GEOTECHNICAL INVESTIGATION REPORT
PROPOSED COMMERCIAL LODGING
APNS# 0308-147-05 THROUGH 0308-147-07 & 0308-161-06
LYNN ROAD

Big Bear Lake, California

Prepared for:
LYNN ROAD LLC

Prepared by:
GEOBODEN INC.
Irvine, CA 92620

February 16, 2023

Project No. Lynn Rd-1-01

GEOTECHNICAL INVESTIGATION REPORT
PROPOSED COMMERCIAL LODGING
APNS# 0308-147-05 THROUGH 0308-147-07 & 0308-161-06
LYNN ROAD
BIG BEAR LAKE, CALIFORNIA

LYNN ROAD LLC

Prepared by:

GEOBODEN INC.
5 Hodgenville
Irvine, California 92620

February 16, 2023

J.N. Lynn Road-1-01

GEOBODEN, INC.
Geotechnical Consultants

February 16, 2023 Project No. Lynn Road-1-01

Attention: Lynn Road LLC

Subject: Geotechnical Investigation Report
Proposed Commercial Lodging
APNS# 0308-147-05 Through 0308-147-07 & 0308-161-06
Lynn Road
Big Bear Lake, California

GeoBoden, Inc. is pleased to provide you with this report on our geotechnical investigation for the proposed Commercial Lodging to be constructed on the subject site.

Based upon the findings of our investigations, we have concluded that the proposed Commercial Lodging is feasible from the geotechnical perspective.

It has been a pleasure to be of service to you on this project. Should you have any questions regarding the contents of this report, or should you require additional information, please do not hesitate to contact us.

Respectfully submitted,
GEOBODEN INC.

REGISTERED PROFESSIONAL ENGINEER
SHAHROKH E. RADVAR
2742
GEOTECHNICAL
STATE OF CALIFORNIA

Shahrokh (Cyrus) E Radvar, G.E.#2742
Principal Engineer

Copies: 4/Addressee

GEOTECHNICAL INVESTIGATION REPORT

PROPOSED COMMERCIAL LODGING
APNS# 0308-147-05 THROUGH 0308-147-07 & 0308-161-06
Lynn Road
BIG BEAR LAKE, CALIFORNIA

TABLE OF CONTENTS

FIGURES

APPENDIXES

GEOTECHNICAL INVESTIGATION REPORT
PROPOSED COMMERCIAL LODGING
APNS# 0308-147-05 THROUGH 0308-147-07 & 0308-161-06
LYNN ROAD

Big Bear Lake, California

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation performed by GeoBoden, Inc. (GeoBoden) for the proposed Commercial Lodging to be constructed at Lynn Road in Big Bear Lake, California. The general location of the project is shown on Figure 1.

The purposes of this investigation were to determine the geotechnical properties of subsurface soil conditions, to evaluate their in-place characteristics, evaluate site seismicity, and to provide geotechnical recommendations with respect to design and construction of the proposed Commercial Lodging.

The scope of the authorized investigation included performing a site reconnaissance, conducting field exploration and laboratory testing programs, performing engineering analyses, and preparing this Geotechnical Investigation Report. Evaluation of environmental issues or the potential presence of hazardous materials was not within the scope of services provided.

2.0 SITE LOCATION AND DESCRIPTION

The subject lot is located at Lynn Road in Big Bear Lake, California. The site is located on a flat land. A new development for a hotel type, transient use. The rental units are designed as cabin style, single dwelling units intended as short-term rentals. An onsite office with small storage area is included. The proposed Commercial Lodging will be of wood-frame structures and will be located on shallow foundation system with slabs-on-grades.

3.0 GEOTECHNICAL INVESTIGATION

Our geotechnical investigation included a field exploration program and a laboratory testing programs. These programs were performed in accordance with our scope of services. The field exploration and laboratory testing programs are described below.

3.1 FIELD EXPLORATION PROGRAM

The field exploration program was initiated under the supervision of an engineer. Six (6) exploratory borings were drilled using a truck-mounted drilling rig equipped with 8-inch diameter hollow stem augers. The borings were advanced to depths ranging from 6.5 to of 21.5 feet (below ground surfaces). The approximate locations of exploratory borings are shown on Figure 2.

Logs of subsurface conditions encountered in the borings were prepared in the field by a representative of our firm. Soil samples consisting of relatively undisturbed brass ring samples and Standard Penetration Tests (SPT) samples were collected at approximately 5-foot depth intervals and were returned to the laboratory for testing. The SPTs were performed in accordance with ASTM D 1586. Final borings logs were prepared from the field logs and are presented in Appendix A.

3.2 LABORATORY TESTING

Selected samples collected during drilling activities were tested in the laboratory to assist in evaluating controlling engineering properties of subsurface materials at the site. Physical tests performed included moisture determination, Atterberg, Passing No. 200, direct shear testing, and corrosion. The results of the laboratory testing are presented in Appendix B.

4.0 DISCUSSION OF FINDINGS

4.1 GEOLOGY AND SUBSURFACE SOIL CONDITIONS

The site is underlain by alluvial soils to the maximum explored depth 21.5 feet below ground surface. Near surface sandy clayey soils were found stiff. Deeper clayey soils were found stiff to hard.

4.2 GEOLOGIC HAZARDS

The most significant geologic hazard to the project is the potential for moderate ground shaking resulting from earthquakes generated on the faults within the vicinity of the site. The site is not mapped in an Alquist-Priolo Special Studies zone for earthquake rupture hazard. No seismic settlement, liquefaction or flooding is anticipated to affect the site, provided geotechnical recommendations provided in this report are followed. The site is relatively flat, the risk of landsliding is nil as long as proper drainage is maintained and footings are designed in accordance with the recommendations contained in this report.

4.3 GROUNDWATER CONDITIONS

Groundwater was not encountered within our exploratory borings. Fluctuations of the groundwater level, localized zones of perched water, and soil moisture content should be anticipated during and following the rainy season. Irrigation of landscaped areas on or adjacent to the site can also cause a fluctuation of soil moisture content and local groundwater levels and should be minimized.

4.4 FAULTING AND SEISMICITY

We consider the most significant geologic hazard to the project to be the potential for moderate to strong seismic shaking that is likely to occur during the design life of the proposed project. The project site is located in the highly seismic Southern California region within the influence of several faults that are considered to be active or potentially active. An active fault is defined by the State of California as a "sufficiently active and well defined fault" that has exhibited surface displacement within the Holocene time (about the last 11,000 years). A potentially active fault is defined by the State as a fault with a history of movement within Pleistocene time (between 11,000 and 1.6 million years ago).

These active and potentially active faults are capable of producing potentially damaging seismic shaking at the site. It is anticipated that the project site will periodically experience ground acceleration as the result of small to moderate magnitude earthquakes. Other active faults without surface expression (blind faults) or other potentially active seismic sources are not currently zoned and may be capable of generating an earthquake are known to be locally present under the region.

Based on our review of published and unpublished geotechnical maps and literature pertaining to the site, the North Frontal (West) fault is located 13.18 kilometers from the site with anticipated maximum moment magnitudes (Mw) of 7.2.

4.5 SEISMIC DESIGN PARAMETERS

The project site is located in a seismically active area typical of Southern California and likely to be subjected to a strong ground shaking due to earthquakes on nearby faults.

To accommodate effects of ground shaking produced by regional seismic events, seismic design can, at the discretion of the designing Structural Engineer, be performed in accordance with the 2022 edition of the California Building Code (CBC). Table below, 2022 CBC Seismic Parameters, lists (next) seismic design parameters based on the 2022 CBC methodology:

2022 CBC Seismic Design Parameters	Value
Site Latitude (decimal degrees)	34.2430
Site Longitude (decimal degrees)	-116.9147
Site Class Definition	D
Mapped Spectral Response Acceleration at 0.2s Period, S_s	1.89
Mapped Spectral Response Acceleration at 1s Period, S_1	0.58
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	2.08
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	1.43
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	1.39
Design Spectral Response Acceleration at 1s Period, S_{D1}	0.95

4.6 LIQUEFACTION POTENTIAL

For liquefaction to occur, all of three key ingredients are required: liquefaction-susceptible soils, groundwater within a depth of 50 feet or less, and strong earthquake shaking. Soils susceptible to liquefaction are generally saturated loose to medium dense sands and non-plastic silt deposits below the water table.

The site is underlain by clayey soils. Groundwater was not encountered within our exploratory test borings. It is our opinion that potential for liquefaction at the site is very low.

5.0 DESIGN RECOMMENDATIONS

Based upon the results of our investigation, the proposed Commercial Lodging is considered geotechnically feasible provided the recommendations presented herein are incorporated into the design and construction. If changes in the design of the structure are made or variations or changed conditions are encountered during construction, GeoBoden should be contacted to evaluate their effects on these recommendations. The following geotechnical engineering recommendations for the proposed building addition are based on observations from the field investigation program and the physical test results.

5.1 EARTHWORK

All earthworks, including excavation, backfill and preparation of subgrade, should be performed in accordance with the geotechnical recommendations presented in this report and applicable portions of the grading code of local regulatory agencies. All earthwork should be performed under the observation and testing of a qualified geotechnical engineer.

5.2 SITE AND FOUNDATION PREPARATION

The construction area should be cleared of any vegetation and stripped of miscellaneous debris and other deleterious material. Organic matter and all other material that may interfere with the completion of the work should be removed from the limits of the construction area. Vegetation, construction debris, and organic matter should not be incorporated into engineered fill. We recommend all existing unsuitable surficial soils be removed and replaced with properly compacted fill within the area of proposed construction. We recommend that the depths of removal be in the approximate range of 3 feet below existing grades; however, deeper removals may be required in local areas depending on actual subsurface conditions encountered during construction.

5.3 FILL PLACEMENT AND COMPACTION REQUIREMENTS

Material for engineered fill should be select free of organic material, debris, and other deleterious substances, and should not contain fragments greater than 3 inches in maximum dimension. On-site excavated soils that meet these requirements may be used to backfill the excavated building pad area.

All fill should be placed in 6-inch-thick maximum lifts, watered or air dried as necessary to achieve a few percent above optimum moisture content, and then compacted in place to a maximum relative compaction of 90 percent. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with Test Method ASTM D 1557. A representative of the project consultant should be present on-site during grading operations to verify proper placement and compaction of all fill, as well as to verify compliance with the other geotechnical recommendations presented herein.

Imported soils, if any, should consist of clean materials exhibiting a VERY LOW expansion potential (Expansion Index less than 20). Soils to be imported should be approved by the project geotechnical consultant prior to importation.

5.4 GEOTECHNICAL OBSERVATIONS

Exposed bottom surfaces in each removal area should be observed and approved by the project geotechnical consultant prior to placing fill. No fill should be placed without prior approval from the geotechnical consultant.

The project geotechnical consultant should be present on site during grading operations to verify proper placement and compaction of fill, as well as to verify compliance with the recommendations presented herein.

5.5 POST-GRADING CONSIDERATIONS

Positive drainage devices such as concrete flatwork, graded swales, and area drains should be provided around the new construction to collect and direct all water to a suitable discharge area. Neither rain nor excess irrigation water should be allowed to collect or pond against building foundations.

5.6 UTILITY TRENCH BACKFIL

All utility trench backfill should be compacted to a minimum relative compaction of 90 percent. Trench backfill materials should be placed in lifts no greater than approximately 6 inches in thickness, watered or air-dried as necessary to achieve a few percent above optimum moisture conditions, and then mechanically compacted in place to a minimum relative compaction of 90 percent. A representative of the project geotechnical consultant should probe and test the backfills to verify adequate compaction.

As an alternative for shallow trenches where pipe or utility lines may be damaged by mechanical compaction equipment, imported clean sand exhibiting a sand equivalent (SE) value of 30 or greater may be utilized. The sand backfill materials should be watered to achieve a few percent above optimum moisture conditions and then tamped into place. No specific relative compaction will be required; however, observation, probing, and if deemed necessary, testing should be performed by a representative of the project geotechnical consultant to verify an adequate degree of compaction and that the backfill will not be subject to settlement.

Where utility trenches enter the footprint of the building, they should be backfilled through their entire depths with on-site fill materials, sand-cement slurry, or concrete rather than with any sand or gravel shading. This "Plug" of less- or non-permeable materials will mitigate the potential for water to migrate through the backfilled trenches from outside of the building to the areas beneath the foundations.

5.7 SOLUBLE SULFATES AND SOIL CORROSIVITY

Soluble sulfate, pH, and chloride concentration tests were performed on near-surface collected samples. Corrosion test results are presented in Appendix B. The minimum resistivity tests on near collected bulk samples indicate that the onsite surficial soils are corrosive when in contact with ferrous materials. Typical recommendations for mitigation of the corrosive potential of the soil in contact with building materials are the following:

- Below grade ferrous metals should be given a high quality protective coating, such as an 18 mil plastic tape, extruded polyethylene, coal tar enamel, or Portland cement mortar.

- Below grade ferrous metals should be electrically insulated (isolated) from above grade ferrous metals and other dissimilar metals, by means of dielectric fittings in utilities and exposed metal structures breaking grade.

- Steel and wire reinforcement within concrete in contact with the site soils should have at least two inches of concrete cover.

If ferrous building materials are expected to be placed in contact with site soils, it may be desirable to consult a corrosion specialist regarding chosen construction materials, and/or protection design for the proposed structures.

Corrosion test results also indicate that the surficial soils at the site have negligible sulfate attack potential on concrete, according to ACI. As a result, a mix design such as Type II cement should provide resistance against possible sulfate attack.

5.8 SHALLOW FOUNDATIONS

Following the site and foundation preparation recommended above, foundation for load bearing walls and interior columns may be designed as discussed below.

5.8.1 Bearing Capacity and Settlement

Load bearing walls and interior columns may be supported on continuous spread footings and isolated spread footings, respectively, and should bear entirely either upon properly engineered fill. In general, continuous and isolated spread footings supported on fill soils may be designed using an allowable (net) bearing capacity of 2,000 pounds per square foot (psf).

Continuous and pad Footings should have minimum width of 14 inches and 24 inches, respectively. Footings should be embedded a minimum depth of 24 inches measured from the lowest adjacent grade. The maximum allowable bearing values apply to combined dead and sustained live loads. The allowable bearing pressures may be increased by one-third when considering transient live loads, including seismic and wind forces.

Based on the allowable bearing value recommended above, total settlement of the shallow footings are anticipated to be less than one inch, provided foundation preparations conform to the recommendations described in this report. Differential settlement is anticipated to be approximately half the total settlement for similarly loaded footings spaced up to approximately 30 feet apart.

5.8.2 Lateral Load Resistance

Lateral load resistance for structures will be developed by a combination of friction between concrete footings against compacted fill and passive earth pressure developed against the sides of foundations and walls below grade. Total resistance to lateral loads may be determined by combining passive and frictional resistance, without reduction.

In general, an allowable passive earth pressure of 250 psf per foot of foundation depth below final grade may be used for the sides of foundations placed against properly compacted fill. This passive earth pressure against fill may be assumed to increase linearly to a maximum of 2,000 psf. The uppermost one foot of soil below lowest adjacent grade should not be used in determining passive resistance. The above allowable passive pressure incorporates a factor of safety of 2.0.

A coefficient of friction of 0.30 may be used for dead and sustained live load forces to compute the friction resistance of foundations constructed directly on compacted soils. These are allowable values having a factor of safety of 1.5. Under seismic and wind loading conditions, the passive and frictional resistance may be increased by one-third.

5.8.3 Footing Reinforcement

Reinforcement for footings should be designed by the structural engineer based on the anticipated loading conditions. Footings for lightly loaded wood-frame structures that are supported in low expansive soils should have No. 4 bars, two top and two bottom.

5.9 CONCRETE SLAB ON-GRADE

Concrete slabs will be placed on undisturbed natural soils or properly compacted fill. Moisture content of subgrade soils should be maintained a few percent above the optimum moisture content.

At the time of the concrete pour, subgrade soils should be firm and relatively unyielding. Any disturbed soils should be excavated and then replaced and compacted to a minimum of 90 percent relative compaction. Slabs should be designed to accommodate low expansive fill soils. The structural engineer should determine the minimum slab thickness and reinforcing depending upon the expansive soil condition intended use. Slabs placed on low to medium expansive soils should be at least 4 inches thick and have minimum reinforcement of No. 4 bars placed at mid-height of the slabs and spaced 18 inches on centers, in both directions. The structural engineer may require thicker slabs with more reinforcement depending on the anticipated slab loading conditions.

If moisture-sensitive floor covering is planned, a layer of open-graded gravel, at least 4 inches thick, should be placed below the concrete slab to form a capillary break. Alternately, moisture-proof membrane (such as 10-mil) may be utilized. The vapor barrier should be placed between sand layers (2 inches above and below) to protect the membrane from damage during construction. Gravel for use under a concrete floor slab should be clean, crushed rock that meets the gradation requirements presented below.

Sieve Size	Percentage
1 inch	100
¾ inch	90-100
No. 4	0-10

5.10 RETAINING WALLS AND WALLS BELOW GRADE

The project may include walls below grade and retaining walls supporting soil materials. Retaining walls can be founded on shallow foundations in accordance with the recommendations presented in Foundation Section of this report. Design lateral earth pressure, backfill criteria, and drainage recommendations for walls below grade are presented below.

5.10.1 Lateral Earth Pressures

The earth pressure behind retaining walls depends primarily on the allowable wall movement, wall inclination, type of backfill materials, backfill slopes, surcharges, and any hydrostatic pressure. The potential pressure components of subterranean walls include a uniform surcharge pressures for traffic or surcharges, active and restrained horizontal pressure components, and pressures from compaction effort.

Retaining walls should be designed to resist the applicable lateral earth pressures. On-site soil materials may be used as backfill behind retaining walls. These onsite soils belong to low expansive category. Therefore, if these materials are used as backfill, active earth pressures equivalent to fluids having densities of 40 and 63 pounds per cubic foot should be used for design of cantilevered walls retaining a level backfill and ascending 2:1 backfill, respectively. For walls that are restrained at the top, at-rest earth pressures of 60 and 95 pounds per cubic foot (equivalent fluid pressures) should be used. The above values are for retaining walls that have been supplied with a proper subdrain system. All walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings in addition to the above recommended active and at-rest earth pressures.

Where sufficient area exists behind the proposed walls, imported clean sand exhibiting a sand equivalent value (SE) of 30 or greater, or pea gravel or crushed rock may be used for wall backfill to reduce the lateral earth pressures provided these granular backfill materials extend behind the walls to a minimum horizontal distance equal to one-half the wall height. In addition, the sand, pea gravel or rock backfill materials should extend behind the walls to a minimum horizontal distance of 2 feet at the base of the wall or to a horizontal distance equal to the heel width of the footing, whichever is greater. For the above conditions, cantilevered walls retaining a level backfill and ascending 2:1 backfill may be designed to resist active earth pressures equivalent to fluids having densities of 30 and 41 pounds per cubic foot, respectively. For walls that are restrained at the top, at-rest earth pressures equivalent to fluids having densities of 53 and 78 pounds per cubic foot are recommended for design of restrained walls supporting a level backfill and ascending 2:1 backfill, respectively. These values are for retaining walls supplied with a proper subdrain system. Furthermore, as with native soil backfill, the walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings in addition to the above recommended active and at-rest earth pressures.

We have used ½ of 2/3 the PGAM in our analysis (2/6)*0.76g = 0.253. Evaluation of lateral earth pressures under static and seismic loading conditions is based on using the Coulomb (1776) and Mononobe-Okabe (1929) Methods. For a level backfill, we recommend using combined of static and seismic active equivalent earth pressure 66 pounds per cubic feet (pcf) for onsite soils. Onsite soils are anticipated to be used for backfill. The supportive calculations are attached for reference. For walls with a retained height over 6 feet, or where otherwise required by Code or deemed appropriate by the structural engineer, we recommend that the wall designs be checked seismically using an additive seismic Equivalent Fluid Pressure (EFP) of 29 pcf. Such walls that are to be designed in the static case assuming the at-rest condition should be checked seismically using this additive seismic EFP added to the active condition (i.e., the additive seismic EFP is not added to the at-rest EFP). The additive seismic EFP should be applied with a standard EFP pressure distribution (i.e., it is not an inverted triangle).

5.10.2 Drainage and Waterproofing

If walls are designed for drained earth pressures, adequate drainage should be provided behind the walls. This can be accomplished by installing subdrains at the base of the walls. Wall backdrains should consist of a system of filter material and perforated pipe and should be approved by GeoBoden. The perforated pipe system should consist of 4-inch diameter, schedule 40, PVC pipe or equivalent, embedded in 1 cubic foot of Class II Permeable Material (CALTRANS Standard Specifications, latest edition) or equivalent per lineal foot of pipe. Alternatively, ¾-inch open graded gravel or crushed rock enveloped in Mirafi 140 geofabric or equivalent may be used instead of the Class II Permeable Material. The pipe should be placed at the base of the wall, have a gradient of approximately 2 percent, and should be connected to the subdrains and then routed to a suitable area for discharge of accumulated water.

Wall backfill should be protected against infiltration of surface water. Backfill adjacent to walls should be sloped so that surface water drains freely away from the wall and will not pond. Waterproofing of walls below grade is recommended.

6.0 CONSTRUCTION CONSIDERATIONS

Based on our field exploration program, earthwork can be performed with conventional construction equipment.

6.1 TEMPORARY DEWATERING

Groundwater was not encountered within our test hole to the maximum explored depth of 21.5 feet below ground surface. Based on the anticipated excavation depths, temporary dewatering is not considered a design factor for this project.

6.2 CONSTRUCTION SLOPES

An Excavation during construction should be conducted so that slope failure and excessive ground movement will not occur. The short-term stability of excavation depends on many factors, including slope angle, engineering characteristics of the subsoils, height of the excavation and length of time the excavation remains unsupported and exposed to equipment vibrations, rainfall and desiccation.

Where space permits, and providing that adjacent facilities are adequately supported, open excavations may be considered. In general, unsupported slopes for temporary construction excavations should not be expected to stand at an inclination steeper than 1:1 (horizontal:vertical). The temporary excavation side walls may be cut vertically to a height of 3 feet and then laid back at a 1:1 slope ratio above a height of 3 feet.

Surcharge loads should be kept away from the top of temporary excavations a horizontal distance equal to at least one-half the depth of excavation. Surface drainage should be controlled along the top of temporary excavations to preclude wetting of the soils and erosion of the excavation faces. Even with the implementation of the above recommendations, sloughing of the surface of the temporary excavations may still occur, and workmen should be adequately protected from such sloughing.

7.0 EXTERIOR CONCRETE FLATWORK

7.1 THICKNESS AND JOINT SPACING

Concrete sidewalks should be at least 4 inches thick and provided with construction joints or expansion joints every 10 feet or less. Concrete subslabs to be covered with decorative pavers should also be at least 4 inches thick and provided with construction joints or expansion joints every 10 feet or less. Concrete driveway slabs should be at least 6 inches thick and provided with construction joints or expansion joints every 10 feet or less.

7.2 REINFORCEMENT

Consideration should be given to reinforcing all concrete patio-type slabs, driveways and sidewalks greater than 5 feet in width with No. 4 bars spaced 18 inches on centers, both ways. The reinforcement should be positioned near the middle of the slabs by means of concrete chairs or brick.

7.3 SUBGRADE PREPARATION

The subgrade soils below concrete flatwork areas should first be compacted to a minimum relative compaction of 90 percent and then thoroughly moistened to achieve a moisture content that is a few percent above the optimum moisture content. Pre-wetting of the soils will promote uniform curing of the concrete and minimize the development of shrinkage cracks. A representative of the project geotechnical consultant should observe and verify the density and moisture content of the soils, and the depth of moisture penetration prior to pouring concrete.

8.0 POST INVESTIGATION SERVICES

Final project plans and specifications should be reviewed prior to construction to confirm that the full intent of the recommendations presented herein have been applied to design and construction. Following review of plans and specifications, observation should be performed by the geotechnical engineer during construction to document that foundation elements are founded on/or penetrate onto the recommended soils, and that suitable backfill soils are placed upon competent materials and properly compacted at the recommended moisture content.

9.0 CLOSURE

The conclusions, recommendations, and opinions presented herein are: (1) based upon our evaluation and interpretation of the limited data obtained from our field and laboratory programs; (2) based upon an interpolation of soil conditions between and beyond the borings and test pits; (3) are subject to confirmation of the actual conditions encountered during construction; and, (4) are based upon the assumption that sufficient observation and testing will be provided during construction.

If parties other than GeoBoden are engaged to provide construction geotechnical services, they must be notified that they will be required to assume complete responsibility for the geotechnical phase of the project by concurring with the findings and recommendations in this report or providing alternate recommendations.

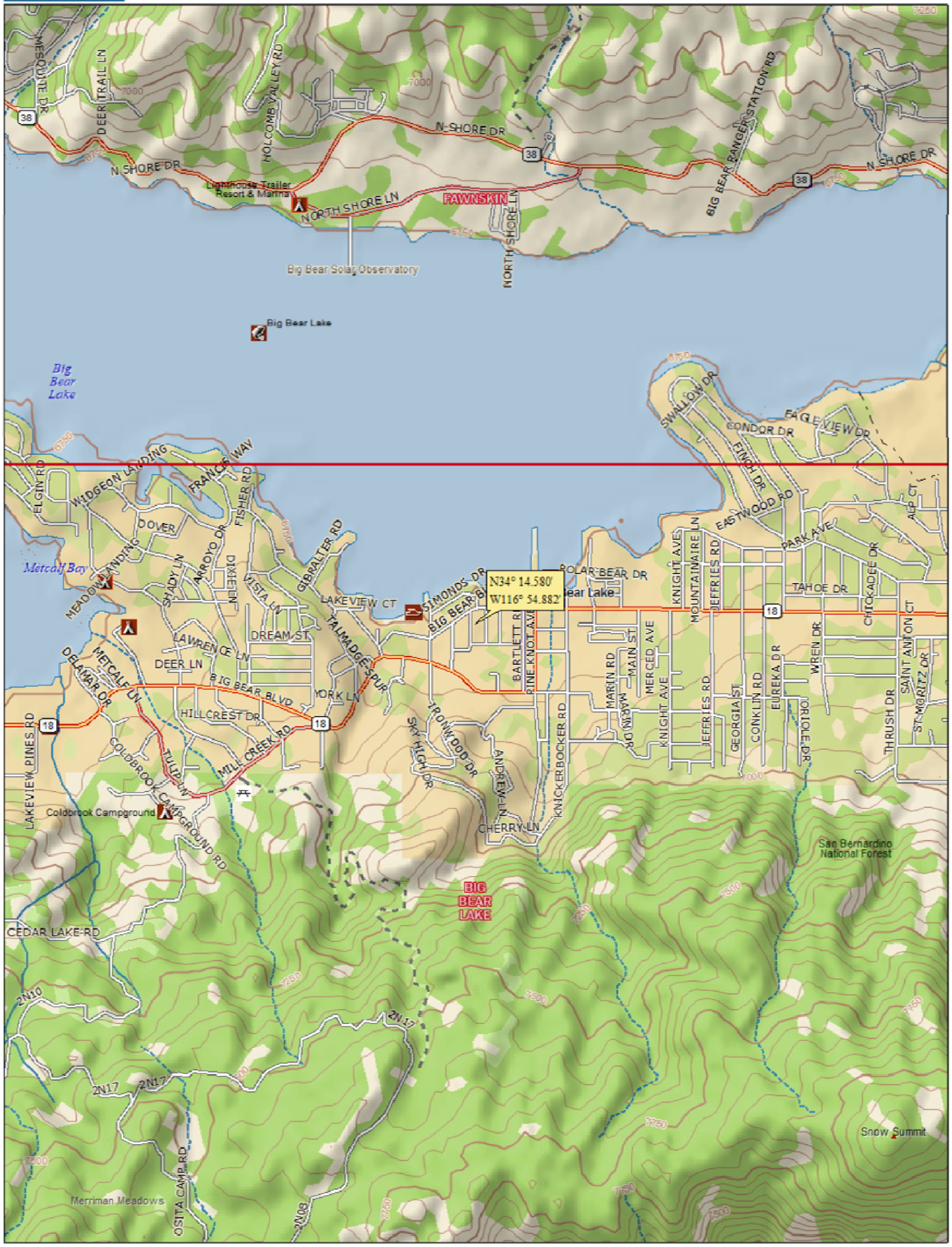
If pertinent changes are made in the project plans or conditions are encountered during construction that appear to be different than indicated by this report, please contact this office. Significant variations may necessitate a re-evaluation of the recommendations presented in this report.

10.0 REFERENCES

2022 California Building Code, Volume 2.

FIGURES

DeLorme Topo USA® 7.0



Data use subject to license.

© DeLorme. DeLorme Topo USA® 7.0.

www.delorme.com

MN (11.2° E)

0 800 1600 2400 3200 4000 ft

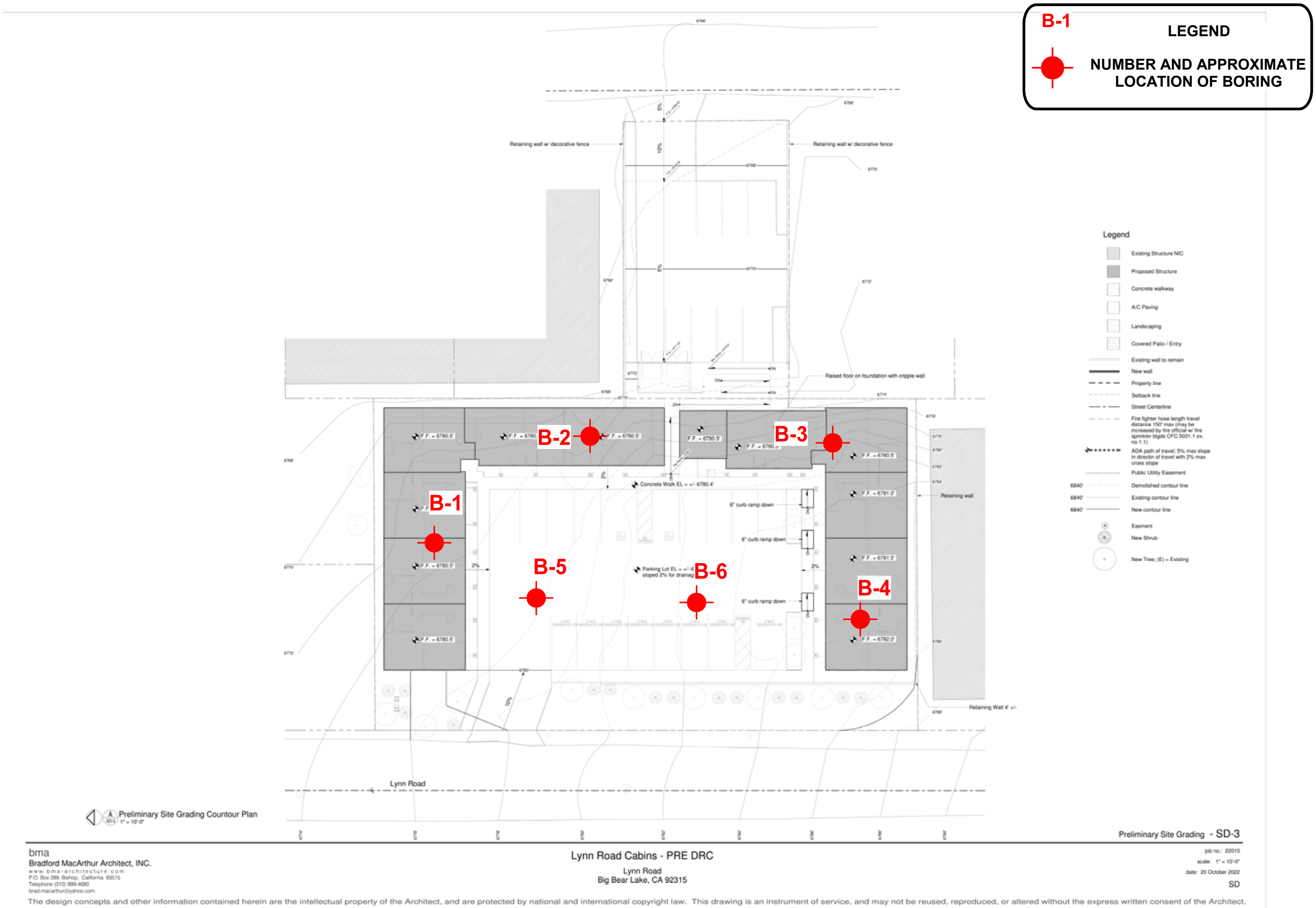
Data Zoom 13-0

GEOBODEN INC.

Geotechnical Consultants

SITE VICINITY MAP
Proposed Commercial Lodging
APNS# 0308-147-05 Through 0308-147-07 & 0308-161-06
Lynn Road
Big Bear Lake, California

Figure By S.R.	Project No. Lynn Rd-1-01
Map No. XX	Figure No. 1
Date 02-16-23	



GEODOBEN INC. Geotechnical Consultants	BORING LOCATION MAP Proposed Commercial Lodging APNS# 0308-147-05 Through 0308-147-07 & 0308-161-06 Lynn Road Big Bear Lake, California	Figure By S.R.	Project No. Lynn Rd
		Map No. XX	Figure No.
		Date 02-16-23	2

APPENDIX A
EXPLORATORY BORINGS LOG

APPENDIX A
SUBSURFACE EXPLORATION PROGRAM

PROPOSED COMMERCIAL LODGING
APNS# 0308-147-05 THROUGH 0308-147-07 & 0308-161-06
Lynn Road
BIG BEAR LAKE, CALIFORNIA

Prior to drilling, the proposed borings were located in the field by measuring from existing site features.

A total of six exploratory borings (B-1 through B-6) were drilled using a Diedrich D-25 drill rig equipped with 6-inch outside diameter (O.D.) hollow-stem augers. GeoBoden of Irvine, California performed the drilling. The borings locations are shown on Figure 2.

Depth-discrete soil samples were collected at selected intervals from the exploratory borings using a 2 ½ -inch inside diameter (I.D.) modified California Split-barrel sampler fitted with 12 brass ring of 2 ½ inches in O.D. and 1-inch in height and one brass liner (2 ½ -inch O.D. by 6 inches long) above the brass rings. The sampler was lowered to the bottom of the borehole and driven 18 inches into the soil with a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler the lower 12 inches is shown on the blow count column of the borings logs.

After removing the sampler from the borehole, the sampler was opened and the brass rings and liner containing the soil were removed and observed for soil classification. Brass rings containing the soil were sealed in plastic canisters to preserve the natural moisture content of the soil. Soil samples collected from exploratory borings were labeled, and submitted to the laboratory for physical testing.

Standard Penetration Tests (SPTs) were also performed at alternative depths in Borings. The SPT consists of driving a standard sampler, as described in the ASTM 1586 Standard Method, using a 140-pound hammer falling 30 inches. The number of blows required to drive the SPT sampler the lower 12 inches of the sampling interval is recorded on the blow count column of the borings log.

The soil classifications and descriptions on field log were performed using the Unified Soil Classification System as described by the American Society for Testing and Materials (ASTM) D 2488-90, "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)." The final borings logs were prepared from the field logs and are presented in this Appendix.

At the completion of the sampling and logging, the exploratory borings were backfilled with the drilled cuttings.

GEOBODEN, INC.

BORING NUMBER B-1

CLIENT Lynn Road LLC

PROJECT NAME Proposed Commercial Lodging

PROJECT NUMBER Lynn Road-1-01

PROJECT LOCATION Lynn Road, Big Bear Lake, California

DATE STARTED 2/9/23 **COMPLETED** 2/9/23

GROUND ELEVATION **HOLE SIZE** 6 inches

DRILLING CONTRACTOR GeoBoden Inc.

GROUND WATER LEVELS:

DRILLING METHOD HSA

AT TIME OF DRILLING ---

LOGGED BY S.R. **CHECKED BY**

AT END OF DRILLING ---

NOTES

AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SANDY CLAY (CL): brown, moist, fine sand										
5			MC R-1		36		108	17	48	23	25	67
10			MC R-2		39		110	16				
		olive brown										
15			SS S-3		22							
		SILTY SAND (SM): light olive brown, moist										
20			SS S-4		32							36

Bottom of borehole at 21.5 feet below ground surface. Groundwater was not encountered. Boring was backfilled with cuttings.

Bottom of borehole at 21.5 feet.

GEOBODEN, INC.

BORING NUMBER B-2

CLIENT Lynn Road LLC

PROJECT NAME Proposed Commercial Lodging

PROJECT NUMBER Lynn Road-1-01

PROJECT LOCATION Lynn Road, Big Bear Lake, California

DATE STARTED 2/9/23 **COMPLETED** 2/9/23

GROUND ELEVATION **HOLE SIZE** 6 inches

DRILLING CONTRACTOR GeoBoden Inc.

GROUND WATER LEVELS:

DRILLING METHOD HSA

AT TIME OF DRILLING ---

LOGGED BY S.R. **CHECKED BY**

AT END OF DRILLING ---

NOTES

AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SANDY CLAY (CL): dark to reddish brown, moist, fine sand										
5			MC R-1		29		103	18	47	26	21	69
10		brown	MC R-2		31		108	16				

Bottom of borehole at 11.5 feet below ground surface. Groundwater was not encountered. Boring was backfilled with cuttings.

Bottom of borehole at 11.5 feet.

GEOBODEN, INC.

BORING NUMBER B-3

CLIENT Lynn Road LLC
PROJECT NAME Proposed Commercial Lodging
PROJECT NUMBER Lynn Road-1-01
PROJECT LOCATION Lynn Road, Big Bear Lake, California
DATE STARTED 2/9/23 **COMPLETED** 2/9/23
GROUND ELEVATION **HOLE SIZE** 6 inches
DRILLING CONTRACTOR GeoBoden Inc.
GROUND WATER LEVELS:
DRILLING METHOD HSA
AT TIME OF DRILLING ---
LOGGED BY S.R. **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SANDY CLAY (CL): reddish brown, moist, fine sand										
5			MC R-1		31							
10			MC R-2		35							

Bottom of borehole at 11.5 feet below ground surface. Groundwater was not encountered. Boring was backfilled with cuttings.

Bottom of borehole at 11.5 feet.

GEOBODEN, INC.

BORING NUMBER B-4

CLIENT Lynn Road LLC
PROJECT NAME Proposed Commercial Lodging
PROJECT NUMBER Lynn Road-1-01
PROJECT LOCATION Lynn Road, Big Bear Lake, California
DATE STARTED 2/9/23 **COMPLETED** 2/9/23
GROUND ELEVATION **HOLE SIZE** 6 inches
DRILLING CONTRACTOR GeoBoden Inc.
GROUND WATER LEVELS:
DRILLING METHOD HSA
AT TIME OF DRILLING ---
LOGGED BY S.R. **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SANDY CLAY (CL): brown, moist, fine sand										
5			MC R-1		27		104	21				
10			MC R-2		33		110	17				
		Bottom of borehole at 11.5 feet below ground surface. Groundwater was not encountered. Boring was backfilled with cuttings.										
		Bottom of borehole at 11.5 feet.										

GEOBODEN, INC.

BORING NUMBER B-5

CLIENT Lynn Road LLC
PROJECT NAME Proposed Commercial Lodging
PROJECT NUMBER Lynn Road-1-01
PROJECT LOCATION Lynn Road, Big Bear Lake, California
DATE STARTED 2/9/23 **COMPLETED** 2/9/23
GROUND ELEVATION **HOLE SIZE** 6 inches
DRILLING CONTRACTOR GeoBoden Inc.
GROUND WATER LEVELS:
DRILLING METHOD HSA
AT TIME OF DRILLING ---
LOGGED BY S.R. **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SANDY CLAY (CL): brown, moist, fine sand										
5			MC R-1		29							

Bottom of borehole at 6.5 feet below ground surface. Groundwater was not encountered. Boring was backfilled with cuttings.
Bottom of borehole at 6.5 feet.

GEOBODEN, INC.

CLIENT Lynn Road LLC
PROJECT NAME Proposed Commercial Lodging
PROJECT NUMBER Lynn Road-1-01
PROJECT LOCATION Lynn Road, Big Bear Lake, California
DATE STARTED 2/9/23 **COMPLETED** 2/9/23
GROUND ELEVATION **HOLE SIZE** 6 inches
DRILLING CONTRACTOR GeoBoden Inc.
GROUND WATER LEVELS:
DRILLING METHOD HSA
AT TIME OF DRILLING ---
LOGGED BY S.R. **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SANDY CLAY (CL): reddish brown, moist, fine sand										
5			MC R-1		36							

Bottom of borehole at 6.5 feet below ground surface. Groundwater was not encountered. Boring was backfilled with cuttings.

Bottom of borehole at 6.5 feet.

APPENDIX B
LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

PROPOSED COMMERCIAL LODGING
APNS# 0308-147-05 THROUGH 0308-147-07 & 0308-161-06
Lynn Road
BIG BEAR LAKE, CALIFORNIA

Laboratory tests were performed on selected samples to assess the engineering properties and physical characteristics of soils at the site. The following tests were performed:

- Moisture content and dry density
- No. 200 Wash sieve
- Atterberg limits
- direct shear
- corrosion potential

Test results are summarized on laboratory data sheets or presented in tabular form in this appendix.

Moisture Density Tests

The field moisture contents, as a percentage of the dry weight of the soils, were determined by weighing samples before and after oven drying. The dry density, in pounds per cubic foot, was also determined fir all relatively undisturbed ring samples collected. These analyses were performed in accordance with ASTM D 2937. The results of these determinations are shown on the borings logs in Appendix A.

No. 200 Wash Sieve

Quantitative determination of the percentage of soil finer than 0.075 mm was performed on selected soil sample by washing the soil through the No. 200 sieve. Test procedures were performed in accordance with ASTM Method D1140. The results of the tests are shown on the borings logs.

Atterberg Limits

Liquid limit, plastic limit, and plasticity index were determined for selected soil sample in accordance with ASTM D 4318. The soil sample was air-dried and passed through a No. 40 sieve and moisturized. The liquid and plastic limit tests were performed on the fraction passing the No. 40 sieve. Results of the Atterberg limits tests are shown graphically and presented in this Appendix.

Direct Shear

Direct shear tests were performed on undisturbed samples of on-site soils. A different normal stress was applied vertically to each soil sample ring which was then sheared in a horizontal direction. The resulting shear strength for the corresponding normal stress was measured at a maximum constant rate of strain of 0.005 inches per minute. The direct shear results are shown graphically on a laboratory data sheet included in this appendix.

Corrosion Potential

Selected soil sample in the near surface was tested to determine the corrosivity of the site soil to steel and concrete. The soil samples were tested for soluble sulfate (Caltrans 417), soluble chloride (Caltrans 422), and pH and minimum resistivity (Caltrans 643). The results of the corrosion tests are summarized in Table B-1.

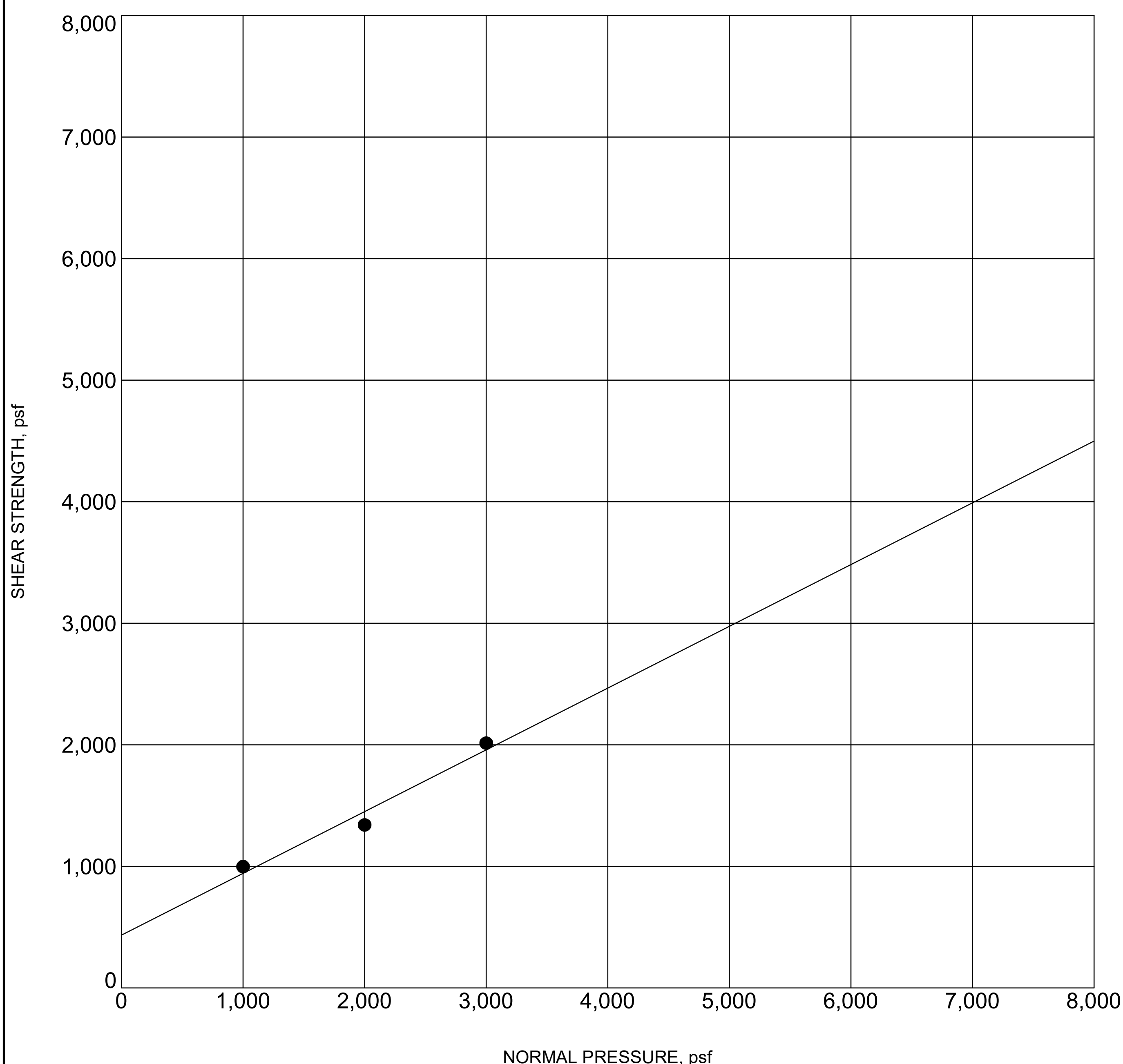
TABLE B-1 (Corrosion Test Results)

Boring No.	Depth (ft)	Chloride Content (Calif. 422) ppm	Sulfate Content (Calif. 417) % by Weight	pH (Calif. 643)	Resistivity (Calif. 643) Ohm*cm
B-1	0-5	143	0.0179	7.5	1,115

GEOBODEN, INC.

DIRECT SHEAR TEST

CLIENT Lynn Road LLC
PROJECT NAME Proposed Commercial Lodging
PROJECT NUMBER Lynn Road-1-01
PROJECT LOCATION Lynn Road, Big Bear Lake, California



	Specimen Identification		Classification	γ_d	MC%	c	ϕ
●	**B-1**	**5.0**	**SANDY LEAN CLAY(CL)**	**108**	**17**	**435.0**	**27**

GEOBODEN, INC.

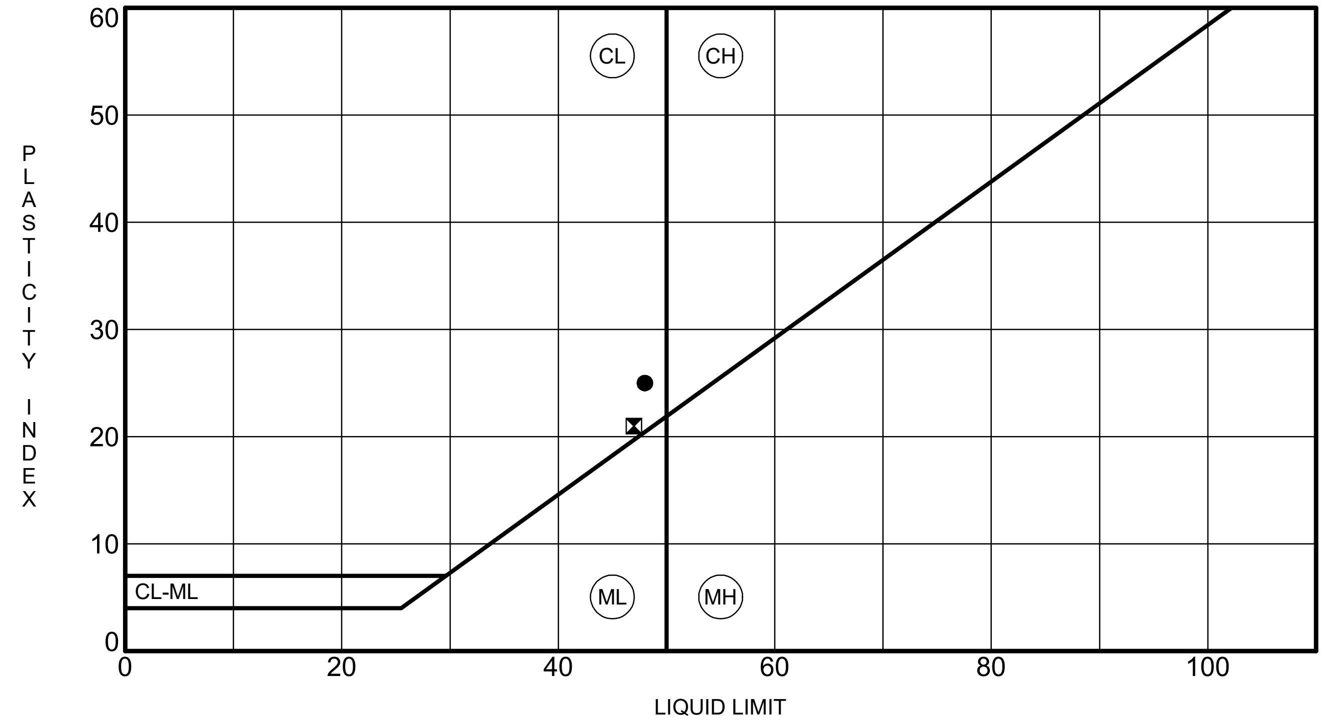
ATTERBERG LIMITS' RESULTS

CLIENT Lynn Road LLC

PROJECT NAME Proposed Commercial Lodging

PROJECT NUMBER Lynn Road-1-01

PROJECT LOCATION Lynn Road, Big Bear Lake, California



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-1**	**5.0**	**48**	**23**	**25**	**67**	**SANDY LEAN CLAY(CL)**
⧗	**B-2**	**5.0**	**47**	**26**	**21**	**69**	**SANDY LEAN CLAY(CL)**

Earth Pressure Calculation

Client: George	Job No.:
By: CR	Date: 2/23/2023

SOIL PROPERTIES

Unit Weight, γ (pcf):	125
Cohesion, c (psf):	0
Friction Angle, ϕ (deg):	27

Seismic Earth Pressure - Mononobe-Okabe Method

Wall Friction Angle, δ (deg):	27
Recommended δ(deg) = 2/3 ϕ:	18.00
Backslope Angle w/ htal., β (deg):	0
Back of Wall Angle w/ vcal., θ (deg):	0
Htal. Ground Acc., k_h (g's):	0.253
Vcal. Ground Acc., k_v (g's):	0
ψ (rad):	0.2478
Height of Wall, H (ft):	10

Pseudo, ϕ (deg): 27.00 (obtaining ϕ from γ.Hwall & 0)

Active Earth Pressure		Pasive Earth Pressure	
K_{AE}:	0.589	KP_E:	1.304
P_{AE} (lb):	3278	P_{PE} (lb):	7259
E.F.P. (pcf):	**65.56**	E.F.P. (pcf):	145.19

NOTE: Values represent combined effect of static & dynamic stresses

SOIL FRICTION FACTOR: 0.51 SQRT(Nphi) . 1.63 Nphi . . . 2.66 (k_p)

PASSIVE PRESSURE

Pp =(2*COHESION*sqrt(Nphi)) + (UNIT WEIGHT*DEPTH*Nphi)

Pp = 0 psf + 332.9 pcf * DEPTH (feet)

Pp = 0 psf at a depth of 0.0 feet

Pp = 1664 psf at a depth of 5.0 feet

Pp = 3329 psf at a depth of 10.0 feet

ACTIVE PRESSURE

Pa = -(2*COHESION/sqrt(Nphi)) + (UNIT WEIGHT*DEPTH/Nphi)

Pa = 0 psf + 46.9 pcf * DEPTH (feet)

Pa = 0 psf at a depth of 0.0 feet

Pa = 469 psf at a depth of 10.0 feet

Pa = 939 psf at a depth of 20.0 feet

Pa = 1408 psf at a depth of 30.0 feet