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BAGG Job No. LAMBR-01-00

Nicholson-Lamb Venture, LP
1162 El Camino Real
Menlo Park, California 94025

Report
Geotechnical Engineering Investigation
Proposed Residential Condominiums Project
Vacant Parcel at the NE Corner of
Woodside Road and Horgan Avenue
885 Woodside Road
Redwood City, California

Dear Mr. Lamb:

We are pleased to present this geotechnical engineering report for the design phase of the proposed Residential Condominiums Project in Redwood City, California. This report presents a description of the subsurface conditions encountered at the site, and summarizes our recommendations for the design and construction of the project. The most significant geotechnical aspect of the site is the loose to medium dense subsurface soils with a potential for differential settlements due to seismic events. However, based on the information obtained from our investigation, it is our opinion the proposed building is feasible from a geotechnical point of view, provided the recommendations presented in this report are incorporated into the design and construction of the project.

Thank you for retaining BAGG to provide these services. Please do not hesitate to contact us if you have any questions or comments regarding this report.

Very truly yours,
BAY AREA GEOTECHNICAL GROUP

Bruce Gaviglio
Geotechnical Engineer

Distribution: 6 copies addressee

REPORT
GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED RESIDENTIAL CONDOMINIUMS PROJECT
VACANT PARCEL AT THE NE CORNER OF
WOODSIDE ROAD AND HORGAN AVENUE
885 WOODSIDE ROAD
REDWOOD CITY, CALIFORNIA

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1.0 INTRODUCTION

This report presents the results of our geotechnical engineering investigation performed for the subject residential condominiums project located in Redwood City, California. The attached Plate 1, Vicinity Map, shows the general location of the site, and Plate 2, Site Plan, shows the general layout of the site, the proposed building footprint, the prominent features in the immediate vicinity, and the approximate locations of the exploratory borings drilled for this investigation. Our services have been provided in accordance with the scope of services outlined in our Proposal No 06-191, dated July 13, 2006.

2.0 SITE AND PROJECT DESCRIPTION

The proposed residential condominiums project will be constructed on an L-shaped vacant parcel located on the northeast corner of Woodside Road and Horgan Avenue, adjacent to an active service station in Redwood City, California. The corner lot is relatively flat and covered with grass. The existing service station to the west and southwest of the proposed condominiums is to remain. A three-story apartment building lines the northern site boundary and homes are situated adjacent to the east boundary of the parcel. During our visit, we observed a stand-pipe at the site which appears to be a fire waterline from an old building that was demolished and removed from the site.

It is our understanding that the proposed project will construct a 5-story structure with a basement L-shaped condominium structure at the subject site with one-level basement parking. The building will encompass a total approximate footprint of 45,000 square feet. Information with respect to the anticipated structural loads for the building, or the nature of the planned site grading other than the expected excavation for the below- grade parking, has not been provided to BAGG as the project is in a preliminary stage.

Because the proposed building will occupy much of the site and its edges will be situated in the close proximity of the property boundaries, sloped excavation for the basement will not be possible. As such, the sides of the basement excavation will likely be vertical. Any vertical excavations near the property boundaries may undermine the foundations and/or otherwise jeopardize the structural integrity of the buildings on the adjacent properties, particularly the apartment building located on the north edge of the site. Therefore, temporary shoring appropriately designed to resist the lateral earth pressures as well as the surcharge loads imposed by the adjacent buildings will be necessary for the support of the basement sidewalls and the neighboring buildings.

3.0 PURPOSE AND SCOPE OF SERVICES

The purpose of our investigation was to obtain information regarding fill (if any), native soil, and foundation conditions at the subject site in order to develop geotechnical recommendations for the design, support, and construction of the proposed 5-story structure with a basement (one basement parking, one parking at grade level, and four residential levels). Based on our understanding of the project, we are presenting conclusions, opinions, and recommendations regarding:

- appropriate soil profile type and seismic parameters per the 1997 Edition of the UBC, and the 2001 Edition of the CBC,

- specific soil and bedrock conditions discovered by our borings (such as loose or expansive/creeping/collapsible surface soils) that may require special mitigation or impose restrictions on the project, the quality and consistency of the on-site fill soils,

- depth to groundwater and criteria for dealing with shallow groundwater during construction and within the basement excavation,

- potential of the site soils for liquefaction and/or compaction during seismic activities,

- criteria for the design and construction of temporary shoring as well as any tie-backs that may be necessary for this purpose, including the impact of the proposed basement excavation on the adjacent buildings and options to reduce such influences,

- lateral active and at-rest earth pressures acting on the site and building retaining walls and basement walls, including seismic loads to be included in the design of the retaining and basement walls,

- criteria for site grading, preparation of the building pads, basement excavation side slopes, and placement of fills and backfills, including the utility trenches,

- criteria for the support of the basement floors, including the need for structurally-supported or non-structural floors,

- settlements of the building foundations to be supported on the recommended foundation type, provided the building loads are made available to BAGG,

- provisions for surface and subsurface drainage, including back-drainage for the building and site retaining walls, and drainage beneath the basement floor, and

- impact of the soil corrosivity on the buried elements of the building and underground utilities, including the appropriate concrete type per UBC 1997.

To fulfill the above purpose, the scope of our services consisted of the following specific tasks:

- Mark the borings at the site and notify Underground Service Alert at least 48 hours prior to the planned exploration activity.

- Obtain a permit for drilling soil borings from the San Mateo County Department of Environmental Health, as required.

- Research and review pertinent geotechnical and geological maps and reports, including those prepared by BAGG relevant to the site area, United States Geological Survey, and/or other sources regarding the seismic and geologic history of the site and the immediate vicinity.

- Drill, log, and sample three exploratory borings to a depth 60 feet with a truck-mounted drilling rig using hollow-stem augers. Advance the borings under the direction of one of our engineers or geologists who will direct the exploration and obtain samples of the subsurface soil and bedrock materials for visual classification and laboratory testing. Measure the depth to groundwater, if encountered within the depths explored. Backfill the borings with cement grout, as per the San Mateo County Department of Environmental Health. Leave the drill cuttings on-site.

- Perform a laboratory testing program on the collected samples to evaluate the engineering characteristics of the subsurface materials. Tests may consist of direct and triaxial (CU) shear strength, classification, and moisture-density, as judged appropriate.

- Perform engineering analyses based on the results obtained from the above tasks and oriented toward the above-described purpose of the investigation.

- Prepare six copies of a final report summarizing our findings and recommendations, including a vicinity map, a site plan showing the boring locations, one or two subsurface profiles if appropriate, a vicinity geologic map, a regional fault map, boring logs, laboratory test results, and our conclusions, opinions, and recommendations.

4.0 FIELD EXPLORATION AND LABORATORY TESTING

Our subsurface investigation at the site was conducted on July 26 and 27, 2006 with a truck-mounted drilling rig using hollow stem augers to drill 3 borings to a depth of 60 feet. The borings were drilled, at the approximate locations as shown on the attached Plate 2, Site Plan, under the technical supervision of one of our geologists, who kept a continuous log of the materials encountered in each boring, and directed the collection of soil samples for visual examination and laboratory testing. The collected samples were visually examined in our laboratory to confirm the field classifications of the geologic materials encountered, and the field logs were edited as necessary.

Samples of the shallow subsurface materials were obtained with a Modified California a Standard Penetration sampler driven into the soil formation as deemed appropriate.

A laboratory testing program was then designed and conducted to evaluate the engineering properties of the subsurface materials with respect to foundation support. Selected samples were tested in direct shear to evaluate the strength characteristics of the foundation soils at the site. The tests performed were at natural moisture contents, while under varying surcharge pressures. One sample was washed through the number 200 sieve to measure the percent silt and clay in the sample.

The moisture contents and dry densities of selected undisturbed samples were also measured for correlation purposes. The results of these tests are summarized at the appropriate depths on the Boring Logs and on the attached plates.

The graphical representation of the materials encountered in the borings, and the results of laboratory tests as well as explanatory/illustrative data are attached as follows:

- Plate 5, Unified Soil Classification System, illustrates the general features of the soil classification system used on the boring logs.
- Plate 6, Soil Terminology, lists and describes other soil engineering terms used on the boring logs.
- Plate 7, Boring Log Notes, describes general and specific conditions that apply to the boring logs.
- Plates 8, Key to Symbols, describes various symbols used and nomenclature on the boring logs.
- Plates 9 through 11, Boring Logs, describe the soils materials encountered, show the depths and blow counts for the drive samples, show depths, and summarize the results of the strength tests, classification tests, and moisture-density data.
- Plate 12, Plasticity Data, graphs and presents the Atterberg Limits test performed on a representative near-surface soil sample.

To evaluate the corrosion potential of the soils, native at the site, we tested two samples obtained within the upper 10 feet of the site. The samples were tested for resistivity, chloride, sulfate, sulfide, and pH. The corrosion test results of the soils at the site are presented on the attached Appendix, Corrosion Test Summary.

5.0 GEOLOGY AND SEISMICITY

5.1 Regional Geology

The site has been mapped on the border between two geologic units on the Geologic Map and Map Database of the Palo Alto 30' x 60' Quadrangle, California, by E.E. Brabb, R.W. Graymer, and D.L. Jones, Miscellaneous Field Studies Map MF-2332, U.S.G.S., 2000. These units consist of floodplain and alluvial fan and fluvial deposits. The Regional Geology Map on Plate 3 shows the mapped geology of the site vicinity. The map describes these materials as:

> **Floodplain Deposits (Qhfp - Holocene)**
> "Medium to dark gray, dense, sandy to silty clay. Lenses of coarser material (silt, sand, and pebbles) may be locally present. Flood plain deposits usually occur between levee deposits (Qhl) and basin deposits (Qhb)."

> **Alluvial Fan and Fluvial Deposits (Qpaf - Pleistocene)**
> "Brown dense gravely and clayey sand or clayey gravel that fines upward to sandy clay. These deposits display variable sorting and are located along most stream channels in the county. All Qpaf deposits can be related to modern stream courses. They are distinguished from younger alluvial fans and fluvial deposits by higher topographic position, greater degree of dissection, and stronger soil profile development. They are less permeable than Holocene deposits, and locally contain fresh water mollusks and extinct late Pleistocene vertebrate fossils. They are overlain by Holocene deposits on lower parts of the alluvial plain, and incised by channels that are partly filled with Holocene alluvium on higher parts of the alluvial plain. Maximum thickness is unknown but at least 50 meters."

5.2 Seismicity

The project site is located within the western portion of the seismically-active San Francisco Bay region. The nearest major fault is the San Andreas fault, located approximately 5.8 kilometers west-southwest of the project site. This fault generated an earthquake with a

Magnitude of 7.0 on the San Francisco peninsula in 1838, and the great San Francisco Earthquake of 1906, of Moment Magnitude 7.9. The Monte Vista - Shannon fault is located approximately 3 kilometers southwest of the site. This fault is considered capable of generating an earthquake with a Moment Magnitude of 6.7. The Hayward fault, located approximately 24 kilometers east-northeast of the site across the San Francisco Bay, is considered capable of generating an earthquake with a Moment Magnitude 7.3.

The distance to the nearest major active faults to the project site, the fault types, and the moment magnitude of each fault are listed below (ICBO, 1998).

TABLE 1
SIGNIFICANT EARTHQUAKE SCENARIOS

Fault	Approx. Distance to the Site (kilometers)	Potential Moment Magnitude (MW)	Shaking Intensity[1]
San Andreas (Entire)	5.8	7.9[3]	IX - Violent
San Andreas (Peninsula Segment)	5.8	7.2[3]	IX - Violent
Monte Vista - Shannon	3	6.7[2]	VIII - Very Strong
Hayward	24	7.3[3]	VII - Strong

[1] Association of Bay Area Governments, 2003.
[2] Working Group on Northern **California Earthquake Potential, 2003.**
[3] Working Group on California Earthquake Probabilities, 2003.

Maps prepared by ABAG, 2003, indicate the site area will experience a Modified Mercalli Intensity of IX, with “Violent” shaking and “Heavy Damage” as a result of a scenario earthquake on the San Andreas fault, a Modified Mercalli Intensity of VIII with “Very Strong” shaking and “Moderate” damage as a result of a scenario earthquake along the Monte Vista - Shannon fault, and a Modified Mercalli Intensity of VII, with “Strong” shaking and “Nonstructural” damage as a result of a scenario earthquake along the Hayward fault (south). The Modified Mercalli Intensity Scale is presented in Table 2.

TABLE 2

MODIFIED MERCALLI INTENSITY SCALE

(From ABAG, *On Shaky Ground*, 2003)

MMI Value	Description of Shaking Severity (1998 maps)	Summary Damage Description (1995 maps)	Full Description
I			Not felt. Marginal and long period effects of large earthquakes.
II			Felt by persons at rest, on upper floors, or favorably placed.
III			Felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognized as an earthquake.
IV			Hanging objects swing. Vibration like passing of heavy trucks; or sensation of a jolt like a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of IV wooden walls and frame creak.
V	Light	Pictures Move	Felt outdoors; direction estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing, close, open. Shutters, pictures move. Pendulum clocks stop, start, change rate.
VI	Moderate	Objects Fall	Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, etc., off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry D cracked. Small bells ring (church, school). Trees, bushes shaken (visibly, or heard to rustle).
VII	Strong	Nonstructural Damage	Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D, including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices (also unbraced parapets and architectural ornaments). Some cracks in masonry C. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.
VIII	Very Strong	Moderate Damage	Steering of motor cars affected. Damage to masonry C; partial collapse. Some damage to masonry B; none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.
IX	Violent	Heavy Damage	General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. (General damage to foundations.) Frame structures, if not bolted, shifted off foundations. Frames racked. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluvial areas sand and mud ejected, earthquake fountains, sand craters.
X	Very Violent	Extreme Damage	Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.
XI			Rails bent greatly. Underground pipelines completely out of service.
XII			Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.

***Masonry A*:** Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces.

***Masonry B*:** Good workmanship and mortar; reinforced, but not designed in detail to resist lateral forces.

***Masonry C*:** Ordinary workmanship and mortar; no extreme weaknesses like failing to tie in at corners, but neither reinforced nor designed against horizontal forces.

***Masonry D*:** Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.

Full descriptions are from: Richter, C.F. 1958, *Elementary Seismology*, W.H. Freeman and Co., San Francisco, pp135-149, 650-653.

5.3 Potential for Liquefaction

Soil liquefaction is a phenomenon in which saturated (submerged) cohesionless soils can be subject to a temporary loss of strength due to buildup of excess pore pressure, and reduction of soil effective stress during cyclic loading, such as those produced by the earthquakes. In the process, the soil acquires a mobility sufficient to permit both horizontal and vertical movements, if not confined. Soils most susceptible to liquefaction are loose, clean, saturated, uniformly-graded, fine-grained sands. Silty sands and clayey sands may also be susceptible to liquefaction during strong ground shaking, although to a lesser extent. The loose to medium dense sand layers can also be subjected to seismic compaction, if they are above the water table.

Our boring information indicates the presence of medium dense to dense and saturated granular soils at depths ranging from 10 to 17 feet and 36 to 50 below the ground surface. It is therefore, our opinion that the site soils have a moderate potential for liquefaction as a result of shaking caused by a major earthquake on the San Andreas or any other active faults in the region. Potential consequences of liquefaction include minor settlements and differential settlements at the ground surface, as discussed later in this report.

6.0 SITE CONDITIONS

6.1 Subsurface Conditions

Borings B-1 and B-2 encountered fill soils from the surface of the site to a depth of approximately 2½ feet. The fill consisted of stiff sandy lean clay (CL).

Below the fill, the borings encountered materials generally consistent with those mapped on the geologic map of the site vicinity. The native soils consisted of heterogenous layers of lean to fat clay (CL/CH), lean clay (CL), sandy lean clay (CL), sandy silt (ML), occasionally gravelly clayey sand (SC), silty sand (SM), poorly-graded sand with clay (SP-SC), and well-graded sand (SW) to the depths explored.

The clayey cohesive portion of the subsurface materials were generally stiff to very stiff, whereas the consistency of the predominantly silty soils ranged from medium stiff to very stiff. The granular soils beneath the site as encountered in the borings, have a widely-varying consistency. These materials were generally medium dense to loose in the upper 16 feet of the borings. Below 16 feet, the granular soils in the borings were found to be generally medium dense with the exception of random layers which were loose and dense.

For more information regarding the subsurface materials, we refer you to Plate 9, 10, and 11, Boring Logs.

6.2 Groundwater

Groundwater was encountered in the borings at respective depths of 13, 16, and 16 feet in Borings B-1, B-2, and B-3. Please note that these levels may not be the stabilized groundwater levels, as the borings were backfilled with grouted shortly after the last soil samples were collected and the groundwater levels were measured. The borings could not be left open beyond the exploration period, as they would most likely cave or collapse because of the presence of extensive loose and saturated granular soils. Many zones of wet soils were present in the borings at depths generally below the anticipated depth of the basement excavation.

Note that groundwater levels could also fluctuate as a result of seasonal changes and development of seepage layers in the subsurface during the rainy season.

7.0 DISCUSSION AND RECOMMENDATIONS

7.1 General

Based on the information obtained from our investigation, it is our opinion the proposed building is feasible from a geotechnical point of view, provided the recommendations presented in this report are incorporated into the design and construction of the project. When the final project plans and specifications have been prepared, they should be reviewed by this office to verify that the intent of our recommendations have been incorporated into the plans, as well as to verify that the recommendations contained in this report adequately address the project in its final form.

The site is underlain by loose granular soils subject to densification (seismic compaction) and liquefaction during major seismic events. The consequences of noted hazards will be settlement of the ground surface, the magnitude of which has been estimated to be a maximum of 2 inches. Because the loose granular soils underlie the site in a non-uniform fashion, the ground settlements are expected to be uneven. Therefore, the differential settlements of the ground surface could potentially range from 1 inch to 3 inches. Moreover, the foundation soils have been found to have widely varying consistencies which may contribute to differential settlements under the building loads. For these reasons, we recommend a basement level mat foundation for the building. Such a foundation system would be rigid enough to mitigate the projected differential settlements without any undue damage to the superstructure.

Groundwater was measured in the borings at depths ranging from 13 to 16 feet below the ground surface. Because the measured levels are not stabilized groundwater levels, and the groundwater may actually be somewhat higher, there is a possibility that the proposed basement excavation may be impacted by the groundwater rising to the bottom of the excavation due to construction equipment, etc., and saturating the foundation soils. This condition which may be mitigated by appropriate dewatering schemes and placement of a layer of crushed rock at the bottom of the excavation as discussed later in this report, also signifies the need for a basement level mat foundation. Such a foundation will provide uniform support to the subject building.

The site could experience very strong ground shaking from future earthquakes during the anticipated lifetime of the project. The intensity of the ground shaking will depend on the magnitude of the earthquake, distance to the epicenter, and the response characteristics of the native soils and bedrock. While it is not possible to totally preclude damage to structures during major earthquakes, strict adherence to good engineering design and construction practices will help reduce the risk of damage to the residential condominium project and the associated improvements.

7.2 Site Grading

As used in this report, the term "compact" and its derivatives mean that all engineered fill material, whether imported or on-site material, should be compacted to at least 95 percent of maximum dry density as determined by ASTM Test Method D1557-01, within the upper 12 inches of pavement subgrades and beneath all footings supporting the building frame, and to at least 90 percent compaction in other areas. The term also implies that immediately prior to being compacted, the fill material should be thoroughly moisture conditioned to a moisture content that is slightly above optimum as determined by the above Test Method, and spread evenly in lifts not exceeding 8 inches in thickness, and that each lift should be thoroughly compacted before subsequent lifts are placed.

All street level areas to be developed should be cleared of deleterious materials, including old fill which should be removed from the site, or stockpiled for use in landscape areas only. All soils disturbed by these operations should be removed from the areas to be graded, and any existing fill soils that do not meet the required compaction standards, should be re-worked and re-compacted as necessary.

Any areas of soft or yielding soils should be further excavated down to firm soils as directed by the Soil Engineer in the field. The areas may then be brought up to final grade with engineered fill placed in properly compacted lifts. The Soil Engineer should be present during all phases of grading to verify that suitable foundation soils have been exposed for the support of various elements of the proposed improvements.

In general, soils excavated from the site are suitable for use as engineered fill material, provided they are free of organic matter and contain no rocks or lumps greater than 4 inches in greatest dimension. Any fill material imported to the site should be a predominantly granular soil that contains no organics and is approved by the Soil Engineer before being imported to the site. As a general guide to acceptance, the imported material should have no particles larger than 4 inches in maximum dimension, a Plasticity Index less than 15, a minimum R-value of 20, a fines content of between 15 and 65 percent, and be approved by the Geotechnical Engineer before importing to the site.

It must be the Contractors responsibility to select equipment and procedures that will accomplish the grading as recommended herein. He must also coordinate his grading schedule with BAGG so that one of our field representatives can observe and test the grading operations, including clearing, stripping, over-excavation, compaction of fill and backfill, and preparation of subgrades.

7.3 2001 CBC Site Characterization

Significant advances have been made in recent years for better design against the destructive power of earthquakes. Much of these advances have been incorporated into the 2001 edition of the California Building Code (CBC). Based on our borings and laboratory testing, the soil profile at the site is defined as S_D, a stiff soil profile. The closest seismic source is the San Andreas, a Seismic Source Type A, located 5.8 km from the site. These values should be used in Tables 16-Q through 16-T of the 2001 CBC.

Based on our geologic research, including published maps of known active fault zones prepared for the 1997 UBC and the distance to the seismic sources, the seismic design parameters tabulated below are recommended for this site, based on Chapter 16 of the 1997 UBC (similar to 2001 California Building Code).

TABLE 3
PARAMETERS FOR SEISMIC DESIGN

UBC, 1997	Site Parameter
Figure 16-2, Seismic Zone Map of the U.S.	Zone 4
Table 16-I, Seismic Zone Factor Z	0.4
Table 16-J, Soil Profile Type	S_D, Stiff Soil Profile
Table 16-Q, Seismic Coefficient C_a	$0.44N_a$
Table 16-R, Seismic Coefficient C_v	$0.64N_v$
Table 16-U, Seismic Source Type	A (San Andreas)
Closest Distance to Known Seismic Source	5.8 kilometers
Table 16-S, Near-Source Factor, N_a	1.2
Table 16-T, Near-Source Factor, N_v	1.6

7.4 Foundations

As discussed above, to span over localized irregularities in soil conditions, and to spread out settlements due to densification and liquefaction of the loose to medium dense granular materials underlying the site, the proposed structure should be supported on a mat foundation. The mat should be designed to tolerate potential differential settlement caused by liquefaction and seismic compaction of the granular soils underlying the site, on the order of 1 inch over a horizontal distance of 30 feet in both directions.

The mat should be a two way reinforced concrete feature with a minimum thickness of 12-inch. Structural consideration may dictate a thicker mat. The allowable bearing pressure at the bottom of the mat may be taken as 1,000 pounds per square foot (psf) for dead loads, and 1,500 psf for total design loads. The latter value may be increased by one-third for short-term wind and seismic loads. The foundation system should be designed as a rigid unit capable of tolerating the differential settlements estimated above.

The bottom of the basement excavations should be over excavated 12 inches and a free draining gravel layer installed as a working base for the construction of the mat. This would eliminate the need for compacting the bottom of the excavation which would otherwise be very difficult.

Where the recommended mat cannot effectively resist seismic uplift loads, seismic anchors may be designed using an allowable skin friction value of 500 psf below a depth of 16 feet.

7.5 Settlements

We have estimated that the seismically-induced settlements at the subject site will be a maximum of 2 inches, also causing differential settlements on the order of 1 inch over a horizontal distance of 30 feet.

Static settlements due to the weight of the building will be relatively minor, because the planned basement will likely remove approximately 10 feet of soil. The weight of the removed soil would likely be similar in magnitude to the structural loads on the basement mat foundation, which may be a uniform load of approximately 1,000 psf, or less. With a mat foundation supporting the building at the basement level, the differential settlements between the adjacent columns will be less than ½ inch, possibly no more than ¼ inch. However, the settlement of the ramp may impact the mat foundation depending on how it is supported, and/or the way it interacts with the mat foundation. Because the project is in early stages, these issues must be addressed once the project details and the building loads are known.

The proposed building may contain non-basement portions constructed at the street level. We recommend limiting the structural loads for such features to a minimum, because settlements will result from the structural loads in non-basement areas, in turn causing differential settlements between the basement and the non-basement portions of the building. A more refined estimate of settlements will be made once the structural loads in the basement and non-basement areas have been determined and transmitted to this office.

7.6 Subgrade Modulus

Based on the consistency of the subsurface materials and the anticipated settlements, we have estimated that a modulus of subgrade reaction of 100 pci (psi per inch) may be used for the design of the mat foundation for the building.

7.7 Retaining Walls

Cantilever retaining walls should be designed to resist an active soil pressure taken as an equivalent fluid pressure of 40 pounds per cubic foot (pcf). Restrained walls and walls that are part of the building foundation system should be designed to resist at-rest soil pressures of 60 pcf. These pressures are for level backfill conditions, and should be increased by 3 pcf for every 5 degree increase in backfill slope above the wall. Seismic loads on retaining walls should be taken as a uniform pressure equal to 15H psf, where "H" is the height of the wall in feet.

In general, the retaining walls should also be designed to resist the additional lateral surcharge pressures equal to 60 percent of vertical surcharge loads such as footings from the adjacent buildings, slabs, etc., that are present in the close proximity of the basement.

Specific recommendations for surcharge loads may be made after the relationship between the applied surcharge loads and the proposed retaining walls have been established.

Basement walls may be supported on the mat foundation designed in accordance with the recommendations presented in the "Foundations" section of this report. The lateral earth pressures should be resisted by passive soil pressures and friction acting on the wall foundations as described below under "Lateral Design".

The above lateral pressures do not include any hydrostatic pressures resulting from seepage water or infiltration of natural rainfall and/or irrigation water behind the walls. Therefore all walls over 2 feet in height should be provided with a drainage blanket behind the wall. The drainage blanket may consist of a pre-manufactured drainage panel, or a one-foot-thick blanket of granular material consisting of either free-draining gravel or drain rock protected by a suitable filter fabric, or Class 2 Permeable Material complying with the latest Caltrans Standard Specifications. An 18-inch cap of relatively impermeable soil should be compacted at the top of the drainage blanket to minimize infiltration of surface water. A perforated pipe, with perforations placed down, should be installed at the base of the drainage blanket to conduct water away from the wall. The collected seepage water should be carried to a suitable outfall and discharged in a manner that will not cause erosion or over-saturation of soils in the vicinity of improvements. The wall drain should be kept separate from the storm drain system (down-spouts, etc.), and if the outfall is to be at a local catch basin, it may be necessary to provide measures that assure water will not back up into the wall drain system during peak storm flows. Generally, the one-foot-thick gravel blanket, located under the mat foundation, should be tied to the gravel drains behind the retaining walls, so that any future high groundwater can be pumped into the storm drain pipes.

General backfill material behind the walls, excluding drainage material, should conform to the fill material requirements given under "Site Grading", which may include soils excavated from the site. The backfill soils should also be properly compacted as recommended in that section. If heavy compaction equipment is used adjacent to the walls, the walls should be temporarily braced.

7.8 Lateral Design

Lateral loads can be resisted by a combination of friction acting on the bottom of mat and passive soil pressures acting on the sides of foundation walls. Within properly compacted fill soils and/or undisturbed native soils, the passive soil pressures should be taken as 300 pounds per cubic foot (psf per foot of depth). When resisting long-term loads, such as from retaining walls, the passive pressure within the upper 12 inches should be ignored in the non-basement areas, unless the footing is protected by an adjacent slab or pavement.

A coefficient of friction of 0.30 can be used between the bottom of concrete mat and gravel blanket below the mat.

7.9 Concrete Slabs-on-Grade

Slab-on-grade floors should be supported on a subgrade constructed as recommended under "Site Grading". In addition, the subgrade soils should be kept at a moisture content that is slightly above optimum (per ASTM D1557-01) until the slab is poured. This condition should be verified by this office immediately before the slab is poured.

If it is desired to prevent dampness on slab floors in enclosed areas, the slab should be underlain by at least 4 inches of free-draining gravel, such as ¾ by No. 4 crushed rock, to act as a capillary break between the subgrade soil and the slab. To minimize vapor transmission, a vapor barrier such as a 10-mil polyethylene sheet should be placed over the gravel. The vapor barrier can be covered with 2 inches of sand to protect the membrane during construction, with the sand layer wetted (not saturated) just prior to pouring the slab to aid in curing the concrete.

7.10 Temporary Shoring Walls

The temporary shoring of vertical excavations should conform to OSHA requirements as defined in the State of California Department of Transportation, Trenching and Shoring Manual. At this time the proposed temporary shoring walls for the basement may be designed as a drilled pier and wood lagging wall, to limit movement of the top, the temporary shoring walls may require a tieback rod at the top of the wall. Temporary shoring walls may be designed to withstand an active earth pressure of 30H, where H is the height of the vertical cut to be shored. Where a sloping surface exists above the temporary shoring, the pressures should be increased by 3H psf for every 5 degree increase in the slope angle, or alternately, the slope height can be added to the shoring height when calculating the lateral soil pressures. Additionally, construction equipment should not be allowed closer than a distance equal to the height of the excavation to the top of the cut slope. Surcharge loads will be in addition to the active earth pressures which should be applied at a rate of 40 percent of the surcharge loads.

Where a temporary sloped excavation is desired, it may be opened a gradient of 1:1, provided the excavation height does not exceed 12 feet.

7.11 Utility Trenches

We anticipate the site materials will stand vertically to a depth of at least 5 feet; however, some areas containing granular soils with limited fines (- #200 sieve) may fail. The contractor should therefore comply with all OSHA requirements. For this purpose, the soils at the site should be classified as Type “B” soils. In addition, trench spoils should not be placed closer than 3 feet, or half the trench depth, from the top of the trench sidewalls.

Backfill within trenches should conform the compaction requirements recommended in “Site Grading.” Jetting should not be allowed in any trenches. All backfill within utility trenches that are below an imaginary 1:1 (horizontal to vertical) plane projected downward from the bottom edge of the footing, and/or within 2 feet of pavement subgrade elevation, should be compacted to at least 95 percent compaction. Backfill within other trench areas should be compacted to at least 90 percent compaction.

7.12 Corrosion Potential

To evaluate the corrosion potential of the on site soils, we retained Conceco/Matcor Engineering to test and evaluate two samples obtained from the borings collected at depths of 5 and 9 feet. The samples were tested for resistivity, chloride, sulfate, sulfide, and pH. The results indicate the soils at the site are “moderately corrosive”. The report from Conceco/Matcor Engineering, Inc. and their recommendations for mitigation of corrosion on the concrete and buried steel/ductile iron pipe are attached as Appendix A at to the end of this report.

7.13 Rigid Pavements

The mat foundation will provide support to the cars in the basement parking. There will be other ramps and on-grade parking and high-traffic areas paved with Portland Cement Concrete (rigid) Pavements. Rigid Pavements should be supported on a subgrade that has been prepared as recommended under "Site Grading". For high-traffic areas, exposed to as much as 10 commercial trucks per day (TI = 7), we recommend the pavement consist of 6½ inches of concrete supported on at least 6 inches of Class II Aggregate Base material that has been compacted to a minimum of 95 percent relative compaction. Where only occasional heavy trucks are expected (one a week, or TI = 6), the concrete slab may be 5½ inches thick underlain with at least 4 inches of Class II Aggregate Base.

These slab thicknesses and traffic conditions would require structural concrete with a minimum compressive strength of 3,000 psi, plus nominal temperature reinforcing steel. The slabs should also have interlocking joints.

7.14 Drainage

The tendency for the on-site clayey soil to lose much of its strength when saturated, dictates th importance of proper drainage control as a part of the subject development. Surface gradients should be provided which direct water away from the proposed structure. Within landscaping, lawn, or other areas, the final ground surface around the building should slope at a gradient of at least 5 percent for a distance of about 5 feet away from foundation lines. Impervious surfaces, such as pavements and concrete slabs or sidewalks adjacent to the building, should also slope away from the building foundations. Water should not be allowed to pond in the vicinity of improvements, and any area where water collects should be provided with a catch basin that is discharged at a suitable outfall, in a manner that will not create erosion.

The structure should have roof gutters and downspouts, and all water from the downspouts should be carried away from the building in a manner that will not cause over-saturation of soils in the vicinity of the foundations.

7.15 Plan Review

The recommendations contained in this report are based on our understanding of the proposed project as described herein. The recommendations presented in this report are therefore contingent upon Bay Area Geotechnical Group being retained to review all final development plans, including grading, drainage, and foundation plans. Such a review is required to verify that our understanding of the project corresponds to the final development plans, and as well as to verify that the intent of our recommendations have been incorporated into the final plans.

7.16 Construction Observation and Testing

It is recommended that the Geotechnical Engineer (BAGG) be retained to provide observation and testing services during all phases of the grading, including excavation, foundation construction, and retaining wall backfill. This is to observe compliance with the design concept, specifications and recommendations, and will allow for design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

8.0 CLOSURE

This report has been prepared in accordance with generally-accepted engineering practices for the strict use of Nicholson-Lamb Venture, LP, and other professionals associated with the specific project described in this report. The recommendations presented in this report are based on our understanding of the proposed construction as described herein, and upon the soil conditions encountered in three widely spaced exploratory borings drilled for this investigation. It is not uncommon for unanticipated conditions to be encountered during site excavation, grading and/or foundation installation, and it is not possible for all such variations to be found by a field exploration program appropriate for this type of project. The recommendations contained in this report are therefore contingent upon the review of the final project plans by this office, and upon geotechnical observation and testing by BAGG of all pertinent aspects of construction, including clearing, site grading, basement and foundation excavations, placement of fills or backfills, and preparation of subgrades.

Subsurface conditions and standards of practice change with time. Therefore, we should be consulted to update this report, if the construction does not commence within 18 months from the date this report is submitted. Additionally, the recommendations of this report are only valid for the proposed development as described herein. If the proposed project is modified, our recommendations should be reviewed and approved or modified by this office in writing. Further, this report should not be used for similar projects on other sites, nor for different types of development on this site without review and approval by this office.

The following references and plates are attached and complete this report:

Attached Plates:

Plate 1	Vicinity Map
Plate 2	Site Plan
Plate 3	Regional Geology Map
Plate 4	Regional Fault Map
Plate 5	Unified Soil Classification System
Plate 6	Soil Terminology
Plate 7	Boring Log Notes
Plate 8	Key to Symbols
Plates 9-A through 11-C	Boring Logs
Plate 12	Plasticity Data
Appendix A	Corrosion Testing Report

ASFE document titled "Important Information About Your Geotechnical Engineering Report"

9.0 REFERENCES

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