Steven Raas & Associates, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

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98147-SZ67-F53
October 29, 1998

Santa Cruz Fire Department
711 Center Street
Santa Cruz, CA 95060

APPROVED
CITY OF SANTA CRUZ
These plans and details have been reviewed for compliance with the 2022 California building codes. The approval of these plans shall not be construed to be a permit for any violation of any code or ordinance. These plans shall be on the job for all requested inspections

Attention: Jerry Ochoa

Subject: Geotechnical Investigation
335 Younglove
Santa Cruz, California

Dear Mr. Ochoa,

In accordance with your authorization, we have performed a geotechnical investigation for the addition to the fire station located on Younglove Avenue in Santa Cruz, California.

The accompanying report presents our conclusions and recommendations as well as the results of the geotechnical investigation on which they are based. If you have any questions concerning the data, conclusions or recommendations presented in this report, please call our office.

Very truly yours,

STEVEN RAAS & ASSOCIATES, INC.

REGISTERED PROFESSIONAL ENGINEER
STEVEN M. RAAS
No. 2039
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GEOTECHNICAL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
DALETH FOSTER
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Steven M. Raas
Principal Engineer
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Copies: 3

GEOTECHNICAL INVESTIGATION

PURPOSE AND SCOPE

This report describes the geotechnical investigation and presents results, including recommendations, for the proposed addition of a drive-through bay to the fire station. Our scope of services for this project has consisted of:

1. Discussions with you.
2. Review of the pertinent published material concerning the site including County planning maps, geologic and topographic maps, and other available literature.
3. The drilling and logging of 2 test borings.
4. Laboratory analysis of retrieved soil samples.
5. Engineering analysis of the field and laboratory results.
6. Preparation of this report documenting our investigation and presenting recommendations for the design of the project.

LOCATION AND DESCRIPTION

The site occupies the lot bounded by Younglove and Almar Avenues, approximately 200 feet south of Mission Street, on the west side of the City of Santa Cruz. The site is flat and houses a fire station constructed in the mid 1950's. The fire station currently has two engine bays that are accessed from Younglove Avenue. It is proposed to restrict Younglove Avenue to one-way southbound traffic as part of the proposed Caltrans Mission Street widening project. The fire department proposes to construct one additional drive-through bay (enter Younglove Avenue, exit Almar Avenue) to maintain quick access to Mission Street. The area of the proposed addition is currently landscaped with sod.

SOIL CONDITIONS

The site is located on the first emergent marine terrace at an elevation of approximately 60 feet above sea level. The site is mapped as Marine Terrace Deposits overlying Santa Cruz Mudstone. The soil encountered on the site consisted of approximately 3 feet of gray sandy clay overlying approximately 1 foot hardpan comprised of cemented silty sand. Below a depth of approximately 4 feet yellowish brown sand and silty sand was encountered to the depth of bedrock. The bedrock, encountered at a depth of approximately 8 feet in Boring No. 1 and 9 ½ feet in Boring No. 2, is composed of very dark brown sandy siltstone. Both borings had to be abandoned at a depth of approximately 11 feet below the ground surface due to auger refusal on highly indurated mudstone.

The soils were moist to a depth of approximately 4 feet below the ground surface, and became wet below this depth. Free groundwater was encountered at a depth of approximately 6 feet below the ground surface.

FIELD INVESTIGATION

Two 6 inch diameter test borings were drilled on the site on October 17, 1998. The drilling method used was hydraulically operated continuous flight augers. A geologist from Steven Raas & Associates, Inc. was present during all field operations to log the soil encountered, to choose soil sampling types and locations, and to supervise the field operations.

Relatively undisturbed soil samples were obtained from test borings by driving a split spoon sampler 18 inches into the ground. We selected samplers with 3 inch, 2½ inch and 2 inch outside diameters and have designated these respectively as "L", "M" and "T" on the boring logs. The samplers were driven using a 140 pound down hole safety hammer released with a 30 inch vertical free fall. The number of hammer blows was recorded in the field for each 6 inch drive. The total blow count for the bottom 12 inches is reported as the standard penetration test value (SPT "N") on the boring logs. All standard penetration test data has been normalized to a 2 inch outside diameter sampler to approximate the SPT "N" value.

Appendix A contains the Site Plan showing the locations of the test borings, and the test boring logs presenting the soil profiles, sample locations, SPT "N" values and laboratory test data for each sample. Contacts between soil types on the logs are approximate, as the actual transition may be gradual.

LABORATORY INVESTIGATION

The laboratory testing program was developed to help evaluate the bearing capacity, settlement characteristics, swell potential, and liquefaction potential for the soil on the site. Laboratory tests performed include:

a) Moisture Density relationships in accordance with ASTM test D2937.

b) Unconfined Compression tests in accordance with ASTM test D2166.

c) Atterberg Limits tests in accordance with ASTM test D4318.

d) Gradation tests in accordance with ASTM test D422.

The results of the laboratory tests are presented on the boring logs opposite the sample tested.

SEISMIC HAZARDS

The seismic setting of the site is one in which it is reasonable to assume that the site will experience significant seismic shaking during the lifetime of the project. Active or potentially active faults which may significantly affect the site include the San Andreas (12 miles to the northeast), the Zayante (9miles to the northeast), and the San Gregorio (10 miles to the southwest).

Seismic hazards which may affect areas of Santa Cruz County are surface ground rupture, seismic shaking, liquefaction and lateral spreading, and seismically induced slope instabilities.

Surface ground rupture occurs in active fault zones. Since the nearest known active or potentially active fault is located nine mile from the site, the potential for surface ground rupture at this site is considered to be low.

Ground shaking will be felt on the site. Structures founded on rock will experience shaking with lower amplitude and higher frequency than structures founded on thick soft soils. Generally, shaking will be more intense closer to earthquake epicenters. Structures built in accordance with the latest edition of the Uniform Building Code for Seismic Zone 4 should experience damage that is repairable. The seismic design of the project should be based on the 1997 Uniform Building Code as it has incorporated the most recent seismic design parameters. The following values for the seismic design of the project site were derived or taken from the 1997 UBC.

1997 UBC Seismic Design Parameters

Seismic Zone	Zone 4
Seismic Zone Factor	$Z = 0.4$
Soil Profile Type	Dense Soil / Soft Rock (S_C)
Near Source Factor N_a	$N_a = 1.0$
Seismic coefficient C_a	$C_a = 0.40$
Near Source Factor N_v	$N_v = 1.0$
Seismic coefficient C_v	$C_v = 0.56$

The site is mapped in area with a low potential for liquefaction (Maps showing Geology and Liquefaction Potential of Northern Monterey and Southern Santa Cruz Counties, California, Dupre and Tinsley, 1980). Liquefaction tends to occur in soils composed of loose sands and non-cohesive silts of restricted permeability. In order for liquefaction to occur the soil must be granular and saturated, and there must be cyclic accelerations of sufficient magnitude to progressively increase the water pressures within the soil mass. Shear strength in granular soil is developed by point to point contact between soil grains. As water pressures increase in void spaces surrounding the soil grains the particles become supported more by water than by point to point contact. When water pressures increase sufficiently, soil grains begin to lose contact with each other resulting in loss of shear strength and continuous soil deformation where the soil appears to liquefy.

An approximately 5 foot thick saturated sand layer underlies the building site. The lower portion of this sand layer could liquefy in the event of a large magnitude earthquake on nearby fault system. We analyzed the potential for the ground surface settlement due to liquefaction. Our settlement potential analysis, which was based on the work of Ishihara, was performed for finished grade elevations using high repeatable accelerations of 0.4 g to 0.5 g. Liquefaction induced differential settlement is estimated to be ¼ to ½ inch over a 10 foot span. Our liquefaction and liquefaction induce settlement analysis is included in Appendix A of this report.

Lateral spreading occurs when a liquefied soil mass fails toward a free slope face, or fails on an inclined slope. The potential for lateral spreading to occur is low as the site is essentially level.

The potential for slope failures to occur on the site is low as the site is essentially level.

DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

GENERAL

1. The results of our investigation indicate that from a geotechnical engineering standpoint the property may be developed as proposed provided these recommendations are included in the design and construction of the project. The main geotechnical issues that will need to be addressed during the design and construction of the proposed drive-through bay are as follows:

   a. Mitigation of the effects of expansive clay soils that underlie the site.

   b. Mitigation of the effects of moderately liquefiable sands that underlie the site.

   c. The design should include a structural break between the existing facility and the proposed addition to accommodate differential movement resulting from soil expansion or liquefaction.

2. Grading and foundation plans should be reviewed by Steven Raas & Associates, Inc. during their preparation and prior to contract bidding.

3. Steven Raas & Associates, Inc. should be notified **at least four (4) working days** prior to any site clearing and grading operations on the property in order to observe the stripping and disposal of unsuitable materials, and to coordinate this work with the grading contractor. During this period, a pre-construction conference should be held on the site, with at least the grading contractor, a city representative and one of our engineers present. At this time, the project specifications and the testing and inspection responsibilities will be outlined and discussed.

4. Field observation and testing must be provided by a representative of Steven Raas & Associates, Inc., to enable them to form an opinion regarding the degree of conformance of the exposed site conditions to those foreseen in this report, the adequacy of the site preparation, the acceptability of fill materials, and the extent to which the earthwork construction and the degree of compaction comply with the specification requirements. Any work related to grading performed without the full knowledge of, and not under the direct observation of Steven Raas & Associates, Inc., the Geotechnical Engineer, will render the recommendations of this report invalid.

SITE PREPARATION

5. The initial preparation of the site will consist of the removal of bushes and pavements. This material must be removed from the site. Following the removal of existing improvements, the area to be graded should be stripped of sod and organic material. The required depth of stripping will vary with the time of year and must be based upon visual observations of a representative of Steven Raas & Associates, Inc. It is anticipated that the depth of stripping may be 2 to 4 inches.

6. Following the stripping and backfilling of voids, the expansive clay soils in the building areas should be removed to a minimum depth of 48 inches below existing grade or as designated by a representative of Steven Raas & Associates, Inc. This material should be removed from the site. The soil at the excavation base should be scarified, moisture conditioned and compacted. The moisture conditioning procedure will depend upon the time of year that the work is done, but it should result in the soils being 1 to 3 percent over their optimum moisture contents at the time of compaction. The excavation should be backfilled with properly compacted non-expansive engineered fill. There should be a minimum of 30 inches of engineered fill under all foundation elements. The excavation and recompaction in the driveway and parking areas should extend to a minimum depth of 24 inches below the original ground surface and should result in a minimum of 18 inches of recompacted material below pavement sections. Recompacted sections should extend 5 feet beyond all building and pavement areas.

Note: If this work is done during or soon after the rainy season, on-site or imported soils may be too wet to be used as engineered fill without significant and effective moisture conditioning.

7. The soil on the site should be compacted as follows: 1) In pavement areas the upper 8 inches of subgrade, and all aggregate subbase and aggregate base, should be compacted to a minimum of 95% of its maximum dry density, 2) In pavement areas all utility trench backfill should be compacted to 95% of it maximum dry density, 3) The remaining soil on the project should be compacted to a minimum of 90% of its maximum dry density.

8. The maximum dry density will be obtained from a laboratory compaction curve run in accordance with ASTM Procedure #D1557. This test will also establish the optimum moisture content of the material. Field density testing will be in accordance with ASTM Test #D2922.

9. Should the use of imported fill be necessary on this project, the fill material should be:

a. free of organics, debris, and other deleterious materials
b. granular in nature, well graded, and contain sufficient binder to allow utility trenches to stand open

c. free of rocks in excess of 2 inches in size
d. have a Plasticity Index between 4 and 12
e. have a minimum Sand Equivalent of 20, and
f. have a minimum Resistance "R" Value of 30, and be non-expansive

10. Samples of any proposed imported fill planned for use on this project should be submitted to Steven Raas & Associates, Inc. for appropriate testing and approval not less than 4 working days before the anticipated jobsite delivery.

FOUNDATIONS - SPREAD FOOTINGS

11. At the time we prepared this report, the plans had not been completed and the structure location and foundation details had not been finalized. We request an opportunity to review these items during the design stages to determine if supplemental recommendations will be required.

12. Considering the soil characteristics and site preparation recommendations, it is our opinion that an appropriate foundation system to support the proposed structures will consist of reinforced concrete spread footings bedded into properly compacted non-expansive engineered fill.

13. The addition should be constructed such that it is not structurally integrated with the existing structure to accommodate differential movement associated with soil expansion or settlement that could occur at the site. The foundation should be constructed with interconnected footing designed to accommodate up to ½ inch of settlement over 10 feet.

14. Footing widths should be based on allowable bearing values with minimum requirements as indicated in the table below. Footing excavations must be observed by a representative of Steven Raas & Associates, Inc. before steel is placed and concrete is poured to insure bedding into proper material. The footing excavations should be thoroughly saturated prior to placing concrete.

Minimum Footing Dimensions

Structure Type	Footing Width	Footing Depth
1 Story Structure	12 inches	12 inches
2 Story Structure	15 inches	18 inches

15. Footings constructed to the given criteria may be designed for the following allowable bearing capacities:

a. 2,000 psf for Dead plus Live Load
b. a 1/3rd increase for Seismic or Wind Load

In computing the pressures transmitted to the soil by the footings, the embedded weight of the footing may be neglected.

16. The footings should contain steel reinforcement as determined by the Project Structural Engineer in accordance with applicable UBC or ACI Standards.

SLAB-ON-GRADE FLOOR CONSTRUCTION

17. Concrete slab-on-grade floors may be used for ground level construction on non-expansive engineered fill soils.

18. Slab-on-grade floors should be designed to span differential settlement of ½ inch in 10 feet. Slabs may be structurally integrated with the footings. If the slabs are constructed as "free floating" slabs, they should be provided with a minimum ¼ inch felt separation between the slab and footing. The slabs should be separated into approximately 15' x 15' square sections with dummy joints or similar type crack control devices.

19. All concrete slabs-on-grade should be underlain by a minimum 4 inch thick capillary break of ¾ inch clean crushed rock. It is recommended that neither Class II baserock nor sand be employed as the capillary break material.

20. Where floor coverings are anticipated or vapor transmission may be a problem, a waterproof membrane should be placed between the granular layer and the floor slab in order to reduce moisture condensation under the floor coverings. A 2 inch layer of moist sand on top of the membrane will help protect the membrane and will assist in equalizing the curing rate of the concrete.

21. Requirements for pre-wetting of the subgrade soils prior to the pouring of the slabs will depend on the specific soils and seasonal moisture conditions and will be determined by a representative of Steven Raas & Associates, Inc. at the time of construction. It is important that the subgrade soils be thoroughly saturated at the time the concrete is poured.

22. Slab thickness, reinforcement, and doweling should be determined by the Project Structural Engineer.

UTILITY TRENCHES

23. Utility trenches that are parallel to the sides of the building should be placed so that they do not extend below a line sloping down and away at a 2:1 (horizontal to vertical) slope from the bottom outside edge of all footings.

24. Trenches may be backfilled with the native materials or approved import granular material with the soil compacted in thin lifts to a minimum of 95% of its maximum dry density in paved areas and 90% in other areas.

25. Jetting of the trench backfill should be carefully considered as it may result in an unsatisfactory degree of compaction.

26. Trenches must be shored as required by the local agency and the State of California Division of Industrial Safety construction safety orders.

SURFACE DRAINAGE

27. Surface water must not be allowed to pond or be trapped adjacent to the building foundations nor on the building pad nor in the parking areas.

28. All roof eaves should be guttered, with the outlets from the downspouts provided with adequate capacity to carry the storm water from the structures to reduce the possibility of soil saturation and erosion. The connection should be in a closed conduit which discharges at an approved location away from the structures and the graded area.

29. Final grades should be provided with a positive gradient away from all foundations in order to provide for rapid removal of the surface water from the foundations to an adequate discharge point. Concentrations of surface water runoff should be handled by providing necessary structures, such as paved ditches, catch basins, etc.

30. Irrigation activities at the site should not be done in an uncontrolled or unreasonable manner.

31. The building and surface drainage facilities must not be altered nor any filling or excavation work performed in the area without first consulting Steven Raas & Associates, Inc.

PAVEMENT DESIGN

32. The soils that will comprise the upper 18 inches of pavement subgrade will be comprised of non-expansive imported fill. The imported material for the pavement section should have a minimum "R" Value or 50. We will use an "R" Value of 50 for design of the pavement sections noted below. This must be verified in the field and, if necessary, modifications made to these tentative sections.

33. Using CALTRANS Design Procedure and a 20 year design life, the following pavement sections are suggested:

Traffic Type	**Truck Use Areas**
Traffic Index (TI)	6 ½
Asphaltic Concrete	2 ½ inches
Class 2 Aggregate Base (Minimum R=78)	8 inches

34. To have the selected pavement sections perform to their greatest efficiency, it is very important that the following items be considered:

a. Properly moisture condition the subgrade and compact it to a minimum of 95% of its maximum dry density, at a moisture content 1-3% over the optimum moisture content.

b. Provide sufficient gradient to prevent ponding of water.

c. Use only quality materials of the type and thickness (minimum) specified. All baserock must meet CALTRANS Standard Specifications for Class 2 Aggregate Base, and be angular in shape.

d. Compact the base and subbase uniformly to a minimum of 95% of its maximum dry density.

e. Place the asphaltic concrete only during periods of fair weather when the free air temperature is within prescribed limits.

f. Maintenance should be undertaken on a routine basis.

PLAN REVIEW

35. We respectfully request an opportunity to review the plans during preparation and before bidding to insure that the recommendations of this report have been included and to provide additional recommendations, if needed.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that planned at the time, our firm should be notified so that supplemental recommendations can be given.

2. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to insure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to insure that the Contractors and Subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural process or the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside of our control. This report should therefore be reviewed in light of future planned construction and then current applicable codes.

4. This report was prepared upon your request for our services in accordance with currently accepted standards of professional geotechnical engineering practice. No warranty as to the contents of this report is intended, and none shall be inferred from the statements or opinions expressed.

5. The scope of our services mutually agreed upon for this project did not include any environmental assessment or study for the presence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around this site.

Important Information About Your
Geotechnical Engineering Report

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

The following information is provided to help you manage your risks.

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

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

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

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

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

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

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

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

Subsurface Conditions Can Change

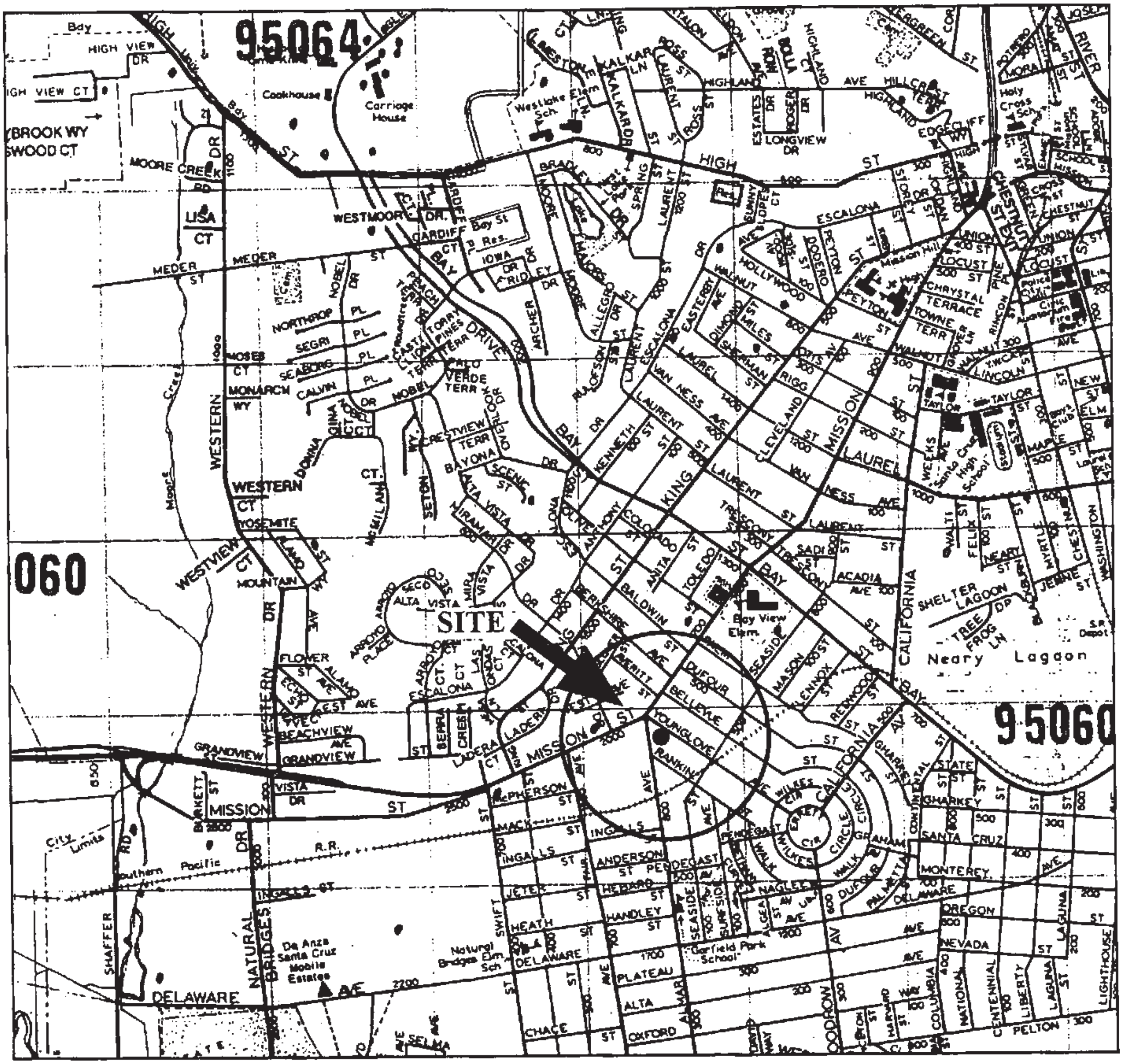
A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural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

Most Geotechnical Findings Are Professional Opinions

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

APPENDIX A

Regional Site Plan
Site Plan Showing Test Borings
Boring Log Explanation
Log of Test Borings
Atterberg Limits



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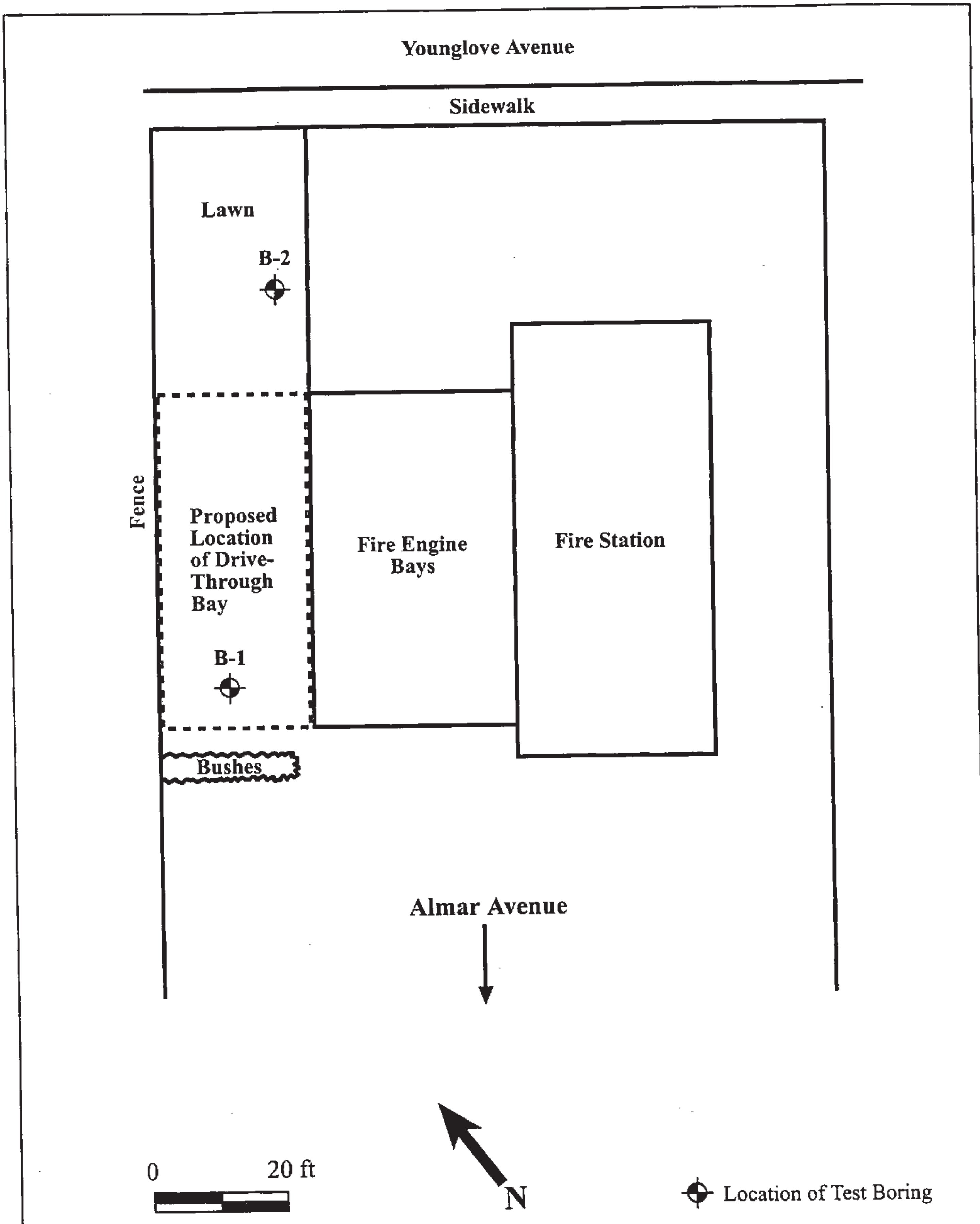
Base Map from Regal Map Company

Steven Raas & Associates, Inc.	Figure No. 1	Regional Site Plan



Steven Raas & Associates, Inc. | Figure No. 2 Site Plan Showing Test Borings

UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2488-84

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN #200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN #4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel sand mixture, little or no fines.
			GP	Poorly graded gravels or gravels-sand mixtures, little or no fines.
		GRAVELS (MORE THAN 12% FINES)	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN #4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS (MORE THAN 12% FINES)	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN #200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine clayey sand silty sands, with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly, sandy, silty or lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Organic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils

BORING LOG EXPLANATION

LOGGED BY______ DATE DRILLED___________ BORING DIAMETER______ BORING NO. ____

Depth, ft.	Sample No. and Type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density, p.c.f.	Moisture % of Dry Wt.	MISC. LAB RESULTS
1			← Ground water elevation	Note: All blows/foot are normalized to 2" outside diameter sampler size					
2	1-1 L		← Soil Sample Number ← Soil Sampler Size/Type L = 3" Outside Diameter M = 2.5" Outside Diameter T = 2" Outside Diameter ST = Shelby Tube BAG = Bag Sample						
3									
4									
5									
6									

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	BLOWS/FOOT
VERY SOFT	0-2
SOFT	2-4
FIRM	4-8
STIFF	8-16
VERY STIFF	16-32
HARD	OVER 32

STEVEN RAAS & ASSOCIATES, INC. | FIGURE NO. 3 | Boring Log Explanation

LOGGED BY CS DATE DRILLED 10/7/98 BORING DIAMETER 6" SS BORING NO. 1

Depth, ft.	Sample No. and Type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density, p.c.f.	Moisture % of Dry Wt.	MISC. LAB RESULTS
			Dark brown Silty SAND/Sandy SILT, moist	SM					
1, 2, 3	1-1 M		Olive gray Sandy CLAY, fine to medium grained sand, moist, stiff	CL-CH	15	28	93.4	24.1	65% Passing #200 Sieve Qu=2,875 psf
4, 5, 6, 7	1-2 M		Yellowish brown SAND, medium to coarse grained sand, wet, medium dense Increase in density at 8'	SP	28		108.2	18.5	
8, 9, 10	1-3 M		Santa Cruz Mudstone Very dark brown SILTSTONE, damp, very slow drilling		50/3"		85.8	37.7	
11–24			Boring Terminated at 10 1/2' on Auger Refusal						

STEVEN RAAS & ASSOCIATES, INC. FIGURE NO. 4 Log of Test Borings

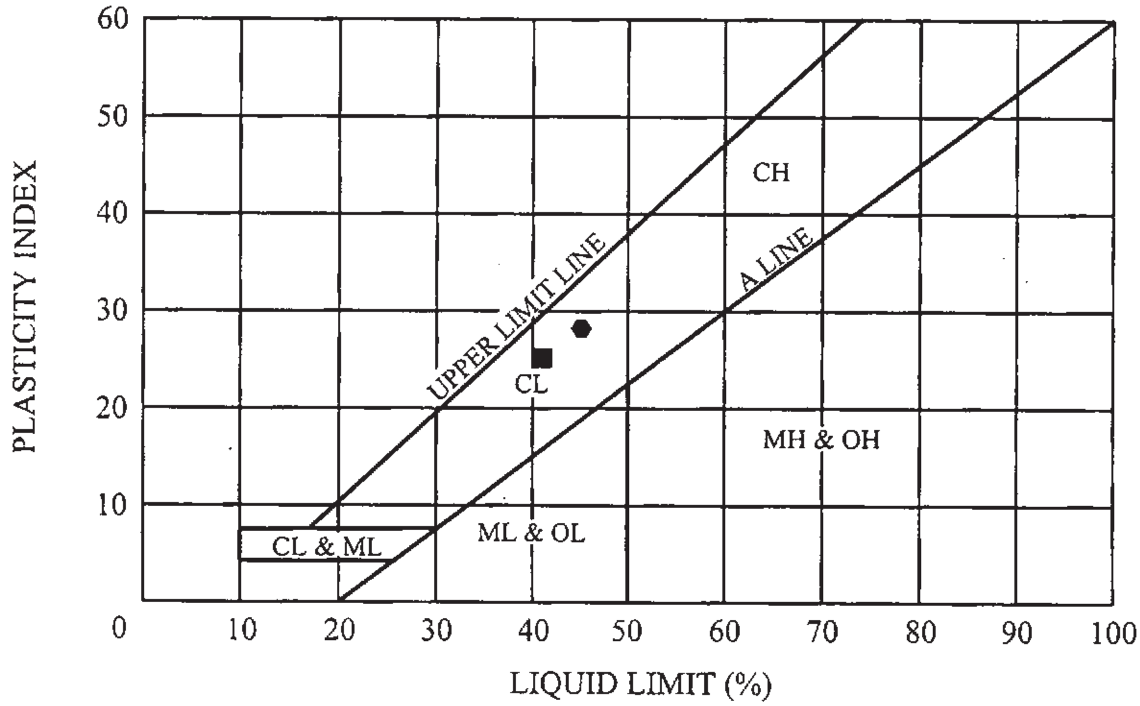
LOGGED BY CS DATE DRILLED 10/7/98 BORING DIAMETER 6" SS BORING NO. 2

Depth, ft.	Sample No. and Type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density, p.c.f.	Moisture % of Dry Wt.	MISC. LAB RESULTS
			Dark brown Silty SAND/Sandy SILT, moist	SM					
1	2-1 L		Olive gray Sandy CLAY, medium grained sand, damp, stiff	CL					
2					13	25	112.1	15.7	54% Passing #200 Sieve Qu=7,900 psf
3	2-2 M		Yellow Silty SAND, fine to medium grained sand, hardpan, damp, very dense	SM					
4	2-3 T		Yellowish brown Silty SAND, medium grained sand, wet, medium dense	SM	65		115.3	14.5	28% Passing #200 Sieve Qu=7,725 psf
5					28			17.3	18% Passing #200 Sieve
6	2-4 L		Mottled yellowish brown and orange Silty SAND, medium to coarse grained sand, wet, medium dense						
7	2-5 M		Yellowish brown SAND, very fine to very coarse grained sand with some rounded gravel to 1", saturated, medium dense	SW	21		113.1	19.0	19% Passing #200 Sieve
8					20			15.6	3% Passing #200 Sieve
9	2-6 T								
10			Santa Cruz Mudstone Brown SILTSTONE, friable, damp Very slow drilling		50/3"			39.4	51% Passing #200 Sieve
11			Boring Terminated at 11' on Auger Refusal						
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									

STEVEN RAAS & ASSOCIATES, INC. FIGURE NO. 5 Log of Test Borings

ATTERBERG LIMITS - ASTM D4318-84

PLASTICITY CHART



SYMBOL	SAMPLE #	LL (%)	PL (%)	PI
⬢	**1-1-1**	**45**	**17**	**28**
■	**2-1-1**	**41**	**16**	**25**

STEVEN RAAS & ASSOCIATES, INC.	FIGURE NO. 6	Atterberg Limits