Geotechnical Investigation

E
APPENDIX

GEOTECHNICAL INVESTIGATION
DESIGN PHASE

FOR
2370 SHORE ROAD
HOLLISTER, SAN BENITO COUNTY, CALIFORNIA

PREPARED FOR
HIPPO HARVEST
PROJECT NO. 24-261-SB

PREPARED BY

BUTANO GEOTECHNICAL ENGINEERING, INC.
FEBRUARY 2025

BUTANO GEOTECHNICAL ENGINEERING, INC.
404 WESTRIDGE DRIVE, WATSONVILLE, CA 95076
PHONE: 831.724.2612
WWW.BUTANOGEOTECH.COM

February 18, 2025
Project No. 24-261-SB

Hippo Harvest
3439 Cloverdale Road
Pescadero, CA 94060

ATTENTION: Wes Hopkins

SUBJECT: **GEOTECHNICAL INVESTIGATION - DESIGN PHASE**
Proposed Agricultural Development
2370 Shore Road (APNs 013-05-024 and 025
Hollister, San Benito County, California

Dear Mr. Hopkins:

In accordance with your authorization, we have completed a geotechnical investigation for the subject project. This report summarizes the findings, conclusions, and recommendations from our field exploration, laboratory testing, and engineering analysis. It is a pleasure being associated with you on this project. If you have any questions, or if we may be of further assistance, please do not hesitate to contact our office.

Sincerely,

BUTANO GEOTECHNICAL ENGINEERING, INC.

GREG BLOOM
C.E. 58819
CIVIL

Greg Bloom, PE, GE
Principal Engineer

Appendices: 1. Appendix A Figures and Standard Details
2. Appendix B Field Exploration Program
3. Appendix C Laboratory Testing Program
4. Appendix D Seismic Analysis Program

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed agricultural construction located at 2370 Shore Road in unincorporated Hollister, San Benito County, California.

The purpose of our investigation is to provide preliminary geotechnical design parameters and recommendations for the proposed construction. Conclusions and recommendations related to site grading, drainage, pavement and foundations are presented herein.

This work included site reconnaissance, subsurface exploration, soil sampling, laboratory testing, engineering analyses, and preparation of this report. The scope of services for this investigation is outlined in our agreement dated December 9, 2024.

The recommendations contained in this report are subject to the limitations presented in Section 8.0 of this report. The Association of Engineering Firms Practicing the Geosciences has produced a pamphlet for your information titled *Important Information About Your Geotechnical Report*. This pamphlet has been included with the copies of your report.

2.0 PROJECT DESCRIPTION

Based on our discussions with the client it is our understanding that the project consists of constructing greenhouses, a processing building, a driveway and parking area, and an on-site wastewater treatment system (OSWTS). The proposed improvements are shown on the attached boring site plan, Figure B-2.

3.0 FIELD EXPLORATION AND LABORATORY TESTING PROGRAMS

Our field exploration program included drilling, logging, and interval sampling of eleven borings on January 16, 17, and 22, 2023. The borings were advanced with hollow stem augers (B1), solid stem augers (B2 through B10), and a 3-inch hand auger (B11). B1 through B4 were advanced with a truck and B5 through B10 were advanced with a tractor. The borings were advanced to depths ranging from 3 ½ to 41 ½ feet below existing grade. Details of the field exploration program, including the Boring Logs, and the Key to the Logs are presented in Appendix B, Figures B-3 through B-14.

Representative samples obtained during the field investigation were taken to the laboratory for testing. Laboratory tests were used to determine physical and engineering properties of the in-situ soils. Details of the laboratory testing program are presented in Appendix C. Test results are presented on the Boring Logs and in Appendix C.

4.0 SITE DESCRIPTION

4.1 Location

The project site is located east of Highway 25 at 2370 Shore Road in unincorporated Hollister, San Benito County, California. The site location is shown on the Site Location Plan, Appendix B, Figure B-1.

4.2 Surface Conditions

The parcels are approximately 80 acres in size combined. Parcel 013-05-024 is rectangular in shape and 013-05-025 is trapezoidal in shape. Parcel 013-05-025 is relatively flat and is currently used for row crops. The majority of 013-05-024 is used for raising row crops and is flat. The southeastern portion of parcel 013-05-024 is developed with a single-family residence, barn and other structures.

4.3 Subsurface Conditions

The site is geologically mapped as alluvium (alluvium-Qa). The soil encountered at the site is consistent with the geologic mapping.

Boring B1 and B2 were advanced near the location of the proposed processing building. The boring encountered stiff dark brown lean clay in the upper 2 to 3 feet. Tan lean clay was then encountered to a depth of 5 feet. Loose to medium dense sandy silt was encountered from a depth of 5 to 9 feet. The silt has low plasticity and the sand is very fine. Soft to firm lean clay, fat clay, and sandy lean clay were encountered from 9 feet to the maximum depth drilled of 41 ½ feet.

Borings B3 through B10 were drilled scattered around the parcels. The borings generally encountered soft to firm lean clay and loose to medium dense sandy silt and silty sand were encountered in these borings to the maximum depth drilled of 12 feet.

Boring B11 was advanced in the proposed parking lot and encountered dark brown lean clay in the upper foot underlain by sandy silt to a depth of 3 ½ feet.

Groundwater was encountered in every boring except for B11. The depth to groundwater varied from 5 ½ to 9 feet across the site. There may be variation in groundwater depth due to seasonal changes.

Complete soil profiles are presented on the Boring Logs, Appendix B, Figures B-4A through B-14. The boring locations are shown on the Boring Site Plan, Appendix B, Figure B-2.

5.0 GEOTECHNICAL HAZARDS

5.1 General

In our opinion the geotechnical and geologic hazards that could potentially affect the proposed project are:

- Intense seismic shaking
- Collateral seismic hazards
- Liquefaction
- Faulting

5.1.1 Intense Seismic Shaking

The hazard of intense seismic shaking is present throughout central California. Intense seismic shaking may occur at the site during the design lifetime of the proposed structure from an earthquake along one of the regions many faults. Generally, the intensity of shaking will increase the closer the site is to the epicenter of an earthquake, however, seismic shaking is a complex phenomenon and may be modified by local topography and soil conditions. The transmission of earthquake vibrations from the ground into the structure may cause structural damage.

The County of San Benito has adopted the seismic provisions set forth in the 2022 California Building Code to address seismic shaking. The seismic provisions in the 2022 CBC are minimum load requirements for the seismic design of the proposed structure(s). The provisions set forth in the 2022 CBC will not prevent structural and nonstructural damage from direct fault ground surface rupture, coseismic ground cracking, liquefaction and lateral spreading, seismically induced differential compaction, seismically induced landsliding, or seismically induced inundation.

Table 1 has been constructed based on the 2022 CBC requirements as adopted from the ASCE 7-16 provisions for the seismic design of proposed structure built in the liquefiable areas. The Site Class has been determined based on our field investigation and laboratory testing. The soil underlying the site is a Type S_F (potentially liquefiable) soil. But, the proposed structure has a short period. Based on the exception in section 20.3.1 of the ASCE 7-16 it is appropriate to analyze the site as being underlain by Type S_E (soft soil) with respect to ground shaking. Specific analysis, recommendations, and discussion with respect to liquefaction are provided in section 5.1.3, the discussion and Appendix D.

Table 1. Seismic Design Parameters

S_S	S_1	Site Class	F_a*	F_V*	S_{DS}*	S_{D1}*	F_{PGA}	PGA_M	Risk Category	Seismic Design Category*
2.29	0.855	E	Null	Null	Null	Null	1.1	1.051	II	Null

Latitude: 36.9573216 Longitude: -121.8011190

*Site specific analysis required for site class E and building structures having a period within the velocity domain of the design response spectrum (T_s<T<=T_L).

5.1.2 Collateral Seismic Hazards

In addition to intense seismic shaking, other seismic hazards that may have an adverse affect to the site and/or the structure are: fault ground surface rupture, coseismic ground cracking, seismically induced liquefaction and lateral spreading, seismically induced differential compaction, seismically induced landsliding, and seismically induced inundation (tsunami and seiche). It is our opinion that the potential for collateral seismic hazards to affect the site and to damage the proposed structure is low except for liquefaction and faulting. See section 5.1.3 for a detailed discussion on liquefaction and lateral spreading and section 5.1.4 for a discussion on faulting.

5.1.3 Liquefaction

Liquefaction is a mechanism of ground failure resulting from increased pore pressures during undrained cyclical shearing of saturated cohesionless soils. The excess pore water pressure causes the effective stress and shear resistance to drop, producing a liquefied soil state. In general terms, when loose, saturated, coarse-grained soils (cohesionless soils) are subjected to earthquake shaking (cyclical shearing) the water pressure (pore pressure) increases. If the water cannot escape fast enough (undrained) stress is transferred from the soil skeleton to the pore water, reducing the grain to grain contact (effective stress). This reduces the soil strength (shear resistance) and if the reduction is great enough the soil deforms and is said to liquefy. Ultimately, sands and non-plastic silts consolidate when subjected to repeated liquefaction.

The processing building site was analyzed for liquefaction potential utilizing the data in the most recent publications of the NCEER Workshop and SP 117 implementations. The analysis was performed for existing grade elevations using a peak ground acceleration of 1.051 g and a magnitude 7.9 earthquake.

The associated hazards related to liquefaction are:

- Loss of bearing capacity.
- Lateral spreading.
- Ground settlement.
- Surface manifestations of underlying liquefaction.

A liquefaction analysis was conducted on borings B1 and B2. The lowest estimated factor of safety against liquefaction using the NCEER method was 0.45. Due to the low factor of safety the site should be mitigated for the hazards of liquefaction.

The borings advanced in the areas of the proposed greenhouses also indicate a potential for liquefaction to occur within the depths explored. A detailed liquefaction analysis was not performed for the greenhouses as they are not considered habitable structures.

Loss of Bearing Capacity

Liquefaction can cause loss of bearing capacity in otherwise stable strata located above the liquefied zone. The loss of bearing capacity has historically resulted in large translational and rotational failures. No recognized analytical methods to evaluate the loss of bearing capacity are available at this time. The liquefiable zone is moderate (approximately 5 feet thick) and 5 feet below existing grade. Due to the shallow potential for liquefaction the probability of loss of bearing capacity to occur at the site is moderate to high.

Lateral Spreading

Lateral spreading occurs when a liquefied soil mass fails toward an open slope face or fails on an inclined plane. There is no near open slope face at the site so the potential for lateral spreading to occur at the site is low.

Ground Settlement

The liquefied soil profile may settle as a result of a seismic event. This occurs as the soil grains are shaken into a slightly denser arrangement. Because sites rarely have homogeneous soil profiles this often results in differential settlement, also known as differential compaction. The probability of ground settlement occurring at the site due to a seismic event is high.

Settlements were calculated based on cyclic stress ratio (CSR) and fines adjusted blow counts ($(N_1)_{60cs}$). The total settlement calculated was 1.5 inches. Differential settlement at the site is anticipated to be approximately 0.75 inches. The full liquefaction analysis is presented in Appendix D.

Surface Manifestations of Underlying Liquefaction

Boils and ground fissures may occur through the top non-liquefiable soil from liquefaction of the underlying soil layers. Evaluation of the potential for surface manifestations was conducted using a 5 feet thick non-liquefiable surface layer and a 5 feet thick liquefiable layer. Based on our calculations the potential for surface manifestations to occur at the site is moderate to high.

5.1.4 Faulting

The eastern parcel is partially located within the Calaveras Fault Zone per the Alquist Prieto Fault Map. Although the Calaveras Fault generally traverses a northwest to southeast alignment, a splinter off the main fault is mapped crossing parcel 013-05-024 in a northeast to southwest direction. The presence and effects of this portion of the Calaveras Fault is discussed in the following paper:

Geology and Natural History of the San Francisco Bay Area: a 2001 NAGT Field-Trip Guidebook, Field Trip 6- The Calaveras and San Andreas Faults In and Around Hollister, Harden, Stenner and Blatz

Based on the paper, a creepmeter was installed in the driveway of the residence. The creepmeter recorded an average of 6.5 and 12 mm of movement per year. The exact timeline that the measurements were taken is unknown, but is suspected to be in the 1970's.

Our firm has overlayed the Calaveras Fault zone onto the Boring Site Plan, Figure B-2. The fault zone is highlighted in yellow. The processing building and parking lot are located outside of the fault zone. The existing single-family residences and some of the proposed greenhouses in the southeastern portion of the project are within the fault zone. There is potential for creep movement within the fault zone. This movement could cause distress to structures from either discrete faulting during a seismic event or slow creep movement which will cause offsets to structures and utilities.

If the client would like to further investigate the hazard of faulting at the site, a licensed geologist should be consulted.

6.0 DISCUSSIONS AND CONCLUSIONS

The site is underlain by potentially liquifiable soil. During our site-specific investigation, potentially liquefiable soil was encountered. The primary effect on the proposed structures will be loss of bearing capacity and differential settlement on the order of 1.5 inches.

Several foundation options are provided for the processing building. These include a structural mat slab foundation or a deep foundation (drilled piers and helical piers). The mat slab foundation will be stiff enough to structurally withstand a reduction in bearing capacity and differential settlement resulting from a seismic event. The structure may need to be re-levelled after a seismic event. The deep foundation options will penetrate through any potentially liquefiable zones and the structure will not be impacted by liquefaction.

The following recommendations are intended to keep the habitable structures (processing building) from collapsing. Significant architectural and structural damage could occur during a design earthquake.

The majority of the site will be covered by greenhouses. The greenhouses are considered to be non-habitable and therefore can be designed without considering seismic conditions. These structures could experience damage or destruction caused by the effects of liquefaction during a seismic event. **The client should understand and accept this risk.**

In addition to the risk of liquefaction, a portion of the property is within the Calaveras Fault zone. Greenhouses placed within the fault zone could be subject to damage or destruction caused by discrete faulting during a seismic event or gradual creep movement. **The client should understand and accept this risk.** A licensed geologist should be consulted if the client would like a detailed study of the fault.

7.0 RECOMMENDATIONS

7.1 General

Based on the results of our field investigation, laboratory testing, and engineering analysis it is our opinion that from the geotechnical standpoint, the subject site will be suitable for the proposed construction.

The client should understand the risks described in section 5 of the report before continuing the project.

7.2 Site Grading

7.2.1 Site Clearing

The site should be cleared of any organic laden topsoil, very loose soil, non-engineered fill, existing foundation elements, decommissioned utilities, deleterious material and debris within the project limits. **The site is currently used for agricultural row crops and the upper 18 to 24 inches is tilled and laden with organics.**

7.2.2 Preparation of On-Site Soils

Areas to receive fill should be over-excavated through organic laden and tilled topsoil, scarified in the upper 8 inches, moisture conditioned to within two percent of optimum and compacted to a minimum of 90 percent relative compaction. Scarification and compaction should extend a minimum of three feet beyond the foundation zone and two feet beyond paved areas.

Engineered fill should be mechanically compacted to a minimum of 90 percent relative compaction per ASTM D1557. Fill should be placed in thin lifts not to exceed 8 inches. See Paved Areas and Utilities Section for additional compaction requirements.

The on-site soil may be re-used as engineered fill.

On-Site and Imported Fill

Imported fill material should be approved by a representative of Butano Geotechnical Engineering, Inc. prior to importing. On-site and imported fill should be primarily granular with no material greater than 2½ inches in diameter and no more than 20 percent of the material passing the #200 sieve. The fines fraction of the fill should not consist of expansive material. The Geotechnical Engineer should be notified not less than 5 working days in advance of placing any fill or base course material proposed for import. Each proposed source of import material should be sampled, tested, and approved by the Geotechnical Engineer prior to delivery of any soils imported for use on the site.

Any surface or subsurface obstruction, or questionable material encountered during grading, should be brought immediately to the attention of the Geotechnical Engineer for proper processing as required.

Paved Areas

The upper 6 inches of subgrade and all aggregate baserock in paved areas should be compacted to a minimum of **95 percent** relative compaction. This should extend a minimum of 2 feet laterally of all paved areas.

7.2.3 Cut and Fill Slopes

No cut or fill slopes are planned for the project.

7.2.4 Excavating Conditions

The on-site soil may be excavated with standard earthwork equipment.

7.2.5 Surface Drainage

Positive drainage should be maintained away from the structures at a minimum gradient of 3 percent for 10 feet. Roof and driveway drainage should be collected into solid plastic pipe and released at approved locations.

7.2.6 Utilities

Utility trenches should be backfilled based on the County of San Benito standard details. At a minimum this should consist of 4 inches of bedding sand below the utility and 8 inches of bedding sand above the utility.

Backfill of all exterior and interior trenches should be placed in thin lifts not to exceed 8 inches and mechanically compacted to achieve a relative compaction of not less than 95 percent in paved areas and 90 percent in other areas per ASTM D1557. Care should be taken not to damage utility lines.

The on-site native soils may be utilized for trench backfill above the bedding sand. If sand or granular material is used for trench backfill, a 3-foot concrete plug should be placed in each trench where it passes under the exterior footings.

Utility trenches that are parallel to the sides of a building should be placed so that they do not extend below a line sloping down and away at an inclination of 2:1 H:V from the bottom outside edge of all footings.

Trenches should be capped with 1 1/2 feet of relatively impermeable material. Import material must be approved by the Geotechnical Engineer prior to its use.

Trenches must be shored as required by the local regulatory agency, the State of California Division of Industrial Safety Construction Safety Orders, and Federal OSHA requirements.

Utilities should be designed using appropriate flexible connections for differential settlement of up to 0.75 inches. Emergency shut-off valves/switches should also be incorporated.

7.3 Foundations (Processing Building-Habitable Structures)

The proposed structures may be founded on a structural mat.

7.3.1 Structural Mat

General

A reinforced concrete structural mat foundation system may be used when designed to span voids, withstand differential settlement, and allow the structure to move as a single unit.

Design Parameters

The structural mat should be designed and constructed to span a 6 foot diameter void appearing anywhere beneath the structure. The structural mat should be designed for 0.75 inches of differential vertical settlement anywhere under the structure.

The allowable bearing capacity used should not exceed 300 psf for slabs bearing on 24 inches of engineered fill per section 7.2.2. The allowable bearing capacity may be increased by one-third in the case of short duration loads, such as those induced by wind (**but not seismic forces**). In the event that footings are founded in structural fill consisting of imported materials, the allowable bearing capacities will depend on the type of these materials and should be re-evaluated.

A modulus of subgrade reaction equal to 40 pci may be used for design.

Friction coefficient - 0.30, between the engineered fill and rough concrete. If a thickened edge is used for increased lateral resistance a passive resistance of 200 pcf may be assumed below a depth of 12 inches. Where both friction and the passive resistance are utilized for sliding resistance, either of the values indicated should be reduced by one-third.

The structural slab-on-grade should be underlain by a vapor barrier and capillary break per section 7.3.5.

7.3.2 Drilled Piers

The processing building may be supported by a pier and grade beam foundation system that penetrates through any potentially liquefiable soil and is founded in the underlying lean and fat clay.

The drilled, cast-in-place concrete shafts, should have a minimum embedment depth of **20** feet, measured from existing grade**.** The minimum recommended shaft diameter is 18 inches. Shafts should be spaced no closer than 2.5 diameters, with a minimum of 3.0 diameters, center to center.

An allowable skin adhesion of 250 psf may be assumed with a 1/3 increase for short term loading. An allowable passive resistance of 250 pcf over 2 pier diameters may be assumed below a depth of 10 feet. Passive resistance should be ignored to a depth of 10 feet from existing grade.

The upper 5 feet of soil should be designed for negative skin friction of 300 psf/ft to account for the downdrag effects of soil caused by a seismic event.

The drilled excavations for the cast-in-place concrete shafts should be clean, dry and free of debris or loose soil. The drilled excavations should not deviate more than 1 percent from vertical.

For drilled, cast-in-place concrete shafts depths in excess of 8 feet, or where water is present, concrete should be placed via a tremie. The end of the tremie pipe must remain embedded a minimum of 4 feet into the concrete at all times and shall start at the base of the shaft.

It is anticipated that the drilled pier excavations may cave during construction. The pier holes should be cased or drilling fluid should be introduced to prevent collapse.

7.3.3 Helical Piers

As an option, the processing building may be supported by helical piers.

We recommend that the ADU foundation be embedded into the underlying medium dense silty sand. The depth required is to embed the helix below the active zone.

Helical Piers

Helix anchors may be used to support vertical loads. Battered and/or concrete reinforced helix anchors may be used to support lateral loads.

The minimum recommended embedment depth for the helix plate (on a multi plated helix anchor) is **15** feet below the base of the grade beam. An allowable bearing/pull-out capacity of 2,750 psf may be used to determine the number and size of the helix plates that will be needed for each anchor to achieve the required minimum service load. The total allowable bearing/pull-out capacity of each anchor is calculated by summing up the bearing/pull-out capacity of each helix plate on the anchor.

These capacities do not include the weight of the shaft. Allowable capacities may be increased by one-third for short term loading. The minimum recommended helix depth to diameter ratio is 5. Individual helix plates attached to a multi-plate anchor should be spaced no closer than 3 diameters. Anchors should be placed no closer than 5 diameters center-to-center, with the diameter of the largest helix plate being used to determine the spacing.

Rotational resistance encountered by an anchor when being screwed into the soil defined as installation torque. The monitoring of installation torque during installation is recommended. Installation torque should not exceed the anchor rating. Installation torque has been empirically related to pull-out capacity. A minimum bearing/pull-out capacity to installation torque ratio of 10 is generally recommended, and this ratio is subject to verification in the field.

Installation tolerances should be within 2±% with regards to plumbness and to within 2± inches in location. We recommend that all helix anchors be protected with galvanized coating. In general, installation procedures should be per the manufacturer's specifications.

All anchor installation must be observed and approved by the Geotechnical Engineer. Any anchors installed without the full knowledge and continuous observation of Butano Geotechnical Engineering, Inc. will render the recommendations of this report invalid.

7.3.4 Conventional Shallow Foundations (greenhouses and other non-habitable structures)

General

The proposed greenhouses and other non-habitable structures may be supported on conventional shallow foundations bearing on a minimum of 24-inches of engineered fill per section 7.2.2.

Footing excavations must be checked by the Geotechnical Engineer before steel is placed and concrete is poured.

The client should understand that any structures founded on a conventional shallow foundation could be subject to bearing failure and differential settlement during a seismic event. If this is not acceptable, the structures should be founded per sections 7.3.1, 7.3.2, or 7.3.3.

Footing Dimensions

Footing widths should be based on the allowable bearing value but not less than 15 inches. The minimum recommended depth of embedment is 12 inches into engineered fill per Section 7.2.2. The engineered fill should extend a minimum of 36 inches laterally of the footing. Embedment depths should not be allowed to be affected adversely, such as through erosion, softening, digging, etc. Should local building codes require deeper embedment of the footings or wider footings, the local codes must apply.

Bearing Capacity

The allowable bearing capacity used should not exceed 2,000 psf for footings bearing on engineered fill. The allowable bearing capacity may be increased by one-third in the case of short duration wind loads. There is no increase for seismic loading. In the event that footings are founded in structural fill consisting of imported materials, the allowable bearing capacities will depend on the type of these materials and should be re-evaluated.

Lateral Resistance

Friction coefficient - 0.30, between the engineered fill and concrete. A passive resistance of 250 pcf may be assumed below a depth of 12 inches for engineered fill. Where both friction and the passive resistance are utilized for sliding resistance, either of the values indicated should be reduced by one-third.

7.3.5 Concrete Slabs-on-Grade (non-structural)

General

We recommend that non-structural concrete slabs-on-grade be founded on a minimum of 24 inches of engineered fill per section 7.2.2. Engineered fill should extend a minimum of 24 inches laterally of slabs-on-grade.

It is recommended that the non-structural slabs be physically separated from the structure's foundation. This should apply to structures supported by a deep foundation

The subgrade for slab-on-grades should be kept moist prior to pouring concrete.

The subgrade should be proof-rolled just prior to construction to provide a firm, relatively unyielding surface, especially if the surface has been loosened by the passage of construction traffic.

Capillary Break and Vapor Barrier

The following paragraph outlines the minimum capillary break and vapor barrier that shall be utilized for interior slab-on-grades, or slab-on-grades where moisture sensitive floor coverings are anticipated.

The vapor barrier shall consist of a waterproof membrane (Stegowrap 15 Mil or equivalent) placed directly below the floor slab and in direct contact with the concrete. Sheet overlap for the vapor barrier shall be a minimum of 6 inches. A 4-inch minimum layer of ¾ inch drainrock shall be placed below the waterproof membrane to act as a capillary break. Care must be taken to not rip the vapor barrier. A 6-inch layer of compacted Class II Baserock may be employed to prevent rips or tears in the vapor barrier if desired, and to keep the subgrade from becoming saturated prior to pouring concrete.

If the manufacturer’s recommendations or the project requirements for the capillary break and vapor barrier are more stringent than the minimums outlined above, the designer should follow those recommendations and requirements. Recommendations by the manufacturer may include but is not limited to specifications for; concrete mix design, puncture resistance of vapor barrier, permeance of vapor barrier, soil flatness, capillary break section, structural section, and testing recommendations.

7.3.6 Settlements

Total and differential settlements beneath the proposed structure under static conditions are expected to be within tolerable limits. Vertical movement is not expected to exceed 1 inch. Differential movements are not expected to exceed ½ inch.

During a seismic event the proposed structures (and non structural slabs) may experience total and differential settlements of 1.5 inches and 0.75 inches respectively as discussed in section 5.1.3. Structures supported by deep foundations would not be subject to differential settlement from liquefaction.

7.4 Pavement Sections

7.4.1 General

Hot Mix Asphalt (HMA) should correspond to Caltrans Type A specification.

All HMA should be compacted to a minimum of 95 percent compaction based on bulk density.

All Aggregate Base (AB) shall consist of ¾” Class 2 based on Caltrans Section 26-1.

Pavement sections for a range of traffic indexes have been provided. The traffic index should be chosen by the civil engineer and the corresponding pavement section used in design.

7.4.2 Design Pavement Sections

An aggregate base with HMA may be used throughout the project.

The underlying subgrade should be processed to a minimum depth of 6 inches and compacted to 95 percent relative compactions according to section 7.2.2.

The following table should be used to design pavement sections. A design R-value of 25 should be used.

Table 2. Full Depth Replacement Sections

Traffic Index	*AB (inches)*	HMA (inches)
5.0	*7*	3
5.5	9	3
6.0	8	4
6.5	10	4

7.5 Plan Review

The recommendations presented in this report are based on preliminary design information for the proposed project and on the findings of our geotechnical investigation. When completed, the Grading Plans, Foundation Plans and design loads should be reviewed by Butano Geotechnical Engineering, Inc. prior to submitting the plans and contract bidding. Additional field exploration and laboratory testing may be required upon review of the final project design plans.

7.6 Observation and Testing

Field observation and testing should be provided by a representative of Butano Geotechnical Engineering, Inc. to enable them to form an opinion regarding the adequacy of the site preparation, the adequacy of fill materials, and the extent to which the earthwork is performed in accordance with the geotechnical conditions present, the requirements of the regulating agencies, the project specifications, and the recommendations presented in this report.

Butano Geotechnical Engineering, Inc. should be notified **at least 5 working days** prior to any site clearing or other earthwork operations on the subject project in order to observe the stripping and disposal of unsuitable materials and to ensure coordination with the grading contractor. During this period, a preconstruction meeting should be held on the site to discuss project specifications, observation and testing requirements and responsibilities, and scheduling.

8.0 LIMITATIONS

The recommendations contained in this report are based on our field explorations, laboratory testing, and our understanding of the proposed construction. The subsurface data used in the preparation of this report was obtained from the borings drilled during our field investigation. Variation in soil, geologic, and groundwater conditions can vary significantly between sample locations. As in most projects, conditions revealed during construction excavation may be at variance with preliminary findings. If this occurs, the changed conditions must be evaluated by the Project Geotechnical Engineer, and revised recommendations be provided as required. In addition, if the scope of the proposed construction changes from the described in this report, our firm should also be notified.

Our investigation was performed in accordance with the usual and current standards of the profession, as they relate to this and similar localities. No other warranty, expressed or implied, is provided as to the conclusions and professional advice presented in this report.

This report is issued with the understanding that it is the responsibility of the Owner, or of his Representative, to ensure that the information and recommendations contained herein are brought to the attention of the Engineer for the project and incorporated into the plans, and that it is ensured that the Contractor and Subcontractors implement such recommendations in the field. The use of information contained in this report for bidding purposes should be done at the Contractor’s option and risk.

This firm does not practice or consult in the field of safety engineering. We do not direct the Contractor's operations, and we are not responsible for other than our own personnel on the site; therefore, the safety of others is the responsibility of the Contractor. The Contractor should notify the Owner if he considers any of the recommended actions presented herein to be unsafe.

The findings of this report are considered valid as of the present date. However, changes in the conditions of a site can occur with the passage of time, whether they are due to natural events or to human activities on this or adjacent sites. In addition, changes in applicable or appropriate codes and standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, this report may become invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and revision as changed conditions are identified.

The scope of our services mutually agreed upon did not include any environmental assessment or study for the presence of hazardous to toxic materials in the soil, surface water, or air, on or below or around the site. Butano Geotechnical Engineering, Inc. is not a mold prevention consultant; none of our services performed in connection with the proposed project are for the purpose of mold prevention. Proper implementation of the recommendations conveyed in our reports will not itself be sufficient to prevent mold from growing in or on the structures involved.

REFERENCES

ASTM International (2006).*Annual Book of ASTM Standards, Section Four, Construction*. Volume 4.08, Soil and Rock (I): D 430 - D 5611.

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Dibblee, T.W. and Minch, J.A. (ed.), 2006, Geologic map of the San Felipe quadrangle, Santa Clara & San Benito Counties, California, Dibblee Geological Foundation, Dibblee Foundation Map DF-229, 1:24,000.

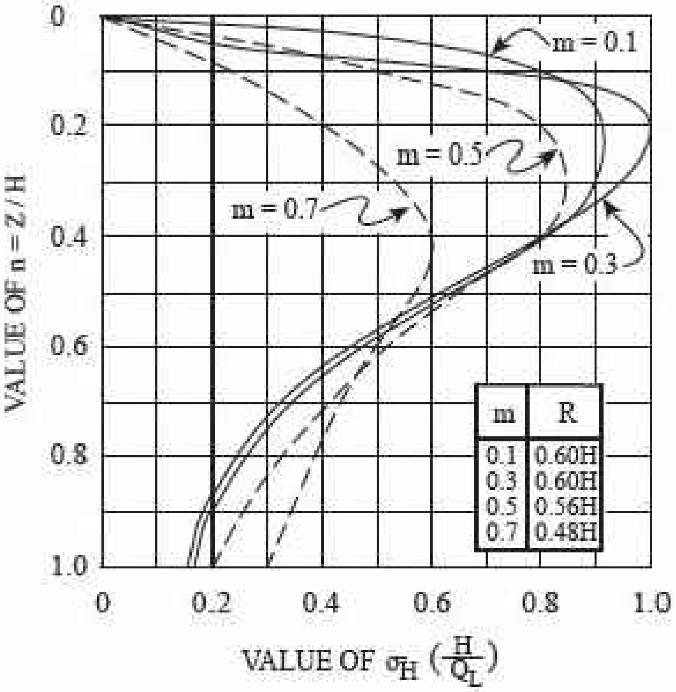
Geology and Natural History of the San Francisco Bay Area: a 2001 NAGT Field-Trip Guidebook, Field Trip 6- The Calaveras and San Andreas Faults In and Around Hollister, Harden, Stenner and Blatz

APPENDIX A

FIGURES AND STANDARD DETAILS

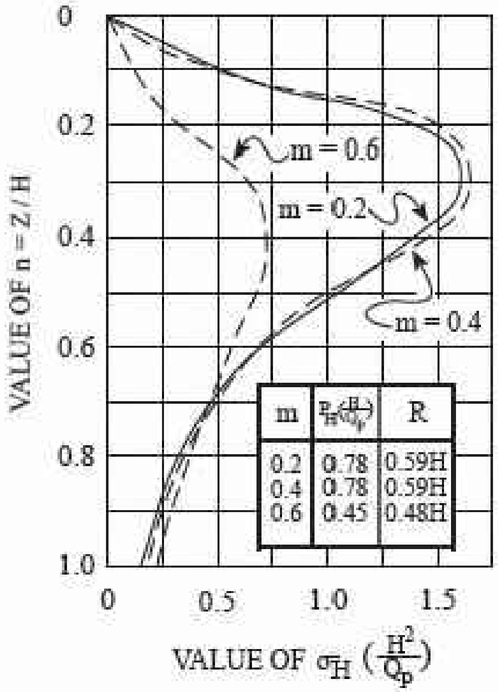
Surcharge Pressure Diagram	Figure A-1

LINE LOAD

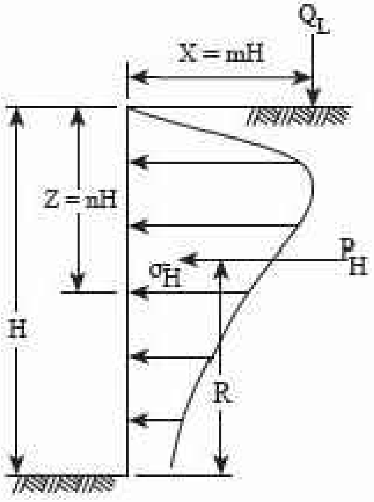


m	R
0.1	0.60H
0.3	0.60H
0.5	0.56H
0.7	0.48H

POINT LOAD



m	$P_H\left(\frac{H}{Q_p}\right)$	R
0.2	0.78	0.59H
0.4	0.78	0.59H
0.6	0.45	0.48H



FOR m ≦ 0.4:

$$\sigma_H\left(\frac{H}{Q_L}\right)=\frac{0.20\,n}{(0.16+n^2)^2}$$

$$P_H = 0.55\,Q_L$$

FOR m > 0.4:

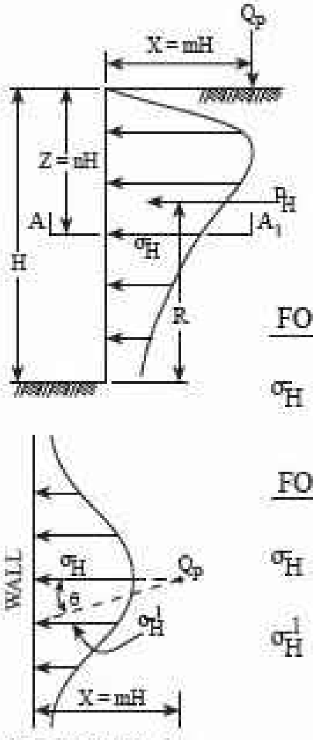
$$\sigma_H\left(\frac{H}{Q_L}\right)=\frac{1.28\,m^2 n}{(m^2+n^2)^2}$$

RESULTANT $P_H=\dfrac{0.64\,Q_L}{(m^2+1)}$

PRESSURES FROM LINE LOAD Q_L

(BOISSINESQ EQUATION MODIFIED BY EXPERMENT)

REFERENCE: Design Manual
NAVFAC DM-7.02
Figure 11
Page 7.2-74



SECTION A-A_1

FOR m ≦ 0.4:

$$\sigma_H\left(\frac{H^2}{Q_p}\right)=\frac{0.28\,n^2}{(0.16+n^2)^3}$$

FOR m > 0.4:

$$\sigma_H\left(\frac{H^2}{Q_p}\right)=\frac{1.77\,m^2 n^2}{(m^2+n^2)^3}$$

$$\sigma_H^1 = \sigma_H \cos^2(1.1\,\theta)$$

PRESSURES FROM POINT LOAD Q_P

(BOISSINESQ EQUATION MODIFIED
BY EXPERMENT)

BUTANO GEOTECHNICAL ENGINEERING, INC.	SURCHARGE PRESSURE DIAGRAM	FIGURE A-1

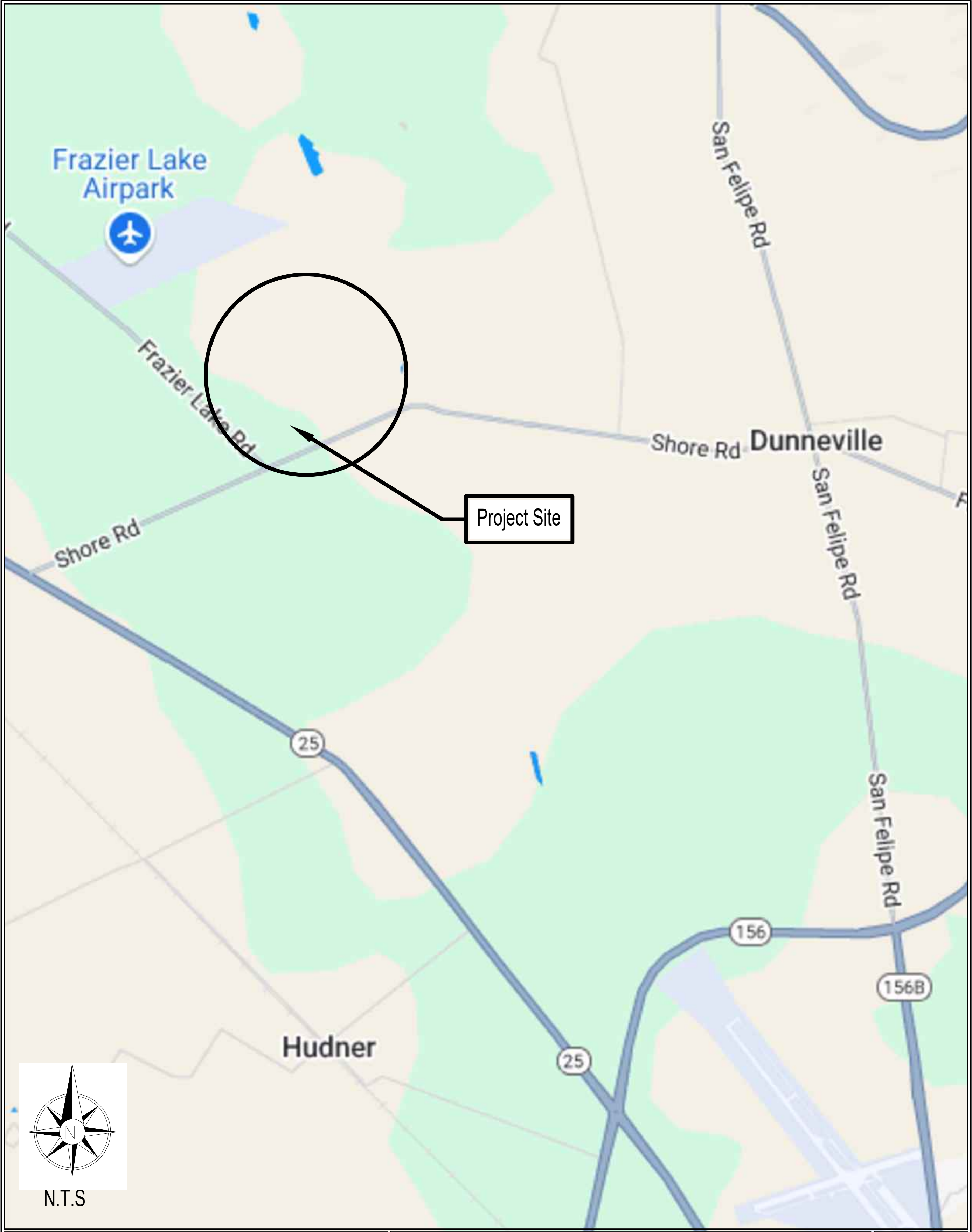
APPENDIX B

FIELD EXPLORATION PROGRAM

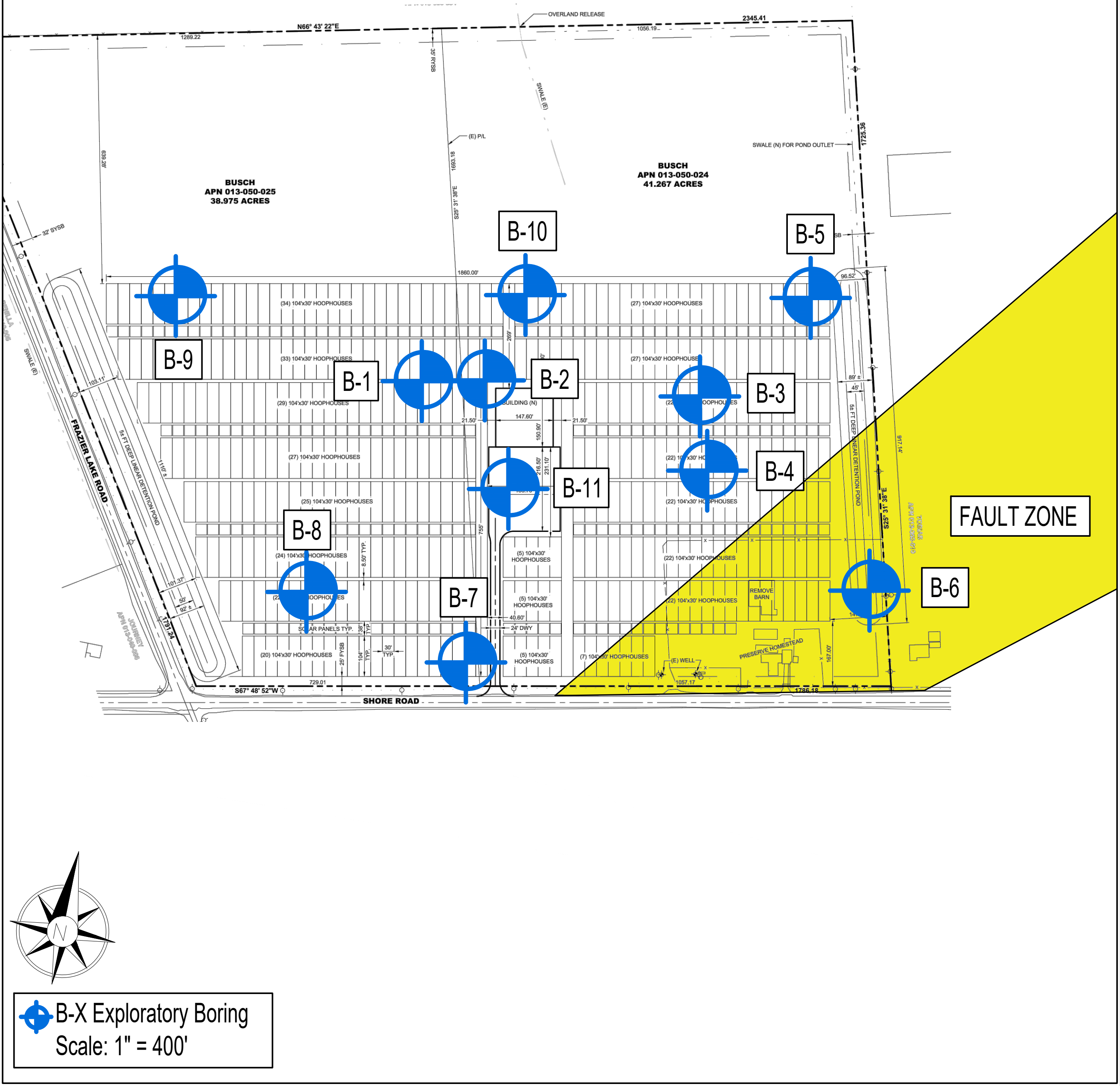
FIELD EXPLORATION PROCEDURES

Subsurface conditions were explored by advancing eleven borings below existing grade. Boring B1 was advanced using eight-inch diameter solid stem augers on a truck mounted drill rig. Borings B2 through B10 were advance using six-inch diameter solid stem augers on a truck or tractor mounted drill rig. Boring B11 was advanced with a 3-inch diameter hand auger. The Key to The Logs and the Logs of the Boring are included in Appendix B, Figures B-3 through B-14. The approximate locations of the borings are shown on the Boring Site Plan, Figure B-2. The borings were located in the field by tape measurements from known landmarks. Their locations as shown are therefore within the accuracy of such measurement.

The soils encountered in the borings were continuously logged in the field by a representative of Butano Geotechnical Engineering, Inc. Bulk and relatively undisturbed soil samples for identification and laboratory testing were obtained in the field. These soils were classified based on field observations and laboratory tests. The classifications are accordance with the Unified Soil Classification System (USCS: Figure B-3).



BUTANO	**SITE LOCATION PLAN**	FIGURE
GEOTECHNICAL ENGINEERING, INC.	2370 Shore Rd	B-1



BUTANO

GEOTECHNICAL ENGINEERING, INC.

BORING SITE PLAN

2370 Shore Rd

FIGURE

B-2

KEY TO LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS More than half of the material is larger than the No. 200 sieve	GRAVELS More than half of the coarse fraction is larger than the No. 4 sieve	CLEAN GRAVELS (Less than 5% fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS More than half of the coarse fraction is smaller than the No. 4 sieve	CLEAN SANDS (Less than 5% fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands, gravelly sands, little or no fines
		SAND WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS More than half of the material is smaller than the No. 200 sieve	SILTS AND CLAYS Liquid limit less than 50		ML	Inorganic silts and very fine sands, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS Liquid limit greater than 50		MH	Inorganic silts, micaceous or diatomacaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils

GRAIN SIZE LIMITS

SILT AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

No. 200 No. 40 No. 10 No. 4 3/4 in. 3 in. 12 in.

US STANDARD SIEVE SIZE

RELATIVE DENSITY

SAND AND GRAVEL	BLOWS/FT*
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CONSISTENCY

SILT AND CLAY	BLOWS/FT*
VERY SOFT	0 - 2
SOFT	2 - 4
FIRM	4 - 8
STIFF	8 - 16
VERY STIFF	16 - 32
HARD	OVER 32

MOISTURE CONDITION

CLAY	DRY
	MOIST
	SATURATED
SAND	DRY
	DAMP
	WET
	SATURATED

* Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1 3/8 inch I.D.) split spoon (ASTM D-1586).

BUTANO GEOTECHNICAL ENGINEERING, INC.

FIGURE B-3

LOG OF EXPLORATORY BORING

Project No.:	24-261-SB	Boring:	B1 (1 of 2)
Project:	2370 Shore Road	Location:	See Figure B-2
		Elevation:	
Date:	January 16, 2025	Method of Drilling:	8-inch Diameter Hollow Stem Augers
Logged By:	GB		Truck Mounted Drill Rig

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
◫ Terzaghi Split Spoon Sample
☒ Bulk Sample
▽ Perched Water Table
▽ Static Water Table
▼ Water Encountered During Drilling
——— Change in Soil Classification
- - - - Gradation or Minor Change in Classification

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Particle Size (% fines)	Swell Pressure(psf)	Atterberg Limits	
												L.L.	P.I.
	CL	⧅		Dark brown Lean CLAY stiff, moist	23	13	105.1	19.0					
			☒						53				
		◫		Tan, stiff, very moist, with seams of Silty SAND	9	7		28.7					
5	ML	⧅		Grades to a Sandy SILT, medium dense,									
				very damp, non-plastic, very fine sand	19	11	97.2	27.3		73			
				▽									
10	CH	◫		Gray Fat CLAY, soft to firm, saturated									
					2	2		42.0					
15		◫											
				some interbedded lenses of clayey sand	4	3		36.1					
20		◫		- - - -									
	CL			Gray Sandy Lean CLAY, stiff, fine sand	8	7		25.5					
25		◫											
					4	4		35.3					
30		◫			8	9		28.6					
35													

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FIGURE
B-4A

LOG OF EXPLORATORY BORING

Project No.: 24-261-SB
Project: 2370 Shore Road
Date: January 16, 2025
Logged By: GB
Boring: B1 (2 of 2)
Location: See Figure B-2
Elevation:
Method of Drilling: 8-inch Diameter Hollow Stem Augers
Truck Mounted Drill Rig

⧄ 2" Ring Sample ⧅ 2.5" Ring Sample ◫ Terzaghi Split Spoon Sample ⊠ Bulk Sample

Perched Water Table ▽ Static Water Table ▽ Water Encountered During Drilling ▼

Change in Soil Classification ——— Gradation or Minor Change in Classification - - - -

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Particle Size (% fines)	Unconfined - q_u (psf)	Atterberg Limits L.L.	Atterberg Limits P.I.
	CL												
40		◫		Gray Sandy Lean CLAY, stiff, saturated	7	8		37.0					
				Boring terminated at a depth of 41 1/2 feet. Goundwater was recorded at a depth of 9 feet after drilling.									
45													
50													
55													
60													
65													
70													

LOG OF EXPLORATORY BORING

Project No.: 24-261-SB

Project: 2370 Shore Road

Date: January 16, 2025

Logged By: AJ

Boring: B2

Location: See Figure B-2

Elevation:

Method of Drilling: 6-inch Diameter Solid Stem Augers
Truck Mounted Drill Rig

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
◫ Terzaghi Split Spoon Sample
⊠ Bulk Sample

Perched Water Table ▿
Static Water Table ∇
Water Encountered During Drilling ▼

Change in Soil Classification ———
Gradation or Minor Change in Classification - - - - -

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Particle Size (% fines)	Unconfined - q_u (psf)	Atterberg Limits L.L.	Atterberg Limits P.I.
	CL	⧅		Dark brown Lean CLAY, stiff, moist	20	12	103.5	16.6					
		◫		Tan, very moist	8	7		27.3					
5	ML	⧅		Tan Sandy SILT, loose, wet, low plasticity, very fine sand	4	3	97.0	27.3		59			
				∇									
10	CL	⧅		Gray Lean CLAY, firm, saturated	4	3	79.8	43.6					
15		⧅		stiff	13	9	109.5	37.8					
20				Boring terminated at a depth of 16 1/2 feet. Groundwater measured at a depth of 8 feet after drilling.									
25													
30													
35													

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FIGURE
B-5

LOG OF EXPLORATORY BORING

Project No.: 24-261-SB
Project: 2370 Shore Road
Date: January 16, 2025
Logged By: AJ
Boring: B3
Location: See Figure B-2
Elevation:
Method of Drilling: 6-inch Diameter Solid Stem Augers
Truck Mounted Drill Rig

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
◫ Terzaghi Split Spoon Sample
⊠ Bulk Sample
▽ Perched Water Table
▽ Static Water Table
▼ Water Encountered During Drilling
Change in Soil Classification ———
Gradation or Minor Change in Classification - - - - -

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Particle Size (% fines)	Unconfined - q_u (psf)	Atterberg Limits L.L.	Atterberg Limits P.I.
	CL	⧅		Gray Clayey SAND, loose, moist	14	8	95.0	24.0					
		◫		seam of tan Silty SAND, loose, damp	8	6		24.2					
5	ML	◫		Tan Sandy SILT, loose, saturated, very fine sand, non-plastic ▽	11	8		29.3		71			
10	CL	◫		Gray Lean CLAY, soft, saturated	2	1		41.5					
15				Boring terminated at a depth of 11 1/2 feet. Groundwater measured at a depth of 5 1/2 feet after drilling.									
20													
25													
30													
35													

BUTANO GEOTECHNICAL ENGINEERING, INC.

FIGURE
B-6

LOG OF EXPLORATORY BORING

Project No.:	24-261-SB	Boring:	B4
Project:	2370 Shore Road	Location:	See Figure B-2
		Elevation:	
Date:	January 16, 2025	Method of Drilling:	6-inch Diameter Solid Stem Augers
Logged By:	AJ		Truck Mounted Drill Rig

⧄ 2" Ring Sample ⧅ 2.5" Ring Sample ◫ Terzaghi Split Spoon Sample ⊠ Bulk Sample

▽ Perched Water Table ▽ Static Water Table ▼ Water Encountered During Drilling

Change in Soil Classification ——— Gradation or Minor Change in Classification - - - -

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Particle Size (% fines)	Unconfined - q_u (psf)	Atterberg Limits L.L.	Atterberg Limits P.I.
	ML	⧅		Tan Sandy SILT, firm, moist, slightly plastic, very fine sand	13	7	102.5	21.1					
	CL	◫		Tan Lean CLAY, stiff, moist	11	8		25.5					
5	ML	◫		Grades back to a Sandy SILT, (no recovery), loose ▽	6	4							
10	CL	◫		Dark gray Lean CLAY, soft, saturated	2	1		41.2					
15				Boring terminated at a depth of 11 1/2 feet. Groundwater measured at a depth of 7 feet after drilling.									
20													
25													
30													
35													

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FIGURE B-7

LOG OF EXPLORATORY BORING

Project No.:	24-261-SB	Boring:	B5
Project:	2370 Shore Road	Location:	See Figure B-2
		Elevation:	
Date:	January 17, 2025	Method of Drilling:	6-inch Diameter Solid Stem Augers
Logged By:	GB		Tractor Mounted Drill Rig

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
◫ Terzaghi Split Spoon Sample
☒ Bulk Sample
Perched Water Table ▽
Static Water Table ▽
Water Encountered During Drilling ▼
Change in Soil Classification ———
Gradation or Minor Change in Classification - - - -

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Particle Size (% fines)	Unconfined - q_u (psf)	Atterberg Limits L.L.	Atterberg Limits P.I.
	CL	◫		Brown Sandy Lean CLAY, firm, moist	6	4		13.6					
	SM	⧅		Tan Silty SAND, loose, damp, fine sand	15	5	96.5	10.9					
5		◫		loose	9	7		15.6					
		◫		medium dense, saturated ▽	14	11		27.6					
10													
15				Boring terminated at a depth of 12 feet. Groundwater measured at a depth of 8 1/2 feet after drilling.									
20													
25													
30													
35													

BUTANO GEOTECHNICAL ENGINEERING, INC.

FIGURE
B-8

LOG OF EXPLORATORY BORING

Project No.:	24-261-SB	Boring:	B6
Project:	2370 Shore Road	Location:	See Figure B-2
		Elevation:	
Date:	January 17, 2025	Method of Drilling:	6-inch Diameter Solid Stem Augers
Logged By:	GB		Tractor Mounted Drill Rig

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
◫ Terzaghi Split Spoon Sample
☒ Bulk Sample
Perched Water Table ▽
Static Water Table ▽
Water Encountered During Drilling ▼
Change in Soil Classification ———
Gradation or Minor Change in Classification - - - - -

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Particle Size (% fines)	Unconfined - q_u (psf)	Atterberg Limits L.L.	Atterberg Limits P.I.
	SM	◫		Brown Silty SAND, loose, damp	7	5		7.7					
		⧅			12	4		8.3					
5		◫			7	5		12.2					
				▽									
	ML	◫		Tan Sandy SILT, loose, saturated, slightly plastic, very fine sand	10	8		27.5		62			
10				Boring terminated at a depth of 9 1/2 feet. Groundwater measured at a depth of 7 feet after drilling.									
15													
20													
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.:	24-261-SB	Boring:	B7
Project:	2370 Shore Road	Location:	See Figure B-2
		Elevation:	
Date:	January 17, 2025	Method of Drilling:	6-inch Diameter Solid Stem Augers
Logged By:	GB		Tractor Mounted Drill Rig

⧄ 2" Ring Sample ⧅ 2.5" Ring Sample ◫ Terzaghi Split Spoon Sample ⊠ Bulk Sample

Perched Water Table ▽ Static Water Table ▽ Water Encountered During Drilling ▼

Change in Soil Classification ——— Gradation or Minor Change in Classification - - - -

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Particle Size (% fines)	Unconfined - q_u (psf)	Atterberg Limits L.L.	Atterberg Limits P.I.
	CL	◫		Dark brown Lean CLAY, firm, moist	8	6		17.8	25				
		⧅		Tan, stiff	15	8	96.2	24.4			4690		
5		◫		firm, trace fine sand	7	5		29.6					
	ML	◫		grades to a tan Sandy SILT, firm, saturated, moderately plastic ▽	7	5		32.3					
10													
15				Boring terminated at a depth of 12 feet. Groundwater measured at a depth of 9 feet after drilling.									
20													
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.: 24-261-SB
Project: 2370 Shore Road
Date: January 17, 2025
Logged By: GB
Boring: B8
Location: See Figure B-2
Elevation:
Method of Drilling: 6-inch Diameter Solid Stem Augers
Tractor Mounted Drill Rig

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
◫ Terzaghi Split Spoon Sample
☒ Bulk Sample
Perched Water Table ▽
Static Water Table ▽
Water Encountered During Drilling ▼
Change in Soil Classification ———
Gradation or Minor Change in Classification - - - - -

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Particle Size (% fines)	Unconfined - q_u (psf)	Atterberg Limits L.L.	Atterberg Limits P.I.
	CL	◫		Dark brown Lean CLAY, firm, moist	7	5		19.0					
		⧅			17	6	102.4	21.3					
5	ML	◫		Grades to a tan Sandy SILT, loose, damp, non-plastic	9	7		24.7		65			
		◫		loose, saturated ▽	6	4		35.1					
10				Boring terminated at a depth of 9 1/2 feet. Groundwater measured at a depth of 9 feet after drilling.									
15													
20													
25													
30													
35													

BUTANO GEOTECHNICAL ENGINEERING, INC.

FIGURE B-11

LOG OF EXPLORATORY BORING

Project No.:	24-261-SB	Boring:	B9
Project:	2370 Shore Road	Location:	See Figure B-2
		Elevation:	
Date:	January 17, 2025	Method of Drilling:	6-inch Diameter Solid Stem Augers
Logged By:	GB		Tractor Mounted Drill Rig

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
◫ Terzaghi Split Spoon Sample
⊠ Bulk Sample

Perched Water Table ▽
Static Water Table ▽
Water Encountered During Drilling ▼

Change in Soil Classification ———
Gradation or Minor Change in Classification - - - -

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Particle Size (% fines)	Unconfined - q_u (psf)	Atterberg Limits L.L.	Atterberg Limits P.I.
	CL	◫		Tan Lean CLAY, firm, moist	8	6		24.7					
		⧅		stiff	15	8	100.6	23.4			1560		
5	ML	◫		grades to a Sandy SILT, loose, minor plasticity, very fine sand	8	6		26.6					
				▽									
	SM	◫		grades to a Silty SAND, loose, saturated	10	8		31.9					
10				Boring terminated at a depth of 9 1/2 feet. Groundwater measured at a depth of 7 feet after drilling.									
15													
20													
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.: 24-261-SB
Boring: B10
Project: 2370 Shore Road
Location: See Figure B-2
Elevation:
Date: January 17, 2025
Method of Drilling: 6-inch Diameter Solid Stem Augers
Logged By: GB
Tractor Mounted Drill Rig

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
◫ Terzaghi Split Spoon Sample
⊠ Bulk Sample

▽ Perched Water Table
▽ Static Water Table
▼ Water Encountered During Drilling

Change in Soil Classification ———
Gradation or Minor Change in Classification - - - - -

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Particle Size (% fines)	Unconfined - q_u (psf)	Atterberg Limits L.L.	Atterberg Limits P.I.
	CL	◫		Tan Sandy Lean CLAY, stiff, moist, very fine sand	7	5		17.9					
	ML	⧅		grades to a Sandy SILT, firm, slightly plastic	12	7	91.5	22.9					
5		◫			8	5		28.4					
				▽									
	SM	◫		grades to a gray Silty SAND, medium dense, saturated	15	8		26.6					
10				Boring terminated at a depth of 9 1/2 feet. Groundwater measured at a depth of 6 1/2 feet. after drilling.									
15													
20													
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.: 24-261-SB
Boring: B11
Project: 2370 Shore Road
Location: See Figure B-2
Elevation:
Date: January 22, 2025
Method of Drilling: 3-inch diameter hand auger
Logged By: GB

▨ 2" Ring Sample ▨ 2.5" Ring Sample ◫ Terzaghi Split Spoon Sample ☒ Bulk Sample
Perched Water Table ▽ Static Water Table ▽ Water Encountered During Drilling ▼
Change in Soil Classification ——— Gradation or Minor Change in Classification - - - -

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Particle Size (% fines)	Unconfined - q_u (psf)	Atterberg Limits L.L.	Atterberg Limits P.I.
	CL			Tan Sandy Lean CLAY, stiff, moist, very fine sand									
	ML		☒	grades to a Sandy SILT, firm, slightly plastic									
5				Boring terminated at a depth of 3 1/2 feet. Groundwater not encountered during drilling.									
10													
15													
20													
25													
30													
35													

APPENDIX C

LABORATORY TESTING PROGRAM

LABORATORY TESTING PROCEDURES

Classification

Soils were classified according to the Unified Soil Classification System in accordance with ASTM D 2487 and D 2488. Moisture content and density determinations were made for representative samples in accordance with ASTM D 2216. Results of moisture density determinations, together with classifications, are shown on the Boring Logs, Figures B-3 through B-14.

Particle Size Analysis

Six sieves were performed on representative sample in accordance with ASTM C 117 and C 136. The grain size distributions from the results of the particle size analyses are shown on Figures C-1 through C-6.

Unconfined Compression

Two unconfined compression tests were performed in accordance with ASTM D 2166. The results are shown on the boring logs.

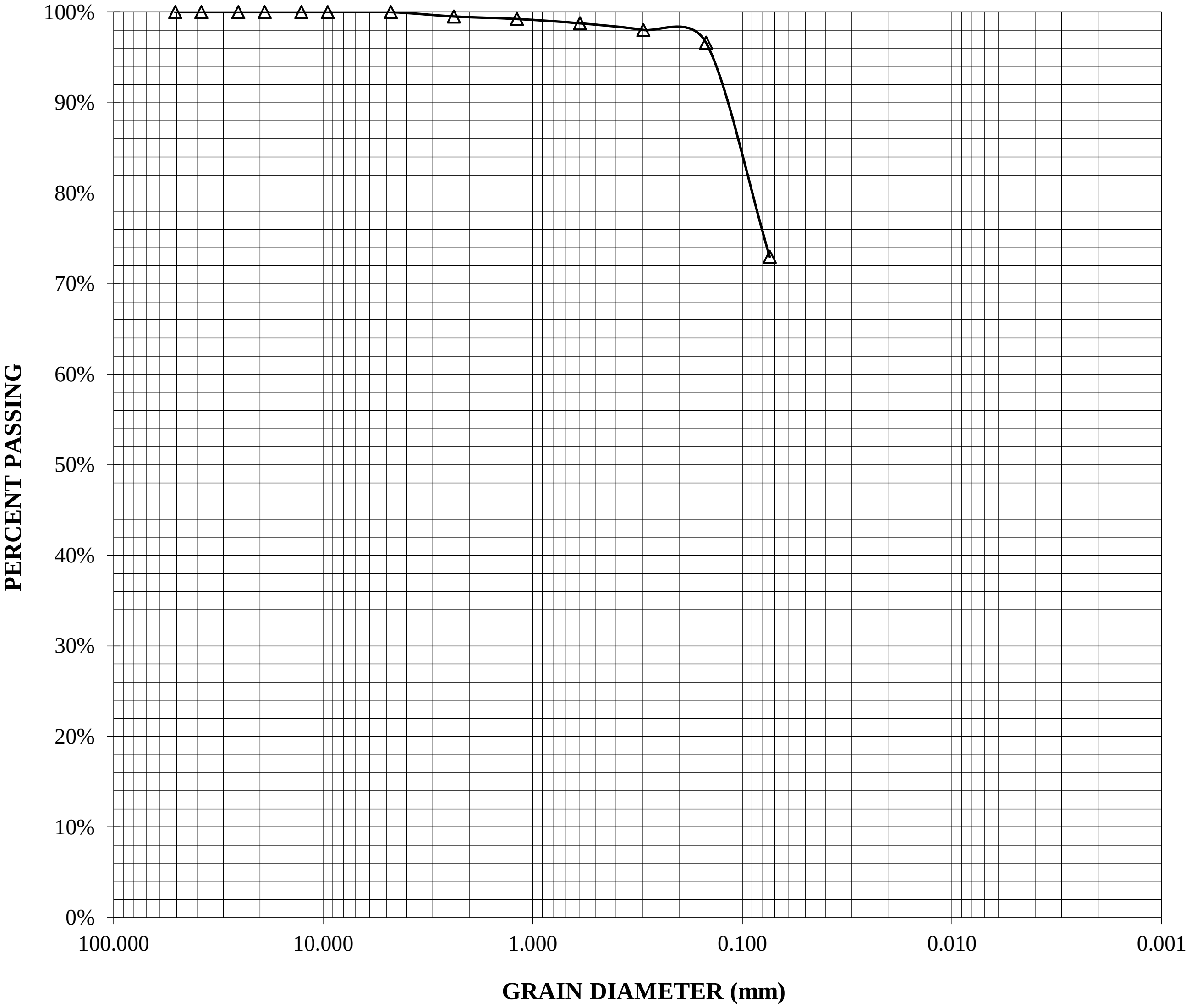
Expansion Index

Two expansion index tests were performed on a representative bulk sample of the foundation zone soil in accordance with ASTM D 4829-03. The results are shown on the Boring Logs.

R-Value Test

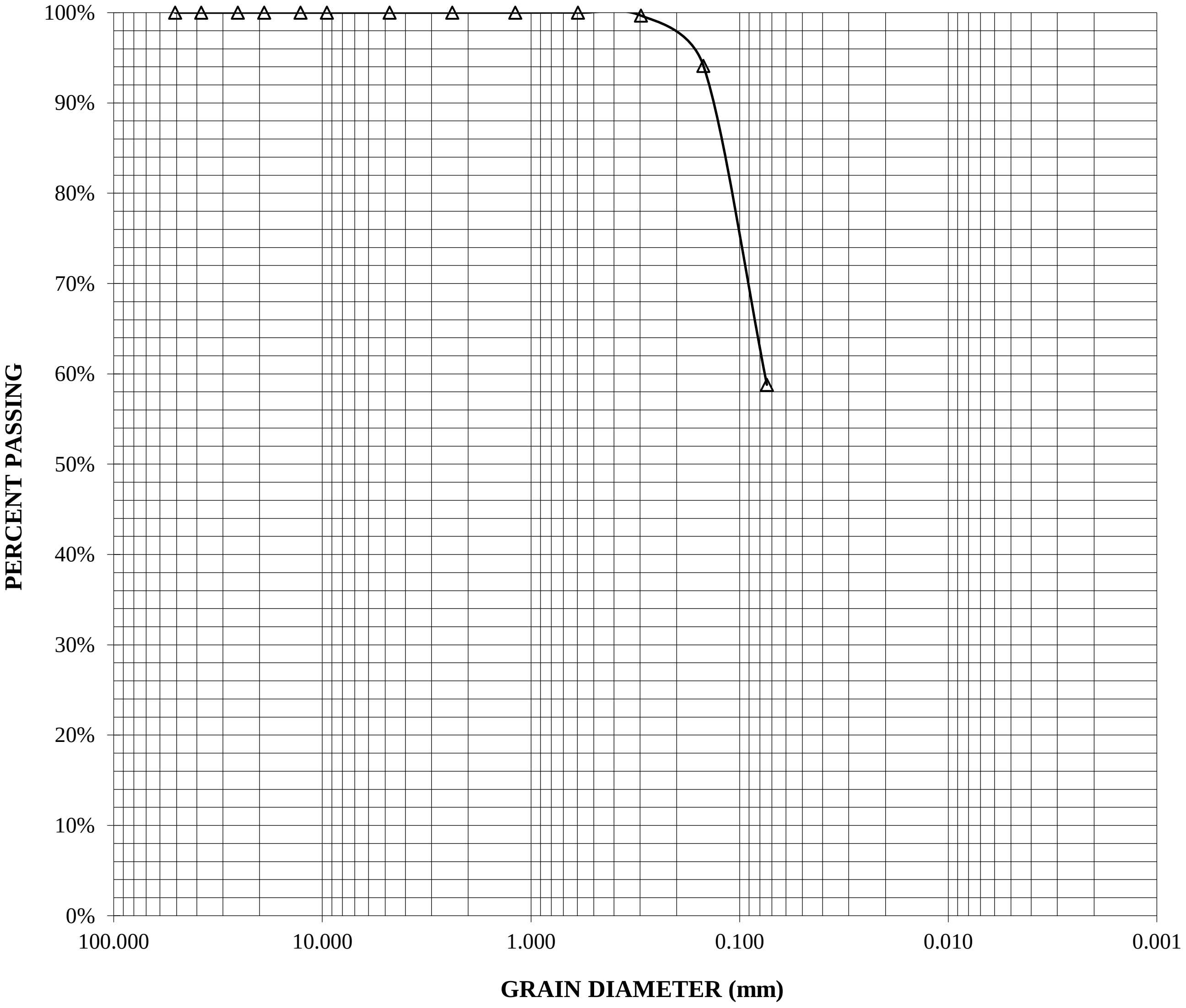
One R-Value test wase conducted on a sample of the pavement zone soil using Caltrans test method 301. The results are presented in Figures C-7.

		PERCENT PASSING No. 4	PERCENT PASSING No. 200
BORING:	B1-3		
DEPTH (ft):	5		
SOIL TYPE (USCS):	ML	100.0%	73.0%



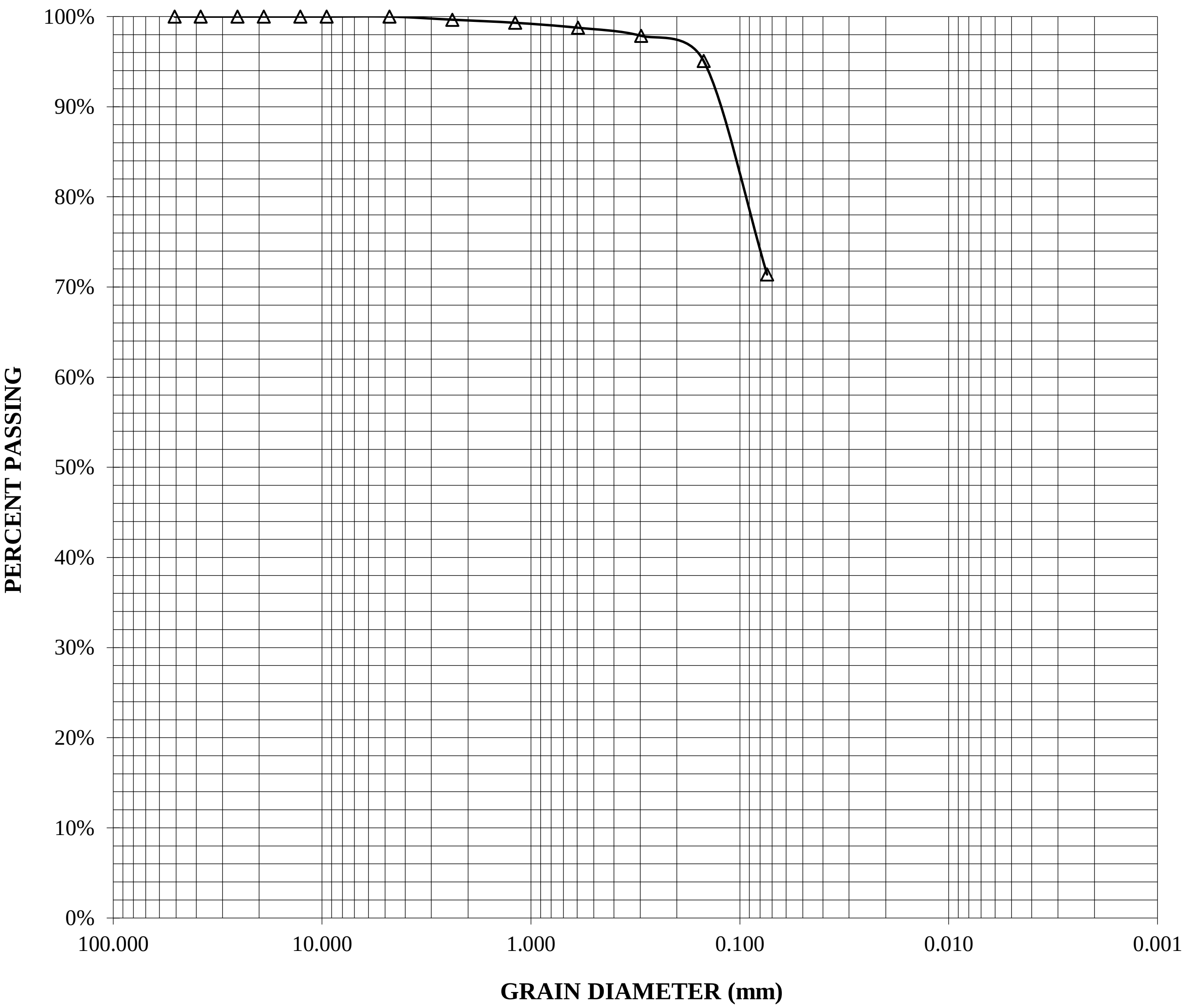
BUTANO	**GRAIN SIZE DISTRIBUTION**	FIGURE
GEOTECHNICAL ENGINEERING, INC.	2370 Shore Road	C-1

		PERCENT PASSING No. 4	PERCENT PASSING No. 200
BORING:	B2-3		
DEPTH (ft):	5		
SOIL TYPE (USCS):	ML	100.0%	58.8%



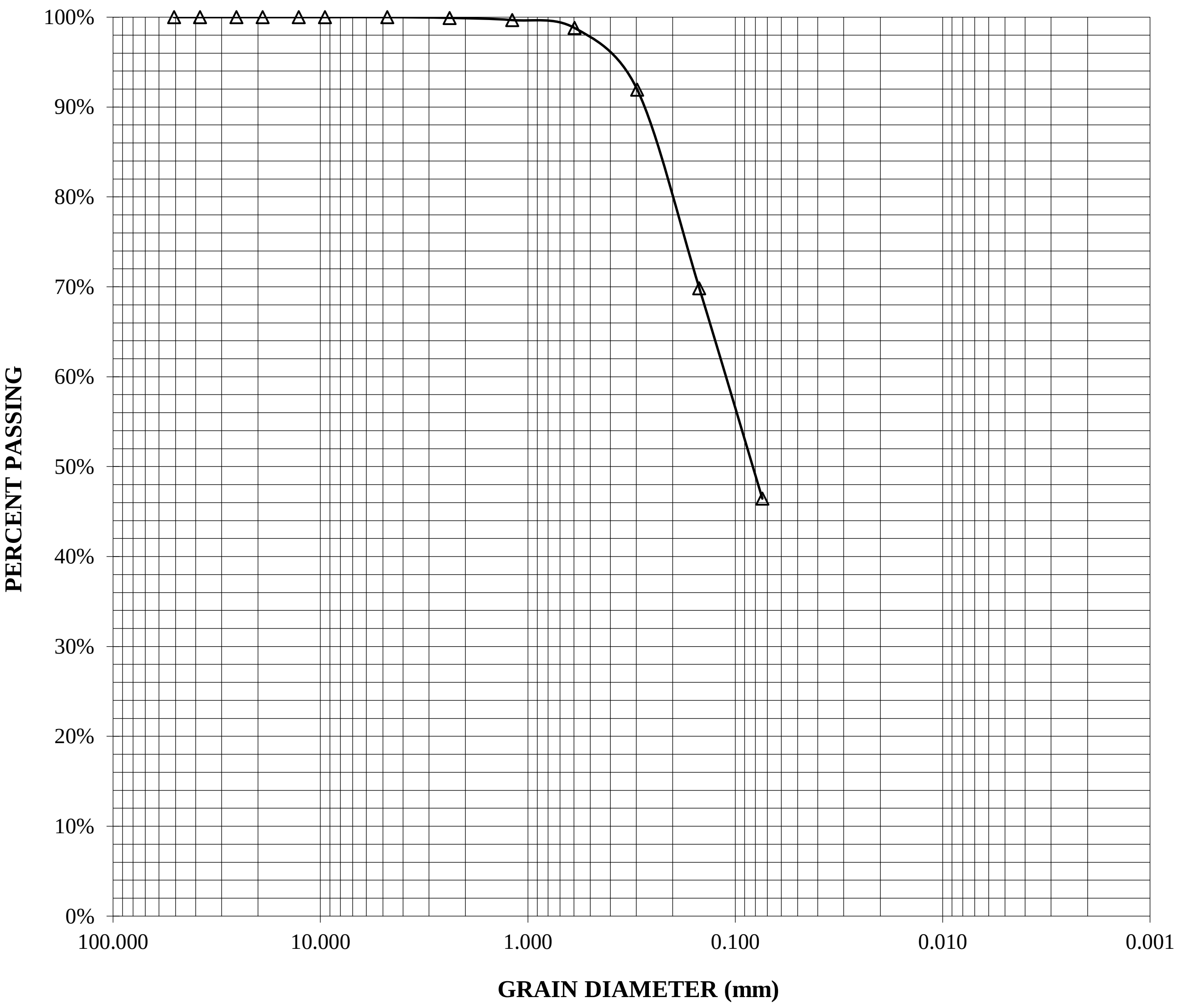
BUTANO	**GRAIN SIZE DISTRIBUTION**	FIGURE
GEOTECHNICAL ENGINEERING, INC.	2370 Shore Road	C-2

		PERCENT PASSING No. 4	PERCENT PASSING No. 200
BORING:	B3-3		
DEPTH (ft):	5		
SOIL TYPE (USCS):	ML	100.0%	71.4%



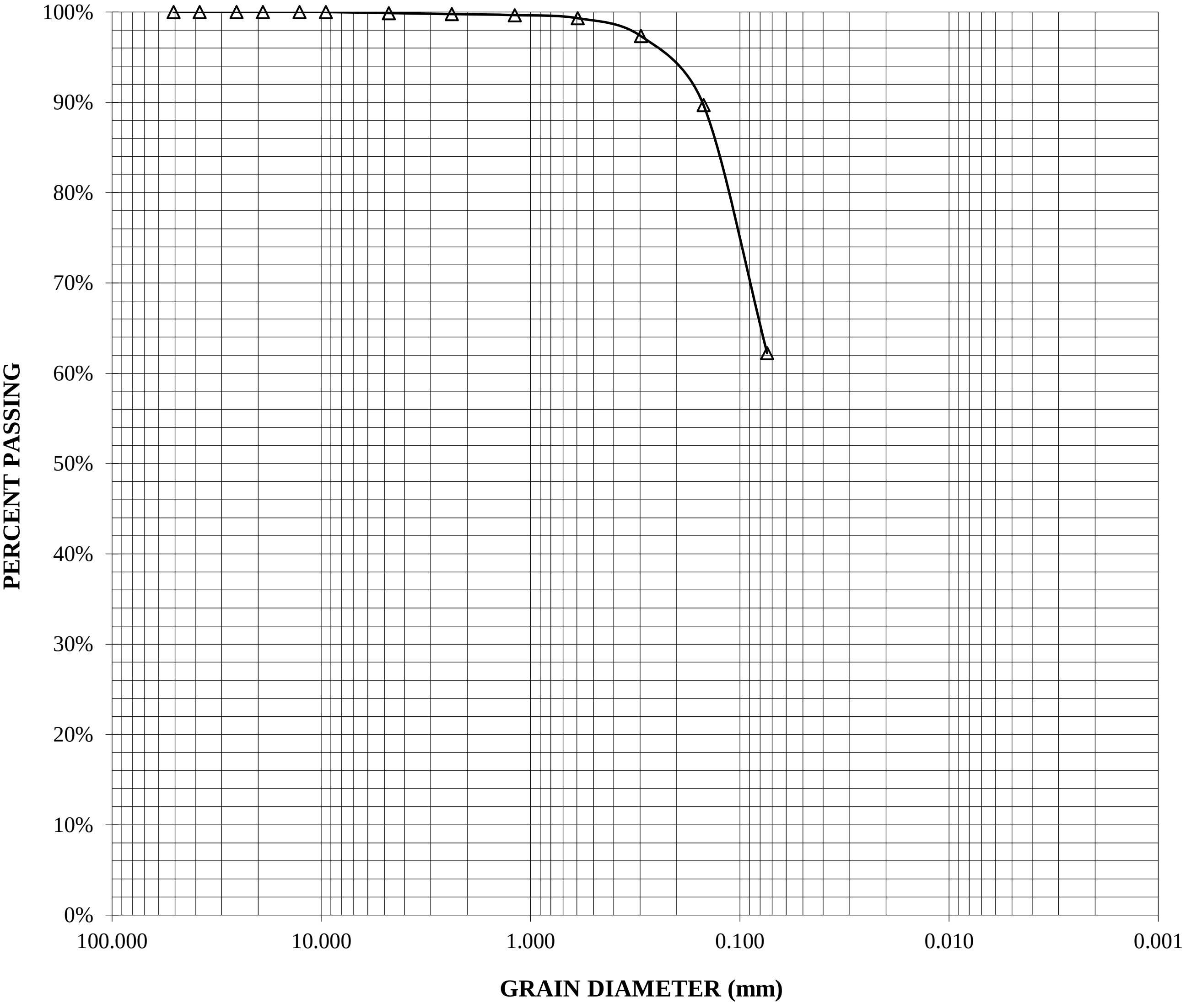
BUTANO GEOTECHNICAL ENGINEERING, INC.	**GRAIN SIZE DISTRIBUTION** 2370 Shore Road	FIGURE C-3

BORING:	B5-4	PERCENT PASSING No. 4	PERCENT PASSING No. 200
DEPTH (ft):	8		
SOIL TYPE (USCS):	SM	100.0%	46.4%



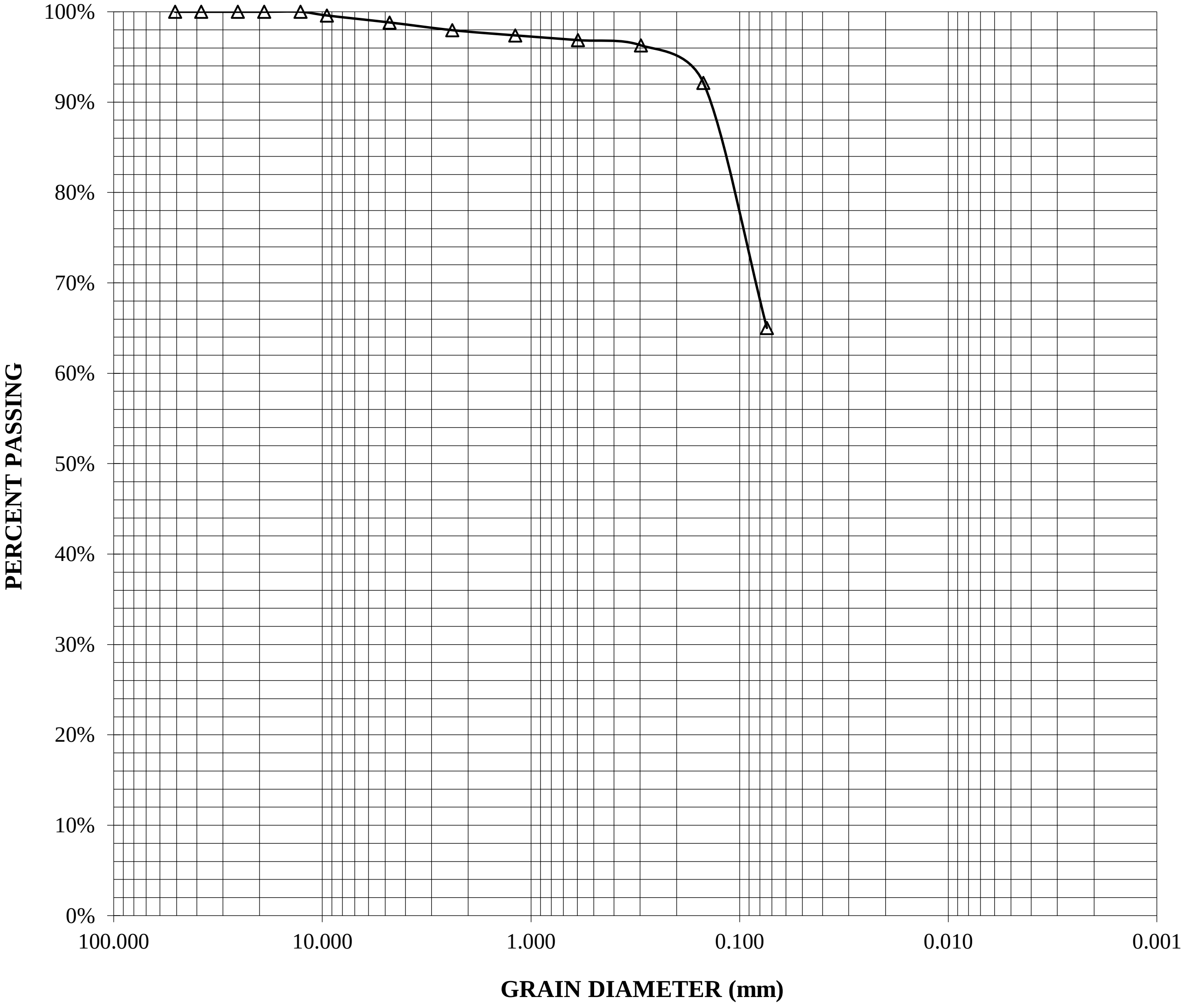
BUTANO GEOTECHNICAL ENGINEERING, INC.	**GRAIN SIZE DISTRIBUTION** 2370 Shore Road	FIGURE C-4

BORING:	B6-4	PERCENT	PERCENT
DEPTH (ft):	8	PASSING No. 4	PASSING No. 200
SOIL TYPE (USCS):	ML	99.9%	62.2%



BUTANO GEOTECHNICAL ENGINEERING, INC.	**GRAIN SIZE DISTRIBUTION** 2370 Shore Road	FIGURE C-5

BORING:	B8-3	PERCENT	PERCENT
DEPTH (ft):	4	PASSING No. 4	PASSING No. 200
SOIL TYPE (USCS):	ML	98.8%	65.0%



BUTANO	**GRAIN SIZE DISTRIBUTION**	FIGURE
GEOTECHNICAL ENGINEERING, INC.	2370 Shore Road	C-6

APPENDIX D

SEISMIC ANALYSIS PROGRAM

SEISMIC ANALYSIS PROCEDURES

The site was analyzed for liquefaction potential utilizing the data in the most recent publications of the NCEER Workshop and SP 117 implementations. A design input summary of calculations are shown in the attached spreadsheets.

Factor of safety for liquefaction is defined as the available cyclic soil strength to resist stress (CSR) divided by cyclic stresses generated by the design earthquake (CSR_{eq}). The Cyclic Stress Ratio (CSR) for the soil is calculated by analyzing the soil strength based on fines corrected blow counts ($(N_1)_{60cs}$). The Cyclic Stress Ratio (CSR_{eq}) is calculated based on the overburden stress (σ_V), estimated shear wave velocity of the soil (V_s), the peak ground acceleration (PGA), and the design earthquake magnitude.

A liquefaction analysis was conducted on borings B1 and B2.

The soil encountered between 5 and 10 feet at the site is potentially liquefiable if the water level were to rise 4 feet.

The CSR for the soil in the sand layers was analyzed using fines corrected blow counts. The lowest estimated factor of safety against liquefaction using the NCEER method was 0.45.

LIQUEFACTION

RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING: A UNIFIED AND CONSISTENT FRAMEWORK
R.B. Seed, January 2025

24-261-SB
2370 Shore Road
Boring B1 and B2

VALUES IN BLUE ARE INPUT VALUES

PGA	1.051
MAGNITUDE	7.9
GW ELEVATION (ft)	5.0
(Below Existing Grade)	
$Vs_{,40ft}$ (ft/s)	450

Settlement

Layer	CSR	$(N_1)_{60}$	Volumetric Strain (%)	Layer Thickness (ft)	Settlement (in)
2	0.51	8	2.5	5	1.5
Total Settlement					**1.5**

SUBSURFACE SOIL LAYERS

Fines Content

LAYER 1

		GW DEPTH (ft)	λ (pcf)	SAMPLE DEPTH (ft)	N_{60}	σv (AT SAMPLE (psf)	$\sigma' v$ AT SAMPLE (psf)	σv AT BOL (psf)	$\sigma' v$ AT BOL (psf)	C_N	N_{160}	FC (%)	C_{FINES}	N_{160CS}	PGA	Mag	Vs (m/s)	Vs (ft/s)	r_d	CSR_{peak}	CSR_{eq}	DWF_M	CSRN	Kσ	CSR_{eq}*	CSR* 50% Prob	FS 50% Prob
START (ft)	0.0																										
FINISH (ft)	5.0	5.0	120.0	2.0	10	240.0	240.0	600.0	600.0	2.9	29	80.0	1.5	42	1.051	7.9	113.4	371.9	0.95	1.00	0.65	0.9	0.71	1.5	0.47	0.92	1.96

LAYER 2

		GW DEPTH (ft)	λ (pcf)	SAMPLE DEPTH (ft)	N_{60}	σv (AT SAMPLE (psf)	$\sigma' v$ AT SAMPLE (psf)	σv AT BOL (psf)	$\sigma' v$ AT BOL (psf)	C_N	N_{160}	FC	C_{FINES}	N_{160CS}	PGA	Mag	Vs (m/s)	Vs (ft/s)	r_d	CSR_{peak}	CSR_{eq}	DWF_M	CSRN	Kσ	CSR_{eq}*	CSR* 50% Prob	FS 50% Prob
START (ft)	5.0																										
FINISH	10.0	5.0	125.0	5.0	8	600.0	600.0	1225.0	913.0	1.8	15	70.0	1.5	22	1.051	7.9	123.8	406.3	0.92	0.97	0.63	0.9	0.69	1.4	0.51	0.23	0.45
				gma																						(Liquefiable)	

Important Information about Your

Geotechnical Engineering Report

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

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

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

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

Read the Full Report

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

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

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

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE
THE BEST PEOPLE ON EARTH

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