GEOTECHNICAL INVESTIGATION FOR PROPOSED APARTMENT BUILDING
at the
Sylvan Avenue Property
500 Sylvan Avenue
San Bruno, California

Report Prepared for:

San Bruno, Inc.

Report Prepared by:

GeoForensics, Inc.

August 2017

GEOFORENSICS INC. **Consulting Soil Engineering**

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File: 217179
August 6, 2017

San Bruno Inc.
535 Sylvan Avenue
San Bruno, CA 94066

Attention: Suresh Ghandi

Subject: **Sylvan Avenue Property**
500 Sylvan Avenue
San Bruno, California
GEOTECHNICAL INVESTIGATION FOR
PROPOSED NEW APARTMENT COMPLEX

Mr. Ghandi:

In accordance with your authorization, we have performed a subsurface investigation into the geotechnical conditions present at the location of the proposed improvements. This report summarizes the conditions we measured and observed, and presents our opinions and recommendations for the design and construction of the proposed new apartment complex.

Site Description

The subject site is a relatively level, rectangularly-shaped parcel located on the northwestern side of Sylvan Avenue (at the approximate location shown on Figure 1). For purposes of description in this report, it is assumed that the property faces south. The property is bounded by other developed apartment complexes north and west, Sylvan Avenue to the south, and Green Drive to the east.

The site is currently occupied by a one-story, wood-framed commercial structure with a partial basement The wooden house floors are supported above crawlspace areas and the partial basement. An asphalt parking lot rings the building on the north and western sides.

The ground surface in the site vicinity has an overall slope down towards the northeast (as shown on Figure 2). At the site, the ground slopes gently to the northeast. Surface gradients range from 20:1 to 10:1 (horizontal:vertical, H:V). During the original development of the property, it appears that little or no grading work was performed on the site.

Proposed Construction

We understand that the current development for the site proposes the demolition of the existing office building, and the subsequent construction of a new apartment building. The building is to be of conventional, wood-framed construction, with parking and a ground story unit, covered by an additional two stories of apartments. New foundation loads are expected to be typical for this type of structure (i.e. light).

Excavation work at the site is expected to be limited to foundation excavations. No significant fill placement is anticipated as part of this work. No significant retaining walls are anticipated for this scope of work. No basement or pool is planned for the project.

INVESTIGATION

Scope and Purpose

The purpose of our investigation was to determine the nature of the subsurface soil conditions so that we could provide geotechnical recommendations for the construction of the proposed new building and associated improvements. In order to achieve this purpose, we have performed the following scope of work:

1 - visited the property to observe the geotechnical setting of the area to be developed;

2 - reviewed relevant published geotechnical maps;

3 - drilled 3 borings near the location of the proposed improvements;

4 - performed laboratory testing on collected soil samples;

5 - assessed the collected information and prepared this report.

The findings of these work items are discussed in the following sections of this report.

Site Observations

We visited the site on July 5, 2017 to observe the geotechnically relevant site conditions. During our visit, we noted the following conditions:

A - The existing building appears to be supported by a perimeter concrete footing with isolated interior wooden posts resting on concrete pedestals, and partially over a basement. The foundation system appeared to be in good condition, with no major cracks (as observed from the exterior).

B - There is a roughly 30 inch diameter depression in the parking lot to the west of the building. This depression may be associated with rotting of an old tree stump, but this should be further investigated during construction. The asphalt pavement appears to be in relatively good condition otherwise.

C - The stucco surfacing on the building walls also appears to be in good condition with no major cracking apparent.

D - The site drainage generally consists of sheet flow through the parking lots to the adjacent streets.

Geologic Map Review

We reviewed the *Geology of the Onshore Part of San Mateo County, California: Derived from the Digital Database Open-File 98-137*, by Earl E. Brabb, R.W. Graymer, and D.L. Jones (1998). The relevant portion of the Brabb, Graymer, and Jones map has been reproduced in Figure 3.

The Brabb, Graymer, and Jones map indicates that the site is underlain by the Colma Formation (map symbol "Qc"). The authors describe these materials as consisting of yellowish gray to orange brown, friable to loose, sand with subordinate amounts of gravel, silt, and clay.

Our subsurface exploration (see below) encountered clays and sands which we judged to be consistent with the mapping.

The active San Andreas fault is mapped approximately 3 km to the southwest of the site.

Subsurface Exploration

On July 5, 2017 we drilled 3 borings at the site at the locations shown on Figure 4. The borings were drilled using a Mobile B-24 truck-mounted drilling rig equipped with 4.0 inch diameter, helical flight augers. Logs of the soils encountered during drilling record our observations of the cuttings traveling up the augers and of relatively undisturbed samples collected from the base of the advancing holes. The final boring logs are based upon the field logs with occasional modifications made upon further laboratory examinations of the recovered samples and laboratory test results. The final logs are attached in Appendix A.

The relatively undisturbed samples were obtained by driving a 3.0 inch (outer diameter) Modified California Sampler and a Standard Penetration Sampler (as noted on logs) into the base of the advancing hole by repeated blows from a 140 pound hammer lifted 30 inches. On the logs, the number of blows required to drive the sampler the final 12 inches of the 18 inch drive, have been recorded as the Blow Counts. These blows <u>have not</u> been adjusted to reflect equivalent blows of any other type of sampler or hammer, or to account for the different samplers used.

Subsurface Conditions

Borings 1 and 2 encountered similar subsurface soil conditions. After penetrating through the parking lot asphalt, the borings encountered an upper soft layer of clayey silt, extending to depths between 9 to 11 feet. Medium dense silty sands were then penetrated to a depth of 17 to 19 feet, before the borings encountered clayey sands and sandy clays to the base of the borings at about 25 feet.

Boring 3, encountered loose to medium dense silty sands to a depth of 15 feet, then clayey sand to 20 feet (base of boring)

Please refer to Appendix A for a more detailed description of each boring.

Ground water was encountered at depths of 8 to 12 feet below grade in the borings. However, during periods of heavy rain or late in the winter, groundwater seepage may exist at shallower depths, most likely as perched water within the sandy soils.

Laboratory Testing

The relatively undisturbed samples collected during the drilling process were returned to the laboratory for testing of engineering properties. In the lab, selected soil samples were tested for moisture content, density, and consolidation. The results of the laboratory tests are attached to this report in Appendix B.

The relatively undisturbed sample was returned to the laboratory for testing of compressibility. The testing suggested that the Bay Mud has a pre-consolidation pressure of about 2000 psf which is higher than the overburden pressure currently pressing on top of the soil. Therefore, we consider these soft upper soils to be lightly over-consolidated, with an Over-Consolidation Ratio (OCR) of approximately 4.

CONCLUSIONS AND RECOMMENDATIONS

General

Based upon our investigation, we believe that the proposed improvements can be safely constructed. Geotechnical development of the site is controlled by the presence of soft near surface soils underlain by a moderately thick layer of potentially liquefiable soil.

The presence of these soft and liquefiable soils suggests that the foundation system for the new apartment complex should penetrate through these soils into the more stable materials below. Due to the shallow ground water, drilling difficulties may be experienced with conventional drilled piers requiring casing, drill-and-pour methods, or alternatively ground improvements such as rammed concrete columns, rammed stone columns, or soil-concrete columns.

The recommendations in this report should be incorporated into the design and construction of the proposed new apartment building.

Seismicity

The greater San Francisco Bay Area is recognized by Geologists and Seismologists as one of the most active seismic regions in the United States. Several major fault zones pass through the Bay Area in a northwest direction which have produced approximately 12 earthquakes per century strong enough to cause structural damage. The faults causing such earthquakes are part of the San Andreas Fault System, a major rift in the earth's crust that extends for at least 700 miles along western California. The San Andreas Fault System includes the San Andreas, San Gregorio, Hayward, Calaveras Fault Zones, and other faults.

During 1990, the U.S. Geological Survey cited a 67 percent probability that an earthquake of Richter magnitude 7, similar to the 1989 Loma Prieta Earthquake, would occur on one of the active faults in the San Francisco Bay Region in the following 30 years. Recently, this probability was increased to 70 percent, as a result of studies in the vicinity of the Hayward Fault. A 23 percent probability is still attributed specifically to the potential for a magnitude 7 earthquake to occur along the San Andreas Fault by the year 2020.

Ground Rupture - The lack of mapped active fault traces through the site, suggests that the potential for primary rupture due to fault offset on the property is low.

Ground Shaking - The subject site is likely to be subject to very strong to violent ground shaking during its life span due to a major earthquake in one of the above-listed fault zones. Current (2016) building code design may be followed by the structural engineer to minimize damages due to seismic shaking, using the following input parameters from the USGS Java Ground Motion Parameter Calculator based upon ASCE 7-10 design parameters:

Site Class - D	$SM_S = 2.372$	$SM_1 = 1.708$	$SD_S = 1.582$	$SD_1 = 1.139$

Landsliding - The subject site and the surrounding area are relatively flat. Therefore, the hazard due to seismically-induced landsliding is, in our opinion, very low for the site.

Liquefaction - Liquefaction most commonly occurs during earthquake shaking in loose fine sands and silty sands associated with a high ground water table. These conditions were encountered at the subject site. Our borings suggest that liquefaction of the soils can be expected to develop during a major earthquake between about 10 and 20 feet below grade. Therefore, it is our opinion the liquefaction is likely to occur on the subject property, with potentially significant settlements associated with the process. Therefore, we have provided recommendations to transmit foundation loads to the deeper non-liquefiable materials.

Ground Subsidence - Ground subsidence may occur when poorly consolidated soils densify as a result of earthquake shaking. The upper soils do not appear to be potentially subject to densification, but are potentially subject to liquefaction induced settlements (see above).

Lateral Spreading - Lateral spreading may occur when a weak layer of material, such as a sensitive or liquefiable soil, loses its shear strength as a result of earthquake shaking. Overlying blocks of competent material may be translated laterally towards a free face. Free face conditions are not present proximate to the site, hence, the hazard due to lateral spreading is, in our opinion, considered to be low.

Site Preparation and Grading

All debris resulting from the demolition of existing improvements should be removed from the site and may not be used as fill. Any existing underground utility lines to be abandoned should be removed from within the proposed building envelope and their ends capped outside of the building envelope.

Any vegetation and organically contaminated soils should be cleared from the building area. All holes resulting from removal of tree stumps and roots, or other buried objects, should be over-excavated into firm materials and then backfilled and compacted with native materials.

Due to the somewhat compressible nature of the upper site soils, we recommend limiting new loads on these upper soils. Therefore, we do not recommend the application of fill to raise site grades.

The placement of fills at the site is expected to be limited to: utility trench backfill, slab subgrade materials, and finished drainage and landscaping grading. These and all other fills should be placed in conformance with the following guidelines:

Fills may use organic-free soils available at the site or import materials. Import soils should be free of construction debris or other deleterious materials and be non-expansive. *A minimum of 3 days prior to the placement of any fill, our office should be supplied with a 30 pound sample (approximately a full 5 gallon bucket) of any soil or baserock to be used as fill (including native and import materials) for testing and approval.*

All areas to receive fills should be stripped of organics and loose or soft near-surface soils. Fills should be placed on level benches in lifts no greater than 6 inches thick (loose) and be compacted to at least 90 percent of their Maximum Dry Density (MDD), as determined by ASTM D-1557. In pavement (concrete or asphalt) areas to receive vehicular traffic, all baserock materials should be compacted to at least 95 percent of their MDD. Also, the upper 6 inches of soil subgrade beneath any pavements should be compacted to at least 95 percent of its MDD.

If fills in excess of 2 feet thick are to be placed, our office should be contacted for further recommendations.

Temporary, dry-weather, vertical excavations should remain stable for short periods of time to heights of 5 feet. All excavations should be shored or sloped in accordance with OSHA standards. Cuts deeper than 8 feet will encounter groundwater and will require temporary (and perhaps permanent) dewatering.

Permanent cut and/or fill slopes should be no steeper than 3:1 (H:V). However, even at this gradient, minor sloughing of slopes may still occur in the future. Positive drainage improvements (e.g. drainage swales, catch basins, etc.) should be provided to prevent water from flowing over the tops of cut and/or fill slopes.

Ground Improvement Piers

If the upper soils are subject to appropriate ground improvement techniques, the foundation system may consist of a mat slab. Suitable ground improvement techniques include: rammed concrete or stone piers, soil-concrete mix piles, among other techniques. When implemented, these techniques provide for strengthened soil conditions at, or around the localized area of improvement, which permits reliance of these improvements for transmission of vertical loads to the underlying stronger strata. Most of these types of ground improvement are based upon proprietary designs, hence we are not providing any design parameters for the improvements. We note that the improvements must extend to a depth of 20 feet below grade, and that we should review the proposed improvements to verify that they will result in ground conditions compatible with the proposed foundation systems.

Foundations

If ground improvement work is not conducted at the site, then we recommend a pier and grade beam foundation system be used.

Piers should penetrate a minimum of 25 feet below lowest adjacent grade, and 10 feet into competent native materials, whichever is deeper. It should be assumed that up to 20 feet of overburden will exist at the site, so nominal pier depths may range from 25 to 30 feet below lowest adjacent grade.

The piers should have a minimum diameter of 16 inches and be nominally reinforced with a minimum of four #5 bars vertically. Piers should be spaced no closer than 4 diameters, center to center. Contact our office for group pier reductions if spacings at less than 4 diameters are desired.

Holes greater than 8 feet will encounter groundwater. The contractor should be prepared to tremmie the piers, drill and pour the piers, and/or case the piers in the event of caving.

Actual pier depth, diameter, reinforcement, and spacing should be determined by the structural engineer based upon the following design criteria:

A friction value of 500 psf may be assumed to act on that portion of the pier below the potentially liquefiable layers (assume a depth of 18 feet for design). Long-term lateral support may be assumed to be developed along the length of the pier below 18 feet, using a uniform passive pressure of 2000 psf. Passive resistance may be assumed to act over 1.5 projected pier diameters. Lateral resistance to short term loads (e.g. earthquake or wind) may be achieved through a passive pressure of 500 psf on any embedded foundation element (grade beams and the upper 5 feet of piers).
Even though piers are designed to derive their vertical resistance through skin friction, the bases of

the piers holes should be clean and firm prior to setting steel and pouring concrete. If more than 6 inches of slough exists in the base of the pier holes after drilling, then the slough should be removed. If less than 6 inches of slough exists, the slough may be tamped to a stiff condition. Piers should not remain open for more than a few days prior to casting concrete. In the event of rain, shallow groundwater, or caving conditions it may be necessary to pour piers immediately.

All perimeter piers, and piers under load-bearing walls, should be connected by concrete grade beams or a mat slab.

All improvements connected directly to any pier supported structure, also need to be supported by piers. This includes, but is not limited to: porches, decks, entry stoops and columns, etc. If the designer does not wish to pier support these items, then care must be taken to structurally isolate them (with expansion joints, etc.) from the pier supported structure.

If the above recommendations are followed, total foundation settlements should be less than 1 inch, while differential settlements should be less than ½ inches.

Mat Slabs

Where ground improvement techniques are employed, the foundation system may consist of structural mat slabs. These slabs should not impart a pressure on the ground in excess of 1500 psf in order to limit potential settlements of the upper soils. The slabs may be designed assuming a modulus of subgrade reaction of 20 pci. Slabs should also be designed to be able to span any untreated areas between ground improvement piers, without excessive deflections. Ideally, building columns should be located directly over ground improvement piers.

Structural Slabs

Where drilled piers will support the building, the ground story slabs should be designed to span between the piers and grade beams for the lower story apartments and parking areas.

Retaining Walls

No new retaining walls are proposed for this scope of work. If plans should change to include retaining walls, then our office should be contacted for additional recommendations.

Slabs-on-Grade

Any driveway, sidewalks or patios, may consist of conventional concrete slabs-on-grade though it should be expected that some post-construction shifting of such slabs will occur. We have provided guidelines to help reduce post-construction movements, however, it is nearly impossible to economically eliminate all shifting.

To help reduce cracking, we recommend slabs be a minimum of 4 inches thick and be nominally reinforced with #4 bars at 18 inches on center, each way. Slabs which are thinner or more lightly reinforced may experience undesirable cosmetic cracking. However, actual reinforcement and thickness should be determined by the structural engineer based upon anticipated usage and loading.

In large non-interior slabs (e.g. patios, etc.), score joints should be placed at a maximum of 10 feet on center. In sidewalks, score joints should be placed at a maximum of 5 feet on center. All slabs should be separated from adjacent improvements (e.g. footings, porches, columns, etc.) with expansion joints. Interior floor slabs (garage and lower story units) will experience shrinkage cracking. These cosmetic cracks may be sealed with epoxy or other measures specified by the architect.

All interior slabs (including mat slabs and structural slabs) should be underlain by 4 inches of clean ¾ inch crushed drain rock. The drain rock should be covered by a vapor barrier which conforms to ASTM E1745-97 (e.g. Stego Wrap or an approved equivalent). The architect or structural engineer should determine if sand is required over the vapor barrier.

Slabs which will be subject to light vehicular loads <u>and</u> through which moisture transmission is not a concern (e.g. driveway) should be underlain by at least 6 inches of compacted baserock, in lieu of any sand and gravel. Exterior landscaping flatwork (e.g. patios and sidewalks) may be placed directly on proof-rolled soil subgrade materials (e.g. no granular subgrade), however, they will be potentially subject to shifting and moisture transmission.

As stated previously, in pavement (concrete or asphalt) areas to receive vehicular traffic, all baserock materials should be compacted to at least 95 percent of their MDD. Also, the upper 6 inches of native soil subgrade beneath any pavements should be compacted to at least 95 percent of its MDD.

<u>Drainage</u>

Surface Drainage - Adjacent to any buildings, the ground surface should slope at least 5 percent away from the foundations within 5 feet of the perimeter. Impervious surfaces should have a minimum gradient of 2 percent away from the foundation.

Surface water should be directed away from all buildings into drainage swales, or into a surface drainage system (i.e. catch basins and a solid drain line). "Trapped" planting areas should not be created next to any buildings without providing means for drainage (i.e. area drains).

All roof eaves should be lined with gutters. The downspouts may be connected to solid drain lines, or may discharge onto paved surfaces which drain away from the structure. The downspouts may be connected to the same drain line as any catch basins, but must not connect to any perforated pipe drainage system.

Under-Slab Drains - Due to the potential for changes to surface drainage provisions, it would be wise (though not required) to install under-slab drains at the base of the drain rock section under any habitable portion of the lower story slabs to minimize the potential for moisture penetrate un through the floor slabs. We would suggest that in these areas, the perforated collector pipes should be spread no more than 20 feet apart.

Drainage Materials - Drain lines should consist of hard-walled pipes (e.g. SDR 35 or Schedule 40 PVC). In areas where vehicle loading is not a possibility, SDR 38 or HDPE pipes may be used. Corrugated, flexible pipes may not be used in any drain system installed at the property.

Surface drain lines (e.g. downspouts, area drains, etc.) should be laid with a minimum 2 percent gradient (¼ inch of fall per foot of pipe). Any subsurface drain systems (e.g. footing drains) should be laid with a minimum 1 percent gradient (1/8 inch of fall per foot of pipe).

Plan Review and Construction Observations

The use of the recommendations contained within this report is contingent upon our being contracted to review the plans, and to observe geotechnically relevant aspects of the construction.

We should be provided with a full set of plans to review at the same time the plans are submitted to the building/planning department for review. A minimum of one working week should be provided for review of the plans.

At a minimum, our observations should include: ground improvement work; compaction testing of fills and subgrades; footing excavations; pier drilling; slab and driveway subgrade preparation; installation of any drainage system (e.g. back-of-wall, under-slab, footing, and surface), and final grading. A minimum of 48 hours notice should be provided for all construction observations.

LIMITATIONS

This report has been prepared for the exclusive use of the addressee, and their architects and engineers for aiding in the design and construction of the proposed development. It is the addressee's responsibility to provide this report to the appropriate design professionals, building officials, and contractors to ensure correct implementation of the recommendations.

The opinions, comments and conclusions presented in this report were based upon information derived from our field investigation and laboratory testing. Conditions between or beyond our borings may vary from those encountered. Such variations may result in changes to our recommendations and possibly variations in project costs. Should any additional information become available, or should there be changes in the proposed scope of work as outlined above, then we should be supplied with that information so as to make any necessary changes to our opinions and recommendations. Such changes may require additional investigation or analyses, and hence additional costs may be incurred.

Our work has been conducted in general conformance with the standard of care in the field of geotechnical engineering currently in practice in the San Francisco Bay Area for projects of this nature and magnitude. We make no other warranty either expressed or implied. By utilizing the design recommendations within this report, the addressee acknowledges and accepts the risks and limitations of development at the site, as outlined within the report.

Respectfully Submitted;
GeoForensics, Inc.

Daniel F. Dyckman, PE, GE
Senior Geotechnical Engineer, GE 2145

cc: 5 to addressee

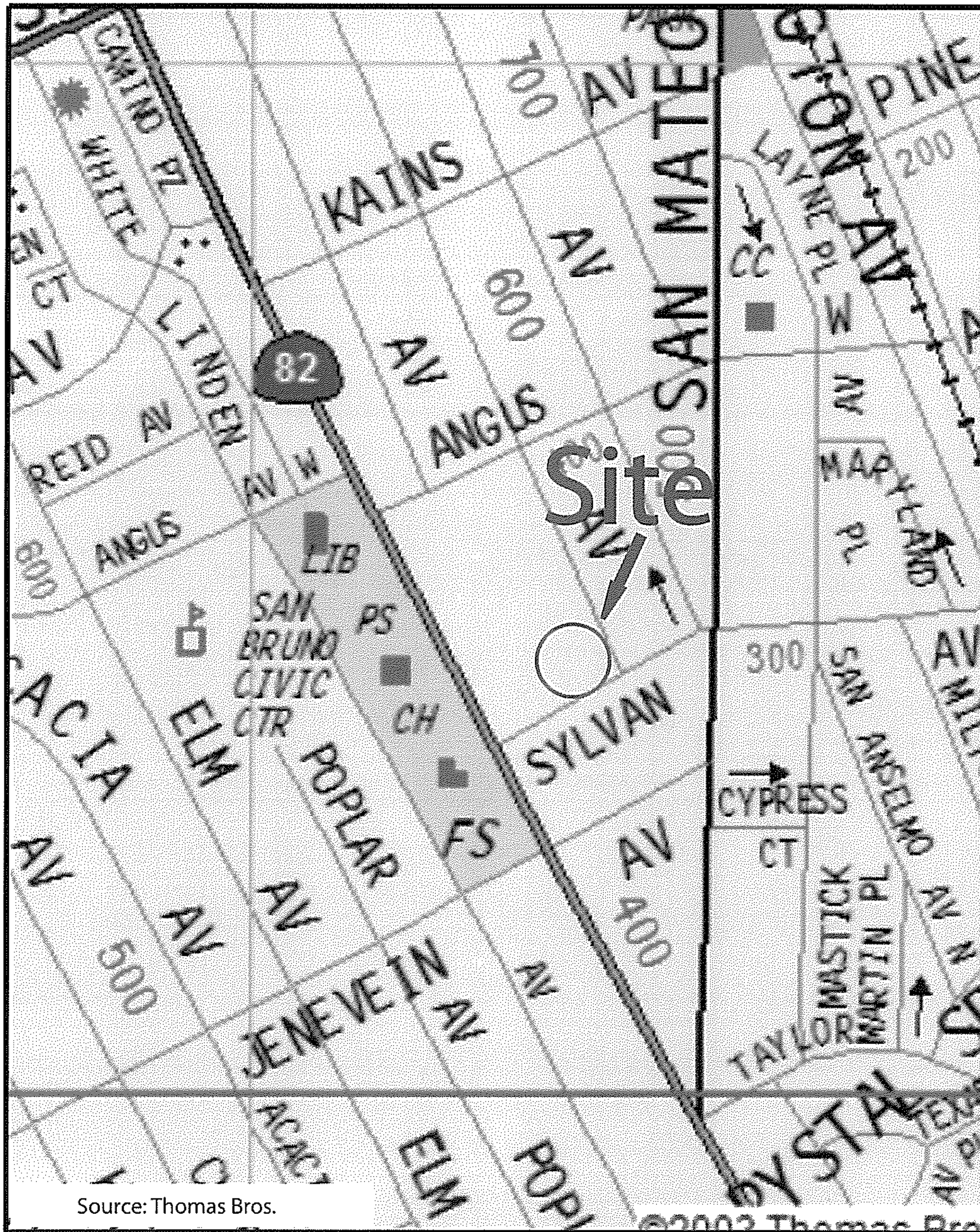

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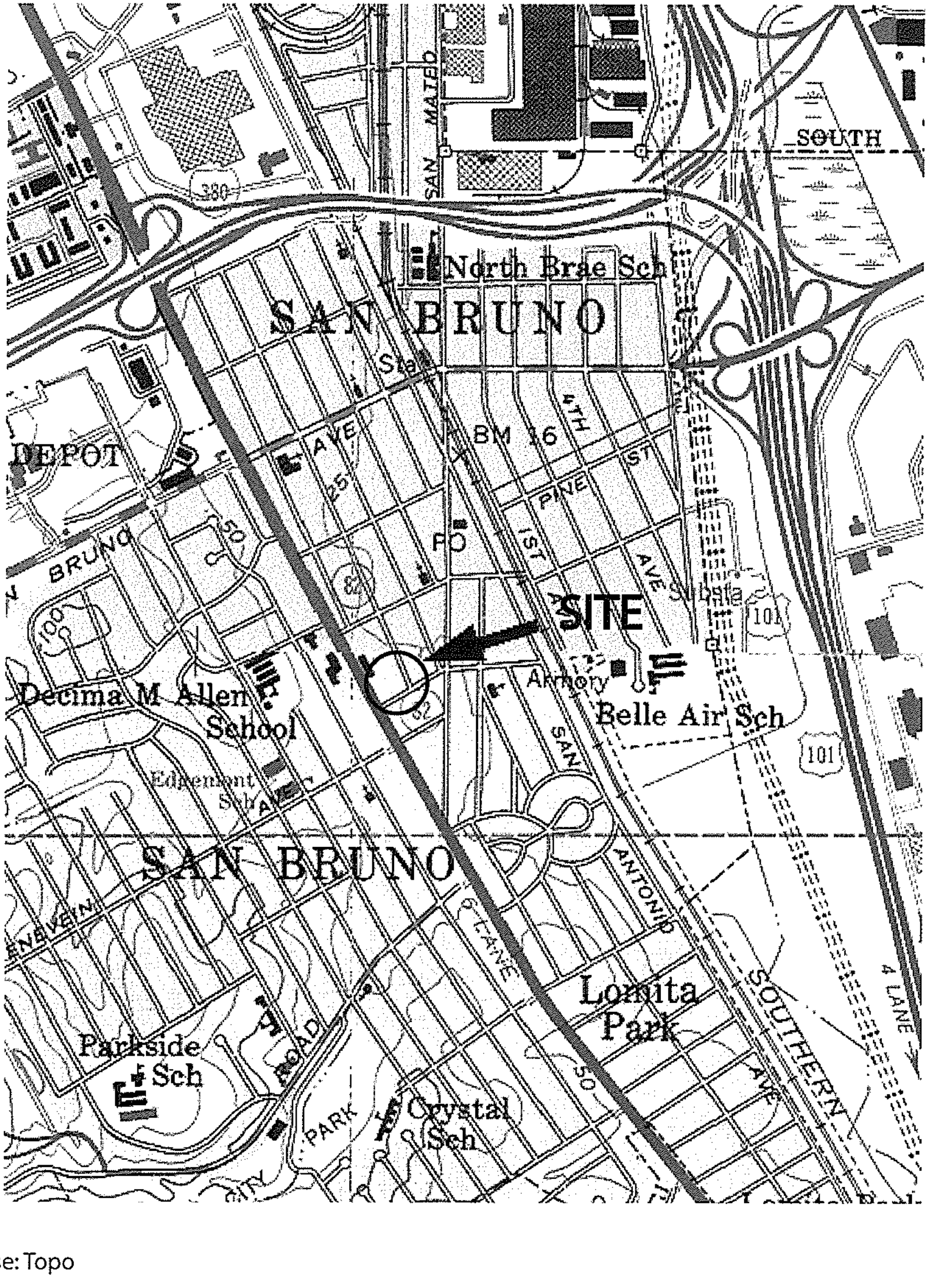
Figure 1 - Site Location



Base: Topo

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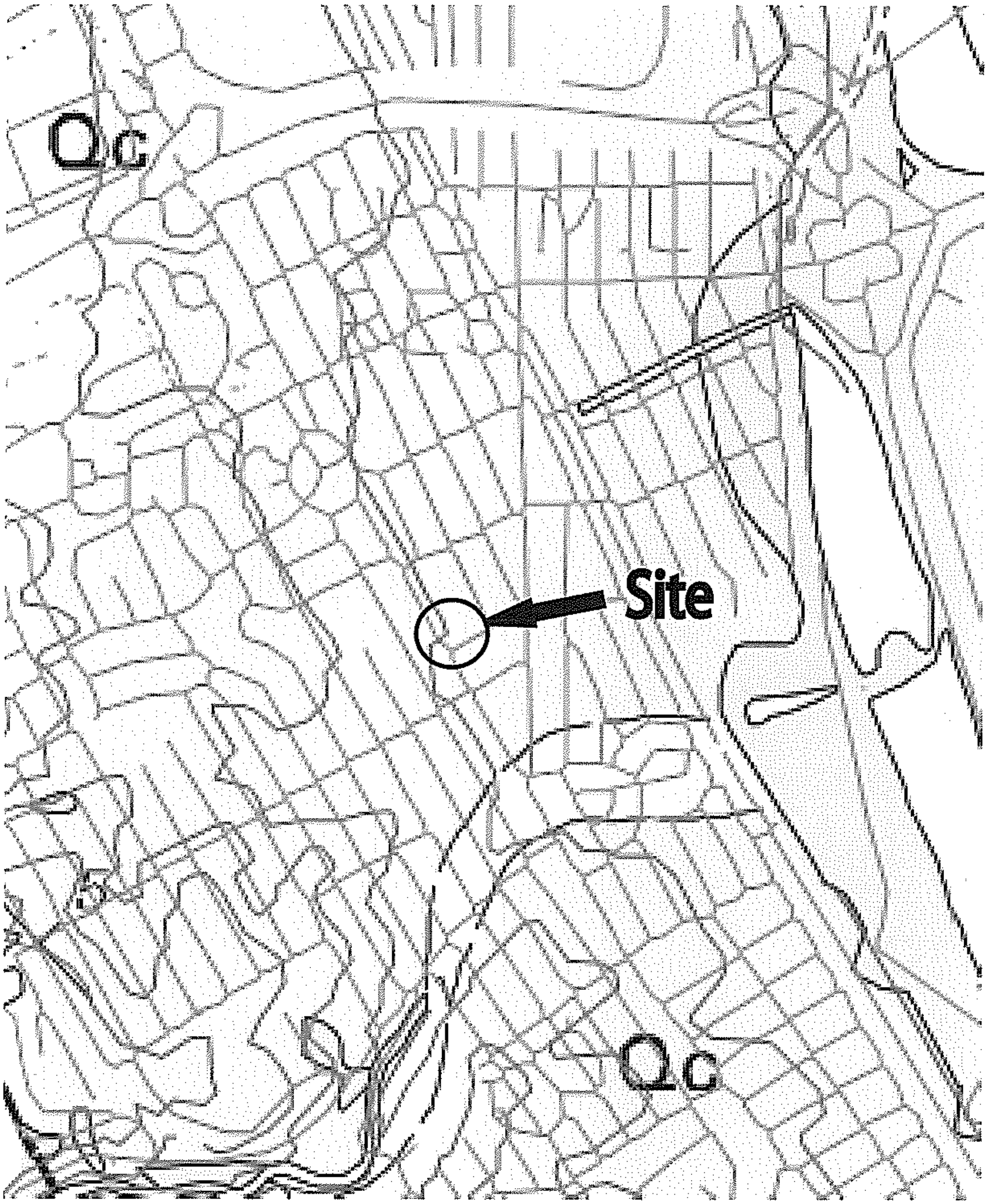
Figure 2 - Regional Topography



Qc	**Colma Formation (Pleistocene)**—Yellowish-gray and gray to yellowish-orange and red-brown, friable to loose, fine–to medium-grained arkosic sand with subordinate amounts of gravel, silt, and clay. Total thickness unknown, but may be as great as 60 m

Base: Geologiy of the OnShore Part of San Mateo County, Califronia

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Figure 3 - Gelogic Map

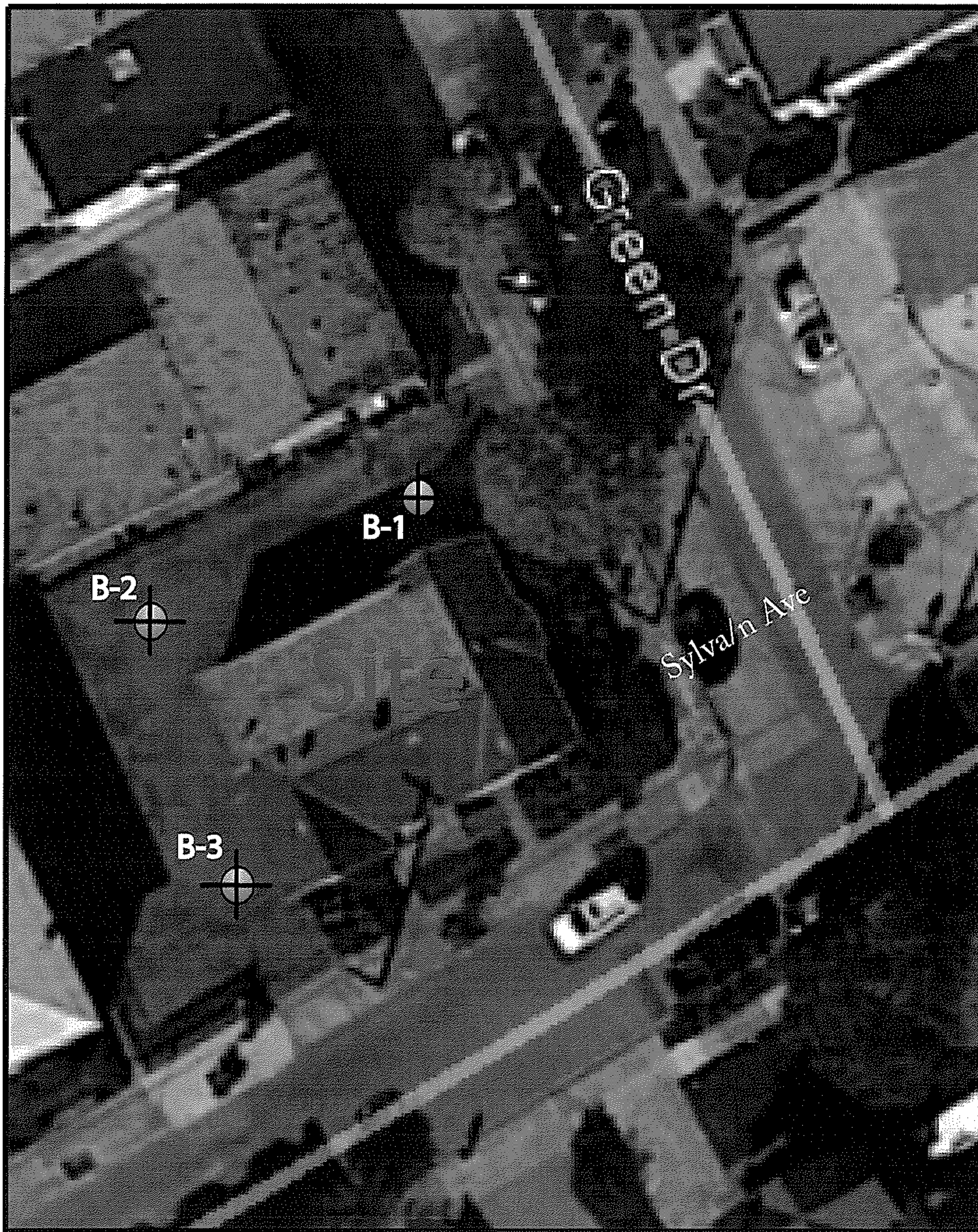

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Figure 4 - Site Photo

APPENDIX A - BORING LOGS

LOG OF BORING

DEPTH (ft)	SAMPLE NUMBER	SAMPLE LOC.	BLOW COUNTS (12 inches)	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (70)
				2"AC over 3" AB Clayey SILT - dark brown, soft, moist (ML)		
5	1-1	Mod. Cal	6		97.2	22.6
				Clayey SILT - Brown, moist, stiff (ML) as above but tan		
10	1-2	Mod. Cal	18		105.5	22.0
				Sandy CLAY - orange brown and brown, moist to v. moist, stiff (CL)		
				Gravelly SIlty SAND - brown, wet, dense (SM)		
15	1-3	Mod. Cal	54		120.7	13.3
				Sandy CLAY - Green Grey, wet, hard		
20	1-4	SPT	48			
25	1-5	SPT	17	as above, but tan and very stiff (CL)		
30				Bottom of Boring @ 25 ft Groundwater @ 12 feet		

Logged by: DD
Job No: 217179
Drilled on 7/5/17

Moble B-24 Drilling Rig
140 Pound Hammer

Mod. Cal Sampler
SPT Sampler

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Figure A1 - Log of Boring 1

LOG OF BORING

DEPTH (ft)	SAMPLE NUMBER	SAMPLE LOC.	BLOW COUNTS (12 inches)	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (70)
				1"AC over 7"AB Clayey SILT - dark Brown, moist, soft to firm		
5	2-1	Mod. Cal Sampler	7	(ML)	99.2	20.4
10	2-2	Mod. Cal Sampler	21	Clayey SILT - brown to tan, moist, stiff (ML)	105.7	21.8
15	2-3	SPT Sampler	13	Silty SAND - grey, wet, med dense (SM)		
20	2-4	SPT Sampler	10	Clayey SAND - grey, wet, med dense (SC)		
25	2-5	SPT Sampler	30	Silty SAND - brown, v. moist, dense (SM)		
30				Bottom of Boring @ 24.5 ft Groundwater @ 8 feet		

Logged by: DD
Job No: 217179
Drilled on 7/5/17

Moble B-24 Drilling Rig
140 Pound Hammer

Mod. Cal Sampler
SPT Sampler

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Figure A2 - Log of Boring 2

LOG OF BORING

DEPTH (ft)	SAMPLE NUMBER	SAMPLE LOC.	BLOW COUNTS (12 inches)	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (70)
				2"AC over 6" AB Silty SAND - brown, damp, med. dense		
	3-1	Mod. Cal Sampler	11		106.8	16.2
5						
10						
	3-2	Mod. Cal Sampler	7	as above but dark grey, wet, loose	119.2	13.7
				(ML)		
15				Clayey SAND - green grey, wet, med. dense		
	3-3	SPT Sampler	9	(SC)		
20				Bottom of Boring @ 20 ft Groundwater @ 8 feet		
25						
30						

Logged by: DD
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Moble B-24 Drilling Rig
140 Pound Hammer

Mod. Cal Sampler
SPT Sampler

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Figure A3 - Log of Boring 3

APPENDIX B - LABORATORY TEST RESULTS

Moisture-Density-Porosity Report

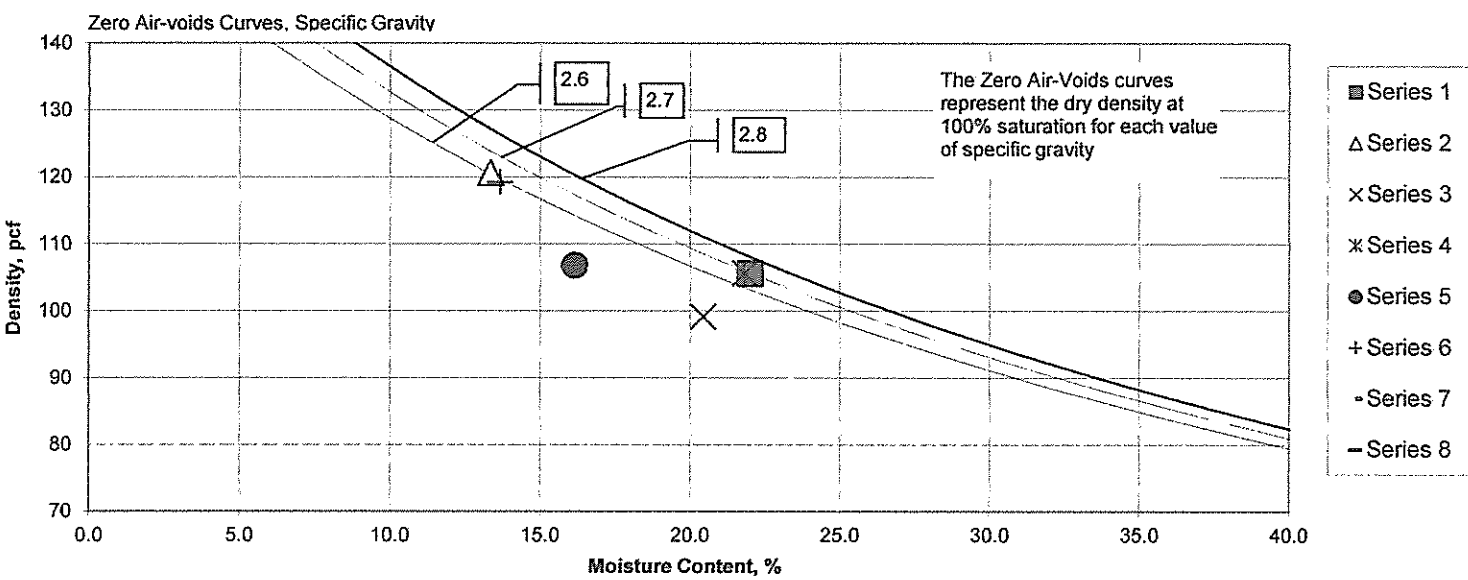
Cooper Testing Labs, Inc. (ASTM D7263b)

CTL Job No:	060-2557	Project No.	217179	By:	RU
Client:	GeoForensics	Date:	07/20/17		
Project Name:	Sylvan	Remarks:			

Boring:	1-2	1-3	2-1	2-2	3-1	3-2		
Sample:								
Depth, ft:	4	14	3	10.5	3.5	8.5		
Visual Description:	Olive Brown Mottled Reddish Brown Sandy CLAY	Olive Brown Clayey SAND	Black CLAY w/ Sand	Dark Olive Brown CLAY	Olive Brown Sandy CLAY	Olive Brown SAND w/ Silt & Gravel		
Actual G_s								
Assumed G_s	2.70	2.70	2.70	2.70	2.70	2.70		
Moisture, %	22.0	13.3	20.4	21.8	16.2	13.7		
Wet Unit wt, pcf	128.8	136.8	119.4	128.7	124.1	135.5		
Dry Unit wt, pcf	105.5	120.7	99.2	105.7	106.8	119.2		
Dry Bulk Dens.ρb, (g/cc)	1.69	1.93	1.59	1.69	1.71	1.91		
Saturation, %	99.3	90.5	78.7	98.8	75.3	88.9		
Total Porosity, %	37.4	28.5	41.2	37.4	36.7	29.3		
Volumetric Water Cont,Θw,%	37.2	25.8	32.4	36.9	27.6	26.1		
Volumetric Air Cont., Θa,%	0.3	2.7	8.8	0.4	9.1	3.3		
Void Ratio	0.60	0.40	0.70	0.60	0.58	0.41		
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

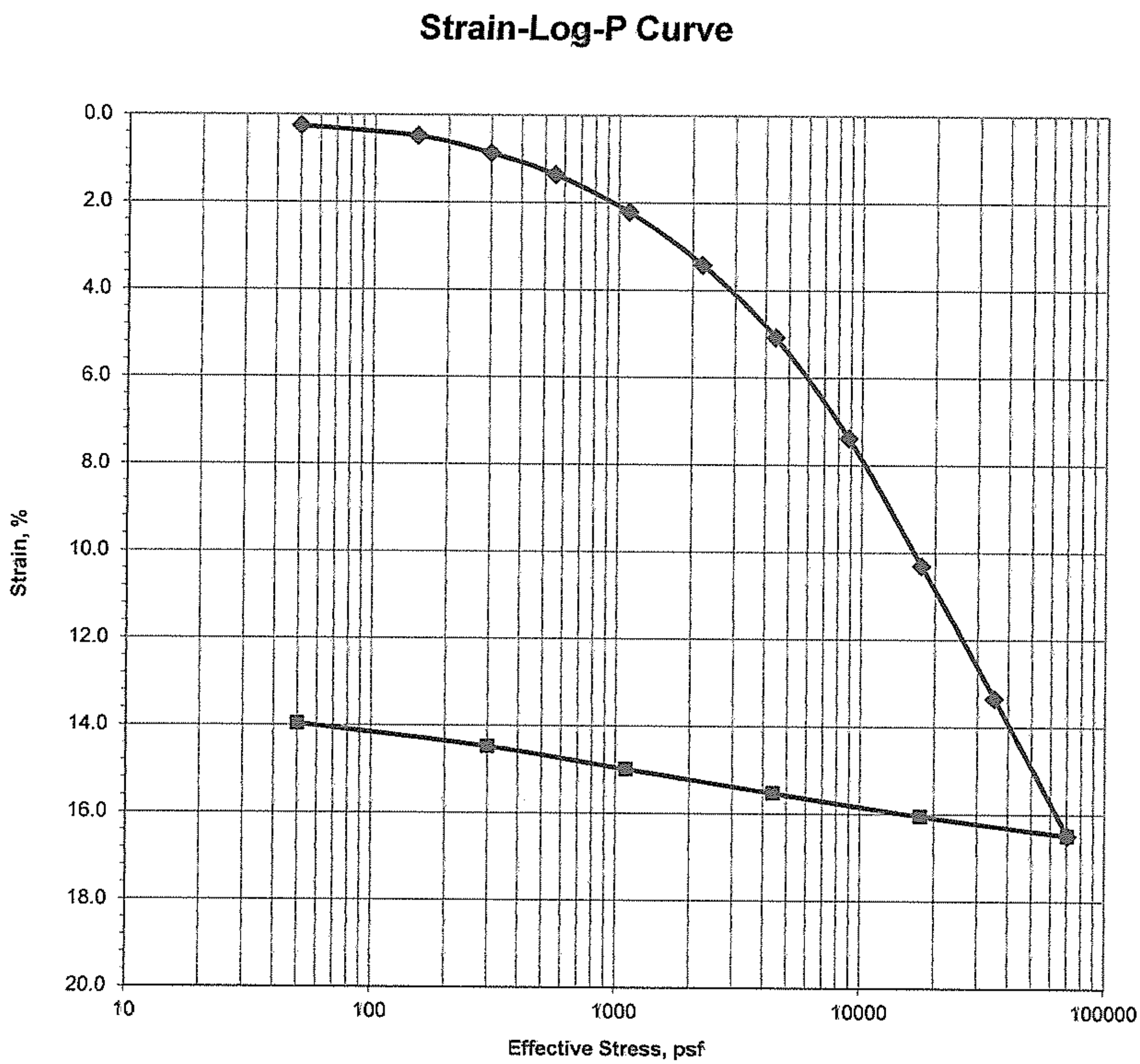
Moisture-Density



Consolidation Test
ASTM D2435

COOPER TESTING LABORATORY

Job No.:	060-2557	**Boring:**	1-1	**Run By:**	MD
Client:	GeoForensics	**Sample:**		**Reduced:**	PJ
Project:	Sylvan - 217179	**Depth, ft.:**	4	**Checked:**	PJ/DC
Soil Type:	Black Clayey SAND			**Date:**	8/3/2017



Assumed Gs 2.65	Initial	Final
Moisture %:	22.6	16.9
Dry Density, pcf:	97.2	114.3
Void Ratio:	0.702	0.448
% Saturation:	85.4	100.0

Remarks: