graded Gravel with Clay and Sand
	Gravels with more than 12% fines	ML, MI or MH fines	GM	Silty Gravel, Silty Gravel with Sand
		CL, CI or CH fines	GC	Clayey Gravel, Clayey Gravel with Sand
		CL-ML fines	GC-GM	Silty Clayey Gravel; Silty, Clayey Gravel with Sand
SANDS (50% or more of coarse fraction is smaller than No. 4 sieve size)	Sands with less than 5% fines	Cu ≥ 6 and 1 ≤ Cc ≤ 3	SW	Well Graded Sand, Well Graded Sand with Gravel
		Cu < 6 and/or 1 > Cc > 3	SP	Poorly Graded Sand, Poorly Graded Sand with Gravel
	Sands with 5% to 12% fines	ML, MI or MH fines	SW-SM	Well Graded Sand with Silt, Well Graded Sand with Silt and Gravel
			SP-SM	Poorly Graded Sand with Silt, Poorly Graded Sand with Silt and Gravel
		CL, CI or CH fines	SW-SC	Well Graded Sand with Clay, Well Graded Sand with Clay and Gravel
			SP-SC	Poorly Graded Sand with Clay, Poorly Graded Sand with Clay and Gravel
	Sands with more than 12% fines	ML, MI or MH fines	SM	Silty Sand, Silty Sand with Gravel
		CL, CI or CH fines	SC	Clayey Sand, Clayey Sand with Gravel
		CL-ML fines	SC-SM	Silty, Clayey Sand; Silty, Clayey Sand with Gravel

US STANDARD SIEVES 3 Inch ¾ Inch No. 4 No. 10 No. 40 No. 200

	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES & BOULDERS	GRAVELS		SANDS			SILTS AND CLAYS

RELATIVE DENSITY (SANDS AND GRAVELS)	STANDARD PENETRATION (BLOWS/FOOT)
Very Loose	0 - 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 - 50
Very Dense	50+

1. Add “with sand” to group name if material contains 15% or greater of sand-sized particle. Add “with gravel” to group name if material contains 15% or greater of gravel-sized particle.

MOISTURE	CRITERIA
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soi is below the water table

GEO-LOGIC ASSOCIATES

DATE: 8/9/2021	LOG OF EXPLORATORY DRILL HOLE	DH-1

PROJECT NAME: 1975 Cambrianna Drive **PROJECT NUMBER:** PA21.1017

DRILL RIG: CME-55, auto hammer **LOGGED BY:** WS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
PAVEMENT: ±2" base rock over ±1.5" AC over ±3" base rock												
SANDY SILTY CLAY: Brown, moist, stiff	CL-ML	1										
		2	SDD	5	1.7			14		111		
		3										
SANDY LEAN CLAY with GRAVEL: Brown, moist, hard	CL	4	SDD	41	4.5+			9		119		
		5										
		6										
		7										
CLAYEY SAND with GRAVEL: Brown, dry to moist, dense; fine to coarse sand, with fine to coarse gravel	SC	8										
		9	SII	28		18		6				
		10										
		11										
		12										
		13										
WELL GRADED SAND with CLAY AND GRAVEL: Light brown, dry to moist, dense; fine to coarse sand, with fine to coarse gravel	SW-SC	14	SDD	39								
		15										
		16										
		17										
		18										
very dense		19	SII	64								
BOTTOM OF HOLE = 20 Feet		20										
No groundwater encountered												

DATE: 8/9/2021	LOG OF EXPLORATORY DRILL HOLE	DH-2

PROJECT NAME: 1975 Cambrianna Drive **PROJECT NUMBER:** PA21.1017

DRILL RIG: CME-55, auto hammer **LOGGED BY:** WS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
PAVEMENT: ±2" base rock over ±1.5" AC over ±3" base rock												
SANDY SILTY CLAY with GRAVEL: Brown, moist, stiff to very stiff	CL-ML	1										
		2	SDD	6	2.7			12		118		
		3										
SANDY LEAN CLAY with GRAVEL: Brown, dry to moist, very stiff to hard	CL	4	SDD	27				8		119		
		5										
		6										
		7										
		8										
CLAYEY GRAVEL with SAND: Brown, dry to moist, dense; fine to coarse gravel, with fine to coarse sand	GC	9	SDD	34		17		7		128		
		10										
		11										
		12										
		13										
WELL GRADED SAND with CLAY AND GRAVEL: Light brown, dry to moist, dense; fine to coarse sand, with fine to coarse gravel	SW-SC	14	SII	33								
		15										
		16										
		17										
		18										
very dense		19	SD	50/4"								
BOTTOM OF HOLE = 19.5 Feet												
No groundwater encountered		20										

DATE: 8/9/2021	LOG OF EXPLORATORY DRILL HOLE	DH-3

PROJECT NAME: 1975 Cambrianna Drive **PROJECT NUMBER:** PA21.1017

DRILL RIG: CME-55, auto hammer **LOGGED BY:** WS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SANDY SILTY CLAY: Light brown, dry, stiff to very stiff	CL-ML	1										
		2	SDD	16				4		81		
		3										
CLAYEY SAND with GRAVEL: Light brown, dry, very dense; fine to coarse sand, with fine to coarse gravel	SC	4	SDD	76				6		122		
		5										
		6										
		7										
POORLY GRADED GRAVEL with CLAY and SAND: Light brown, dry, very dense; mostly fine gravel, with fine to coarse sand	GP-GC	8										
		9	SDD	66		6		3		123		
		10										
		11										
		12										
		13										
WELL GRADED SAND with CLAY AND GRAVEL: Light brown, dry to moist, dense to very dense; fine to coarse sand, with fine to coarse gravel	SW-SC	14	SII	38								
		15										
		16										
		17										
		18										
very dense		19	SII	80								
BOTTOM OF HOLE = 20 Feet		20										
No groundwater encountered												

DATE: 8/9/2021 | **LOG OF EXPLORATORY DRILL HOLE** | **DH-4**

PROJECT NAME: 1975 Cambrianna Drive **PROJECT NUMBER:** PA21.1017

DRILL RIG: CME-55, auto hammer **LOGGED BY:** WS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SANDY SILTY CLAY: Brown, moist, firm	CL-ML	1										
		2	S D D	3		63	18	16	4	107		
		3										
LEAN CLAY: Brown, moist, very stiff to hard	CL	4	S D D	41				16		115		
		5										
		6										
		7										
WELL GRADED SAND with CLAY and GRAVEL: Brown, moist, very dense; fine to coarse sand, with fine to coarse gravel	SW-SC	8										
		9	S D D	50				7		130		
		10										
		11										
		12										
		13										
		14	S D D	43		8		7		125		
dense		15										
		16										
		17										
		18										
CLAYEY SAND with GRAVEL: Brown, moist, dense; fine to coarse sand, with fine to coarse gravel	SC	19	S D D	46								
		20										

GEO-LOGIC ASSOCIATES

DATE: 8/9/2021	LOG OF EXPLORATORY DRILL HOLE	DH-4
PROJECT NAME: 1975 Cambrianna Drive	PROJECT NUMBER:	PA21.1017
DRILL RIG: CME-55, auto hammer	LOGGED BY:	WS
HOLE DIAMETER: 8-inch hollow stem auger	HOLE ELEVATION:	---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAYEY SAND with GRAVEL: (continued)	SC											
		21										
		22										
		23										
very dense		24	S D D	50/6"				6		121		
		25										
		26										
		27										
		28										
POORLY GRADED SAND with CLAY and GRAVEL: Dark brown, moist, dense to very dense; fine to coarse sand, with mostly fine gravel	SW-SC	29	S I I	41								
		30										
		31										
		32										
		33										
brown, moist to wet, very dense		34	S D D	54								
		35										
		36										
		37										
		38										
moist		39	S D D	84								
		40										

DATE: 8/9/2021	LOG OF EXPLORATORY DRILL HOLE	DH-4
PROJECT NAME: 1975 Cambrianna Drive	PROJECT NUMBER: PA21.1017	
DRILL RIG: CME-55, auto hammer	LOGGED BY: WS	
HOLE DIAMETER: 8-inch hollow stem auger	HOLE ELEVATION: ---	
SAMPLER: D = 3" OD, 2½" ID Split-spoon X = 2½" OD, 2" ID Split-spoon I = Standard Penetrometer (2" OD SPT) S = Slough in sample	GROUND WATER DEPTH: Initial: --- Final: ---	

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
POORLY GRADED SAND with CLAY and GRAVEL: (continued)	SW-SC											
		41										
		42										
		43										
moist to wet		44	S D D	93								
BOTTOM OF HOLE = 45 Feet **No groundwater encountered**		45										
		46										
		47										
		48										
		49										
		50										
		51										
		52										
		53										
		54										
		55										
		56										
		57										
		58										
		59										
		60										

DATE: 8/9/2021	LOG OF EXPLORATORY DRILL HOLE	DH-5

PROJECT NAME: 1975 Cambrianna Drive **PROJECT NUMBER:** PA21.1017

DRILL RIG: CME-55, auto hammer **LOGGED BY:** WS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SANDY SILTY CLAY: Light brown, dry, very stiff to hard	CL-ML	1										
		2	S D D	23				4		100		
		3										
CLAYEY SAND with GRAVEL: Light brown, dry to moist, dense; fine to coarse sand, with mostly fine gravel	SC	4	S D D	41				6		125		
		5										
		6										
		7										
		8										
brown		9	S D D	33		15		10		130		
		10										
		11										
		12										
		13										
POORLY GRADED SAND with CLAY and GRAVEL: Brown, dry to moist, very dense; fine to coarse sand, with mostly fine gravel	SP-SC	14	S D D	58		9		6		123		
		15										
		16										
POORLY GRADED GRAVEL with CLAY and SAND: Brown, dry to moist, very dense; mostly fine gravel, with fine to coarse sand	GP-GC	17										
		18										
		19	S D D	96		10		8		131		
BOTTOM OF HOLE = 20 Feet		20										
No groundwater encountered												

DATE: 8/9/2021	**LOG OF EXPLORATORY DRILL HOLE**		**DH-6**
PROJECT NAME:	1975 Cambrianna Drive	**PROJECT NUMBER:**	PA21.1017
DRILL RIG:	CME-55, auto hammer	**LOGGED BY:**	WS
HOLE DIAMETER:	8-inch hollow stem auger	**HOLE ELEVATION:**	---
SAMPLER:	D = 3" OD, 2½" ID Split-spoon X = 2½" OD, 2" ID Split-spoon I = Standard Penetrometer (2" OD SPT) S = Slough in sample	**GROUND WATER DEPTH:**	**Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SANDY SILTY CLAY: Brown, moist, hard	CL-ML	1										
		2	S D D	24	4.5+			7		111		
		3										
CLAYEY SAND: Brown, dry to moist, dense; fine to coarse sand	SC	4	S D D	44				9		118		
		5										
		6										
		7										
		8										
CLAYEY SAND with GRAVEL: Light brown, dry to moist, dense; fine to coarse sand, with fine to coarse gravel	SC	9	S D D	34				7		122		
		10										
		11										
		12										
		13										
POORLY GRADED SAND with CLAY and GRAVEL: Brown, dry to moist, very dense; fine to coarse sand, with mostly fine gravel	SP-SC	14	S D D	56								
		15										
		16										
		17										
POORLY GRADED GRAVEL with CLAY and SAND: Brown, dry to moist, very dense; mostly fine gravel, with fine to coarse sand	GP-GC	18										
		19	S D D	53								
BOTTOM OF HOLE = 20 Feet		20										
No groundwater encountered												

APPENDIX B

LABORATORY TEST RESULTS

ATTERBERG LIMITS

Summary Report

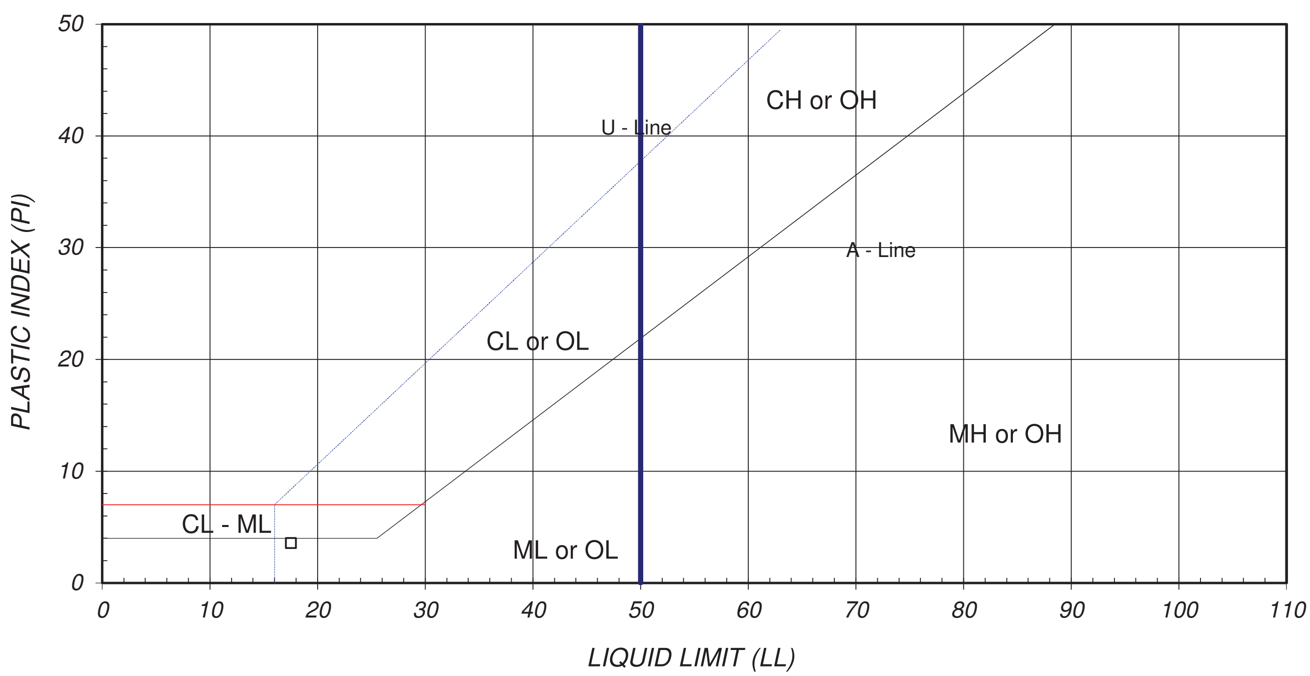
ASTM D-4318

Client : Robson Homes LLC
Project No: PA21.1017.00
Lab Log No.: **4763AF**

Project Name: 1975 Cambrianna Drive
Report Date: August 19, 2021

LSN	SYMBOL	SAMPLE IDENTIFICATION	SAMPLE DESCRIPTION	LIQUID LIMIT	PLASTIC LIMIT	PLASTIC INDEX
4763AF	□	DH-4 2-2.5	brown sandy silty clay (CL-ML)	18	14	4

PLASTICITY CHART



This testing is based upon accepted industry practice as well as the test method listed. These results apply only to the samples supplied and tested for the above referenced job.

L : Labexcel \ Projects \ Client \ Robson Homes LL \ PA21.1017.00

Print Date: 08/19/21
Entered By: PP
Reviewed By: MK
LLN: **4763AF**

DCN: PI-rp (rev. 9/18/12)

FIGURE B-1

PARTICLE SIZE ANALYSIS

Test Report

ASTM D-6913 / D-7928, (replacing D-422)

Method A: (+/-1%)

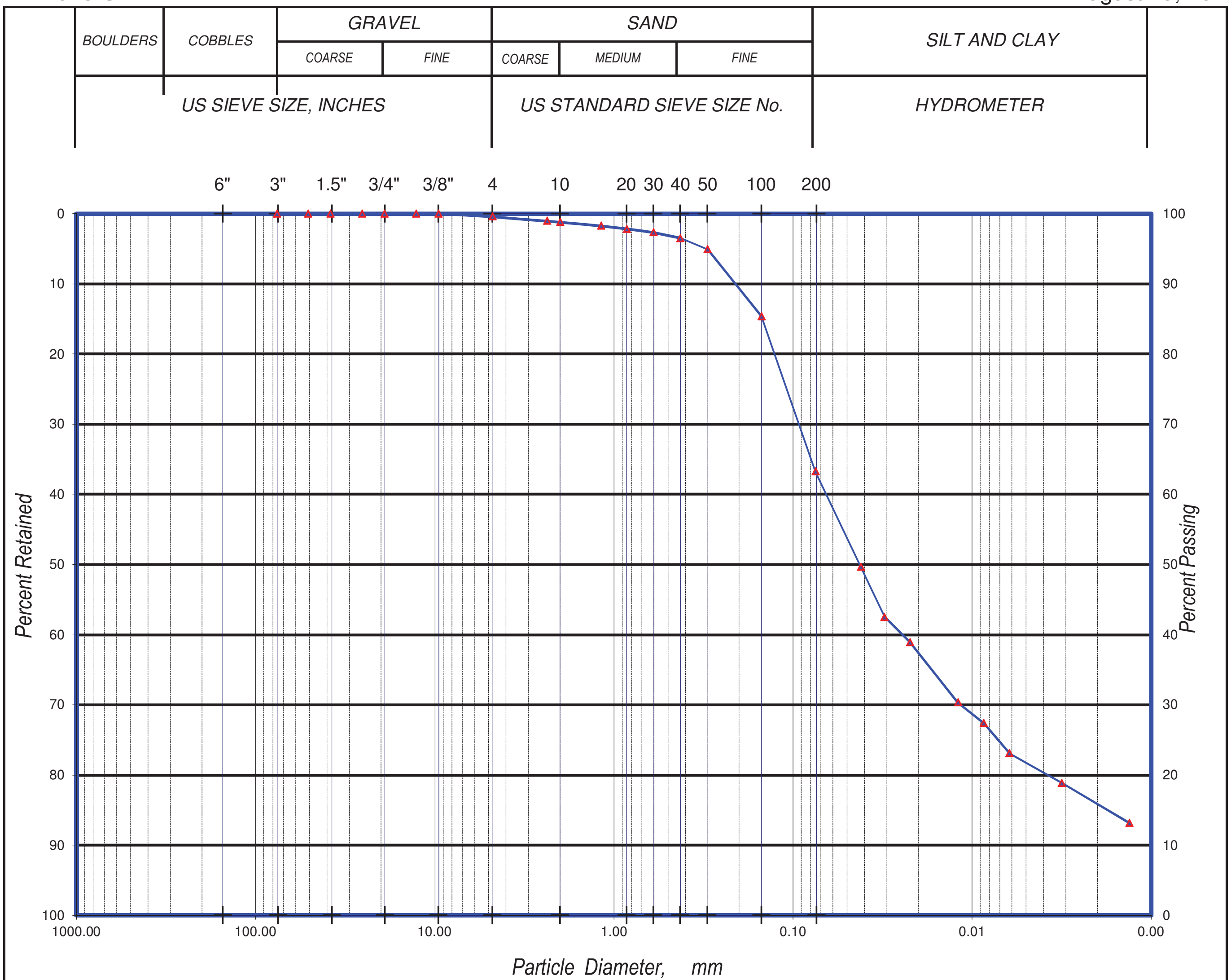
Client : ROBSON HOMES LLC

Project No: PA21.1017.00

Lab Sample No: **4763AF**

Project Name: 1975 CAMBRIANNA DRIVE

Report Date: August 19, 2021



Symbol	Sample ID	Description	% Gravel	% Sand	% Silt - Clay
▲	DH-4 2-2.5	brown sandy silty clay (CL-ML)	0.4	36.3	63.2

Size Passing, mm D_{60} = 0.07 D_{30} = 0.01 D_{10} = N/A 5 micron (%) = 23

Coefficient of Curvature, C_c: N/A Coefficient of Uniformity, C_u: N/A Fineness Modulus = 0.26

Note: * Percentages are +/- 0.1% based on computer rounding as allowed by ASTM D-6026-01 Section 5.2.3.

This testing is based upon accepted industry practice as well as the test method listed. These results apply only to the samples supplied and tested for the above referenced job.

L : Labexcel \ Projects \ Client \ Client Name \ 4763 \ 4763AF-ma

Print Date: 08/19/21 Entered By: PP Reviewed By: MK LSN: **4763AF**

DCN: MA-rp (rev. 6/27/12)

FIGURE B-2

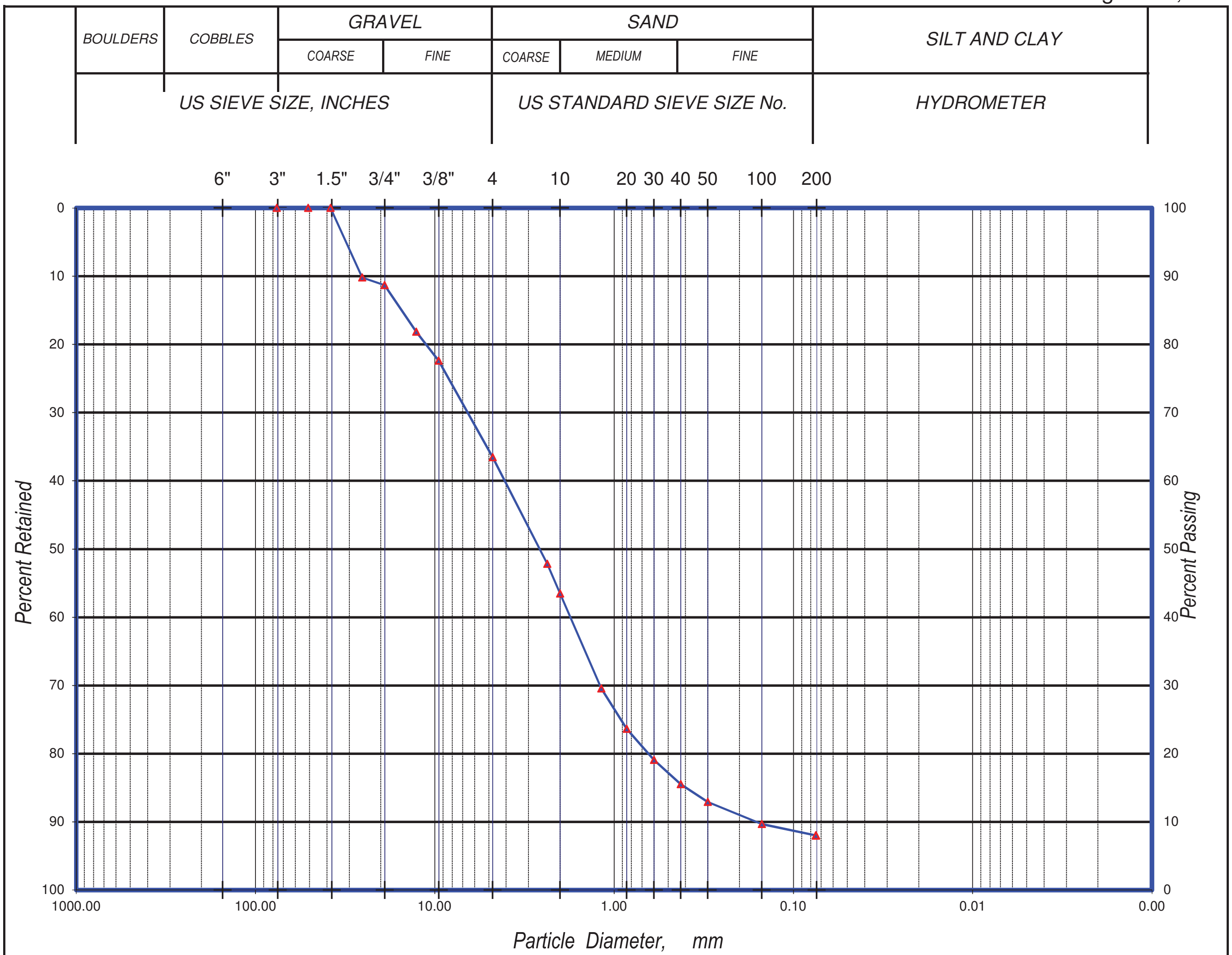
PARTICLE SIZE ANALYSIS

Test Report

ASTM D-6913 / D-7928, (replacing D-422)
Method A: (+/-1%)

Client :	Project No:	Lab Sample No:
ROBSON HOMES LLC	PA21.1017.00	**4763AL**

Project Name:	Report Date:
1975 CAMBRIANNA DRIVE	August 18, 2021



Symbol	Sample ID	* Description	% Gravel	% Sand	% Silt - Clay
▲	DH-4 14.5-15	brown well-graded sand w/ clay and gravel	36.6	55.4	8.0

Size Passing, mm D_{60} = 4.22 D_{30} = 1.21 D_{10} = 0.16

Coefficient of Curvature, C_c: 2.09 Coefficient of Uniformity, C_u: 25.61 Fineness Modulus = 4.51

* Visual Classification based on ASTM D-2488

Note: * Percentages are +/- 0.1% based on computer rounding as allowed by ASTM D-6026-01 Section 5.2.3.

This testing is based upon accepted industry practice as well as the test method listed. These results apply only to the samples supplied and tested for the above referenced job.

L : Labexcel \ Projects \ Client \ Client Name \ 4763 \ 4763AL-ma	Print Date:	Entered By:	Reviewed By:	LSN:
DCN: MA-rp (rev. 6/27/12)	08/19/21	PP	MK	**4763AL**

FIGURE B-3

PARTICLE SIZE ANALYSIS

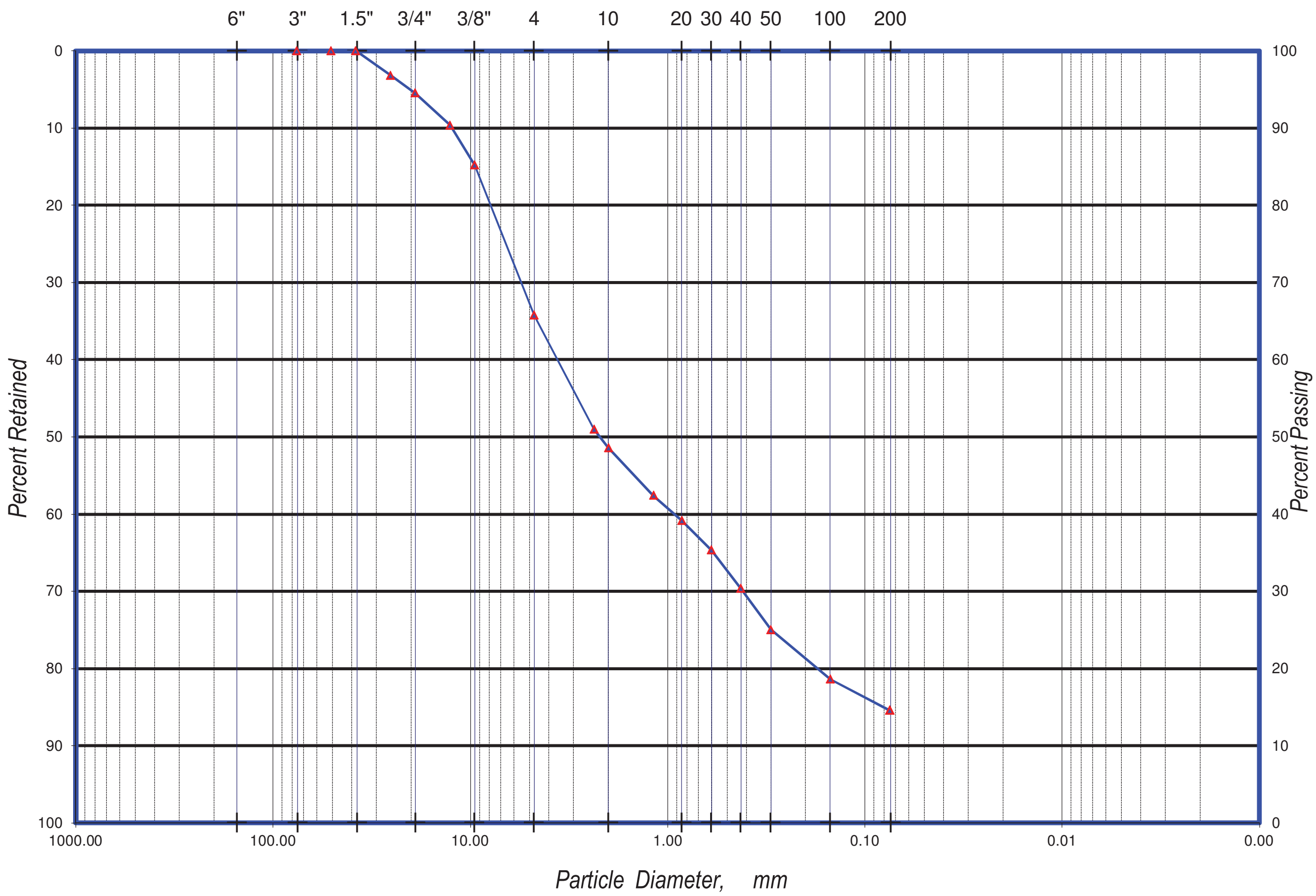
Test Report

ASTM D-6913 / D-7928, (replacing D-422)
Method A: (+/-1%)

Client :	Project No:	Lab Sample No:
ROBSON HOMES LLC	PA21.1017.00	**4763BF**

Project Name:	Report Date:
1975 CAMBRIANNA DRIVE	August 16, 2021

BOULDERS	COBBLES	GRAVEL		SAND			SILT AND CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	
US SIEVE SIZE, INCHES				US STANDARD SIEVE SIZE No.			HYDROMETER



Symbol	Sample ID	* Description	% Gravel	% Sand	% Silt - Clay
▲	DH-5 9.5-10	brown silty, clayey sand w/ gravel	34.3	51.1	14.6

Size Passing, mm D_{60} = 3.82 D_{30} = 0.42 D_{10} = N/A

Coefficient of Curvature, C_c: N/A Coefficient of Uniformity, C_u: N/A Fineness Modulus = 3.82

* Visual Classification based on ASTM D-2488

Note: * Percentages are +/- 0.1% based on computer rounding as allowed by ASTM D-6026-01 Section 5.2.3.

This testing is based upon accepted industry practice as well as the test method listed. These results apply only to the samples supplied and tested for the above referenced job.

L : Labexcel \ Projects \ Client \ Client Name \ 4763 \ 4763BF-ma	Print Date:	Entered By:	Reviewed By:	LSN:
DCN: MA-rp (rev. 6/27/12)	08/17/21	PP	MK	**4763BF**

FIGURE B-4

PARTICLE SIZE ANALYSIS

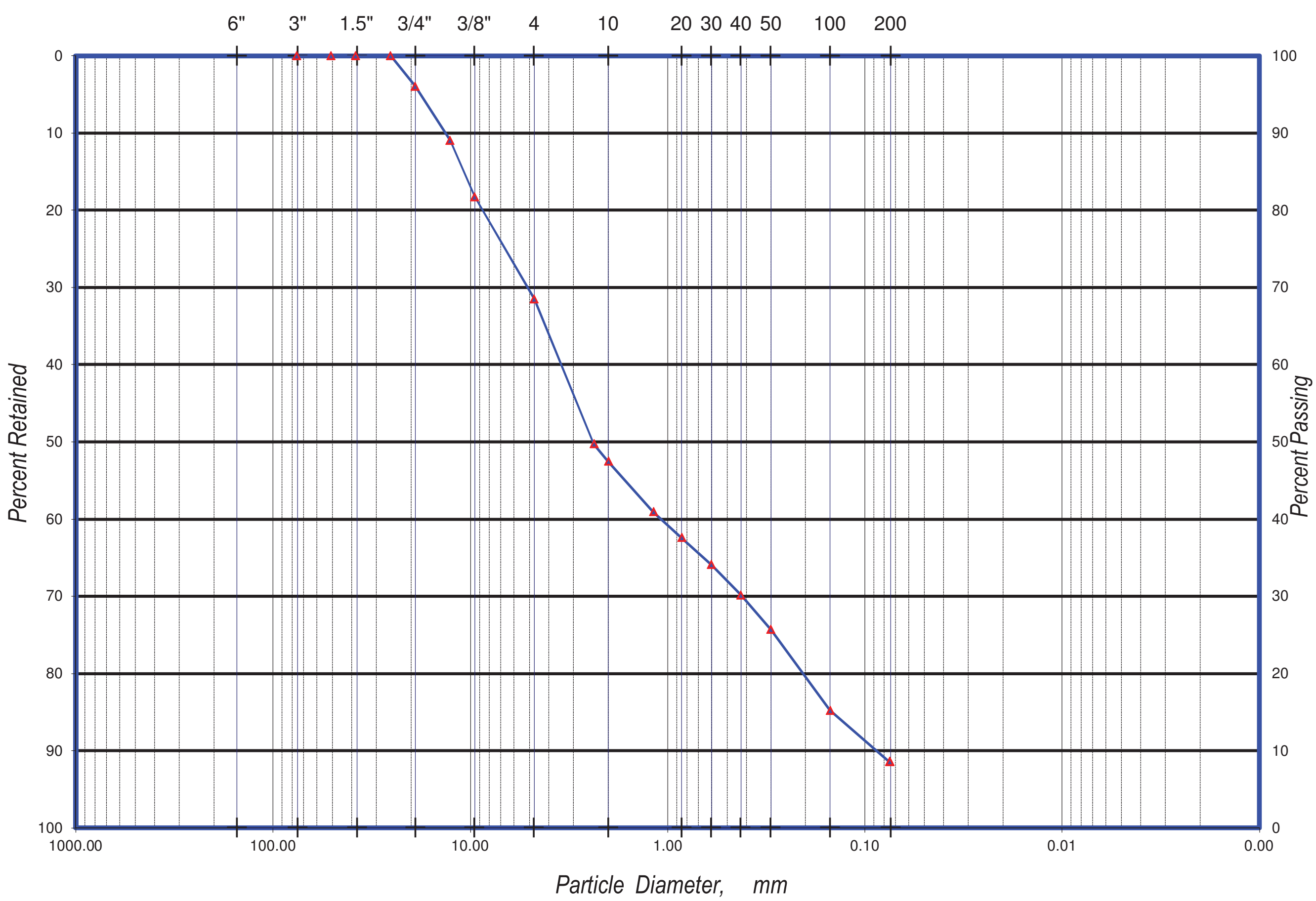
Test Report

ASTM D-6913 / D-7928, (replacing D-422)
Method A: (+/-1%)

Client :	Project No:	Lab Sample No:
ROBSON HOMES LLC	PA21.1017.00	**4763BH**

Project Name:	Report Date:
1975 CAMBRIANNA DRIVE	August 16, 2021

BOULDERS	COBBLES	GRAVEL		SAND			SILT AND CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	
US SIEVE SIZE, INCHES				US STANDARD SIEVE SIZE No.			HYDROMETER



Symbol	Sample ID	* Description	% Gravel	% Sand	% Silt - Clay
▲	DH-5 14.5-15	brown poorly graded sand w/ clay and gravel	31.5	59.9	8.5

Size Passing, mm D_{60} = 3.67 D_{30} = 0.42 D_{10} = 0.09

Coefficient of Curvature, C_c: 0.53 Coefficient of Uniformity, C_u: 40.19 Fineness Modulus = 3.88

* Visual Classification based on ASTM D-2488

Note: * Percentages are +/- 0.1% based on computer rounding as allowed by ASTM D-6026-01 Section 5.2.3.

This testing is based upon accepted industry practice as well as the test method listed. These results apply only to the samples supplied and tested for the above referenced job.

L : Labexcel \ Projects \ Client \ Client Name \ 4763 \ 4763BH-ma	Print Date:	Entered By:	Reviewed By:	LSN:
DCN: MA-rp (rev. 6/27/12)	08/17/21	PP	RZS	**4763BH**

FIGURE B-5

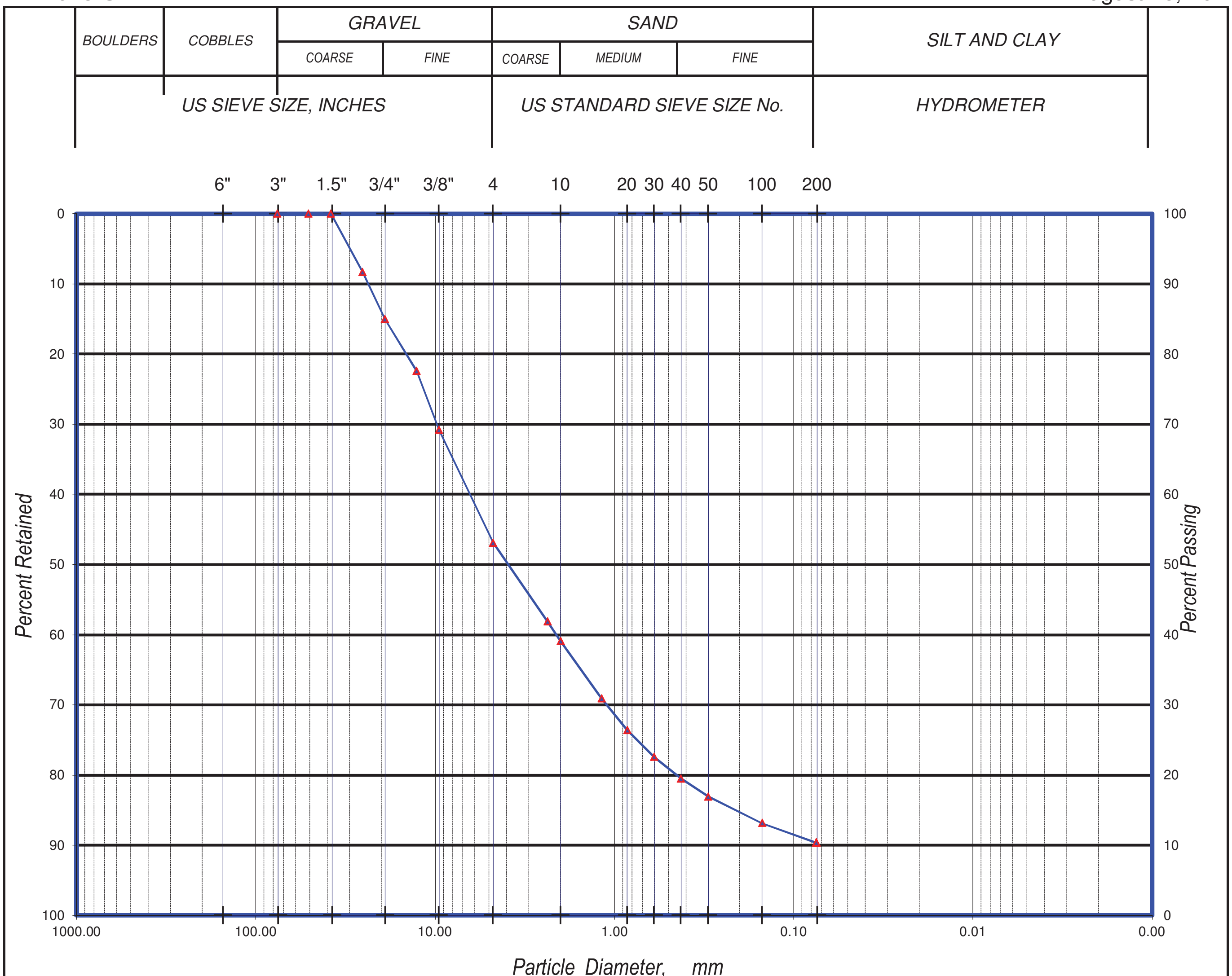
PARTICLE SIZE ANALYSIS

Test Report

ASTM D-6913 / D-7928, (replacing D-422)
Method A: (+/-1%)

Client :	Project No:	Lab Sample No:
ROBSON HOMES LLC	PA21.1017.00	**4763BJ**

Project Name:	Report Date:
1975 CAMBRIANNA DRIVE	August 16, 2021



Symbol	Sample ID	* Description	% Gravel	% Sand	% Silt - Clay
▲	DH-5 19.5-20	brown poorly graded gravel w/ clay and sand	46.9	42.7	10.4

Size Passing, mm D_{60} = 6.80 D_{30} = 1.11 D_{10} = N/A

Coefficient of Curvature, C_c: N/A Coefficient of Uniformity, C_u: N/A Fineness Modulus = 4.67

* Visual Classification based on ASTM D-2488

Note: * Percentages are +/- 0.1% based on computer rounding as allowed by ASTM D-6026-01 Section 5.2.3.

This testing is based upon accepted industry practice as well as the test method listed. These results apply only to the samples supplied and tested for the above referenced job.

L : Labexcel \ Projects \ Client \ Client Name \ 4763 \ 4763BJ-ma	Print Date:	Entered By:	Reviewed By:	LSN:
DCN: MA-rp (rev. 6/27/12)	08/17/21	PP	RZS	**4763BJ**

FIGURE B-6

'R' VALUE CA 301

Project 1975 Cambrianna Dr

Date: 8/14/21

By: LD

Job #: PA21.1017

Sample : On Site R Value

Soil Type: Brown, Clayey Sand w. Gravel

TEST SPECIMEN		A	B	C	D
Compactor Air Pressure	psi	**160**	**350**	**250**	
Initial Moisture Content	%	**5.8**	**5.8**	**5.8**	
Water Added	ml	**50**	**38**	**45**	
Moisture at Compaction	%	10.2	9.2	9.8	
Sample & Mold Weight	gms	**3219**	**3188**	**3210**	
Mold Weight	gms	**2103**	**2075**	**2096**	
Net Sample Weight	gms	1116	1113	1114	
Sample Height	in.	**2.523**	**2.47**	**2.501**	
Dry Density	pcf	121.6	125.1	123.0	
Pressure	lbs	**2965**	**6670**	**4230**	
Exudation Pressure	psi	236	531	337	
Expansion Dial	x 0.0001	**0**	**13**	**6**	
Expansion Pressure	psf	0	56	26	
Ph at 1000lbs	psi	**26**	**18**	**22**	
Ph at 2000lbs	psi	**50**	**35**	**42**	
Displacement	turns	**5.65**	**4.12**	**5.4**	
R' Value		49	68	57	
Corrected 'R' Value		**49**	**68**	**57**	

FINAL 'R' VALUE	
By Exudation Pressure (@ 300 psi):	**54**
By Epansion Pressure :	**N/A**
TI = 5	

Geo-Logic ASSOCIATES

FIGURE B-7

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

30 August, 2021

Job No. 2108037
Cust. No. 10854

Mr. Beeson Liang
Geo-Logic Associates
1175 Branham Lane, Suite #36222
San Jose, CA 95118

Subject: Project No.: PA21.1017.00
Project Name: Bore Holes
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Liang:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on August 24, 2021. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations are none detected and 31 mg/kg and are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations are none detected and 19 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils are 7.05 and 6.76, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are 360 and 380-mV, and are indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Sherri Moore

For, J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Geo-Logic Associates
Client's Project No.: PA21.1017.00
Client's Project Name: Bore Holes
Date Sampled: 23-Aug-21
Date Received: 24-Aug-21
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 30-Aug-2021

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2108037-001	DH-1 + DH-2 + DH-3 @ 4-4.5'	360	7.05	-	7,100	-	N.D.	N.D.
2108037-002	DH-4 @ 1.5-2' + DH-6 @ 4-4.5'	380	6.76		2,300		31	19

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	27-Aug-2021	27-Aug-2021	-	26-Aug-2021	-	27-Aug-2021	27-Aug-2021

Sherri Moore
for Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis
N.D. - None Detected
(1) Detection limit is elevated to 75 mg/kg due to dilution

Quality Control Summary - All laboratory quality control parameters were found to be within established limits