GEOTECHNICAL ENGINEERING AND GEOLOGIC HAZARDS STUDY

Albany High School
Aquatic Center
Portland Avenue and Pomona Avenue
Albany, California

Prepared for:

Albany Unified School District
904 Talbot Avenue
Albany, California 94706

Prepared by:

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Geosphere Project No. 91-02320-PWA & PWB

December 23, 2008

Albany Unified School District
Attn.: Ms. Marla Stephenson - Superintendent
904 Talbot Avenue
Albany, California 94706

Subject: **Geotechnical Engineering and Geologic Hazards Study**
Albany High School Aquatic Center
Portland Avenue and Pomona Avenue
Albany, California
Geosphere Project No. 91-02320-PWA & PWB

Dear Ms. Stephenson:

In accordance with your authorization, Geosphere Consultants Inc. has completed a Geotechnical Engineering and Geologic Hazards Study for the proposed Aquatic Center at Albany High School. This report has been prepared in accordance with the requirements set forth in California Geological Survey Note 48 and the 2007 California Building Code. Transmitted herewith are the results of our findings, conclusions, and recommendations for the design and construction of proposed foundations, swimming pools, interior and exterior concrete slabs, site grading and drainage, temporary cut slope and trench slope stability, and utility trench backfilling. In general, the proposed improvements at the site are considered to be geotechnically and geologically feasible provided the recommendations of this report are implemented in the design and construction of the project.

Should you or members of the design team have questions or need additional information, please contact the undersigned at (925) 314-7100; mah@geosphereinc.net, grh@geosphereinc.net. The opportunity to be of service to Albany Unified School District and to be involved in the design of this project is appreciated.

Sincerely,
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GEOTECHNICAL ENGINEERING AND GEOLOGIC HAZARDS STUDY

Project: **Aquatic Center**
Albany High School

Client: **Albany Unified School District**
Albany, California

1.0 INTRODUCTION

1.1 Purpose and Scope

The purposes of this study were to evaluate the subsurface conditions at the site and prepare geotechnical recommendations for the proposed development. The site will be improved with a new Aquatic Center. This study provides recommendations for the design and construction of proposed foundations, interior and exterior concrete slabs, site grading and drainage, temporary cut slope and trench slope stability, and utility trench backfill. This study was performed in accordance with the scope of work outlined in our proposal dated October 7, 2008.

The scope of this study included the review of pertinent published and unpublished documents related to the site, field exploration, laboratory testing, engineering analysis of the accumulated data, and preparation of this report. The conclusions and recommendations presented in this report are based on the data acquired and analyzed during this study, and on prudent engineering judgment and experience. This study did not include an in-depth assessment of potentially toxic or hazardous materials that may be present on or beneath the site, however, at your request Geosphere collected analytical soil samples from the upper five feet of the site for cursory testing of contaminants. The results of this testing were issued under a separate cover in our report dated December 1, 2008.

1.2 Site Description

The site of the proposed Aquatic Center at the existing Albany High School is be located at the southern end of the campus, near the northeast corner of the intersection between Key Route Boulevard and Portland Avenue in Albany, California, as shown on Figure 1, Site Vicinity Map. The site is presently occupied by an indoor swimming facility, as shown on Figure 2, Site Plan. Surrounding the existing building is a concrete hardscape envelope. The site is relatively flat, and the local topography slopes gently to the west towards San Francisco Bay. The geographic coordinates of the site improvements are approximately 37.8958 degrees north latitude and 122.2913 degrees west longitude.

1.3 Proposed Development

The new facility will be constructed in place of the existing indoor pool facility at the south side of the campus. The new facility will encompass the entire existing footprint and extend somewhat to the north and east of the existing facility. There are extensive utilities adjacent to the proposed project as well as structures to the west. The new facility will include two pools, one indoor and one outdoor. The attached Figure 2 shows the proposed improvement layout.

1.4 Validity of Report

This report is valid for three years after publication. If construction begins after this time period, Geosphere should be contacted to confirm that the site conditions have not changed significantly. If the proposed development differs considerably from that described above, Geosphere should be notified to determine if additional recommendations are required. Additionally, if Geosphere is not involved during the geotechnical aspects of construction, this report may become wholly or in part invalid; since Geosphere's geotechnical personnel need to verify that the subsurface conditions anticipated preparing this report are similar to the subsurface conditions revealed during construction. Geosphere's involvement should include foundation and grading plan review; observation of foundation and pool excavations; grading observation and testing; testing of utility trench backfill; and subgrade preparation and baserock placement in hardscape areas.

2.0 PROCEDURES AND RESULTS

2.1 Literature Review

Pertinent geologic and geotechnical literature pertaining to the site area was reviewed. These included various USGS, CGS, and other government publications and maps, as listed in the References section.

2.2 Field Exploration

A total of six borings were drilled at the site on November 15, 2008. The borings were drilled between approximately 15 and 30 feet deep by a Mobile B-56, truck mounted drill rig. Boring 3 was terminated at approximately three feet deep as we encountered utility bedding sand in the cuttings. Underground utilities were not detected by our private underground locating subcontractor in this location, nor were utilities detected by the utility marking performed by Cruz for the Topographic Survey prepared by KPFF Consulting Engineers.

A Geosphere representative visually classified the materials encountered in the borings according to the Unified Soil Classification System as the borings were advanced. Relatively undisturbed soil samples were recovered at selected intervals using a three-inch outside diameter Modified California split spoon sampler containing six-inch long brass liners. A two-inch outside diameter Standard Penetration Test (SPT) sampler was used to obtain SPT blow counts and obtain disturbed soil samples. The samplers were driven by using a mechanical-trip, 140-pound hammer with an approximate 30-inch fall utilizing N-rods as necessary. Resistance to penetration was recorded as the number of hammer blows required to drive the sampler the final foot of an 18-inch drive. Bulk samples were obtained in the upper few feet of the borings from the auger cuttings.

The boring logs with descriptions of the various materials encountered in each boring, the penetration resistance values, and some of the laboratory test results are presented in Appendix A. The approximate locations of the borings are shown on the Site Plan, Figure 2.

2.3 Laboratory Testing

Laboratory tests were performed on selected samples to determine some of the physical and engineering properties of the subsurface soils. The results of the laboratory testing are presented on the boring logs, and are included in the appendices. The following soil tests were performed for this and the previous studies:

<u>Dry Density and Moisture Content (ASTM D2216 and D2937)</u> – In-situ density and moisture tests are conducted to determine the in-place dry density and moisture content of the subsurface materials. These properties provide information for evaluating the physical characteristics of the subsurface soil.

Particle Size Analysis (Wet and Dry Sieve) and Hydrometer (ASTM D422, D1140, and CT202) - Sieve analysis testing is conducted on selected samples to determine the soil particle size distribution. This information is useful for the evaluation of liquefaction potential and characterizing the soil type according to USCS.

Atterberg Limits (ASTM D4318 and CT204) - Liquid Limit, Plastic Limit, and Plasticity Index are useful in the classification and characterization of the engineering properties of soil, helps evaluate the expansive characteristics of the soil, and for determining the soil type according to the USCS.

Unconfined Compressive Strength –Soil (modified ASTM D2166, CT373, and CT312) - Unconfined compression testing is performed on selected cohesive samples to provide an approximation of the undrained shear strength and allowable bearing capacity of the soil.

Soil Corrosivity - Soil corrosivity testing performed by ETS is performed to determine the effects of constituents in the soil on buried steel and concrete. Water-soluble sulfate testing is required by the UBC, CBC, and IBC.

3.0 GEOLOGIC OVERVIEW

3.1 Geologic Setting

The site is located in the central portion of the northern Coast Ranges geomorphic province of California. The Coast Ranges extend from the Transverse Ranges in southern California to the Oregon border and are comprised of a northwest-trending series of mountain ranges and intervening valleys that reflect the overall structural grain of the province. The ranges consist of a variably thick veneer of Cenozoic volcanic and sedimentary deposits overlying a Mesozoic basement of sedimentary, metamorphic, and basic igneous Franciscan Formation and primarily marine sedimentary rocks of the Great Valley Sequence. East-dipping sedimentary rocks of the Coast Ranges are flanked on the east by sedimentary rocks of the Great Valley geomorphic province (Page, 1966).

More specifically, the project site is located near the base of the Berkeley Hills, which is proximal to the San Francisco Bay structural depression to the west. The site is underlain by Holocene-aged alluvium consisting of clay, sand and gravel-sized sediments derived from the nearby foothills to the east. Relatively shallow Franciscan Formation bedrock underlies the near surface alluvial deposits. The mapped geologic units in the site vicinity are shown on the Geologic Map, Figure 3.

3.2 Geologic Evolution of the Northern Coast Ranges

The subject site is located within the tectonically active and geologically complex northern Coast Ranges, which have been shaped by continuous deformation resulting from tectonic plate convergence (subduction) beginning in the Jurassic period (about 145 million years ago). Eastward thrusting of the oceanic plate beneath the continental plate resulted in the accretion of materials onto the continental plate. These accreted materials now largely comprise the Coast Ranges. The dominant tectonic structures formed during this time include generally east-dipping thrust and reverse faults.

Beginning in the Cenozoic time period (about 25 to 30 million years ago), the tectonics along the California coast changed to a transpressional regime and right-lateral strike-slip displacements as well as thrusting were superimposed on the earlier structures resulting in the formation of northwest-trending, near-vertical faults comprising the San Andreas Fault System. The northern Coast Ranges were segmented into a series of tectonic blocks separated by major faults including the San Andreas, Hayward, and Calaveras. The project site is situated between the Hayward and San Andres faults, but no active faults with Holocene movement (last 11,000 years) lie within the limits of the site. The site is not mapped within an Alquist-Priolo Earthquake Fault Zone.

4.0 HYDROLOGY

4.1 Historic Groundwater Levels

According to previous borings performed by Kleinfelder in 1996 at Albany High School, and data compiled by CGS, the historic groundwater levels in the vicinity are on the order of 10 to 20 feet deep. This is near the range of groundwater levels encountered in our exploratory borings. Based on the groundwater depth and the depth of bedrock underlying the site, groundwater encountered within at least the upper 30 feet of the site can be considered to be perched.

4.2 Current Groundwater Levels

Groundwater was encountered in Boring 1 at about 15 feet below the ground surface at the time of drilling. Groundwater was not encountered in the other borings performed for this study. Groundwater levels can vary in response to time of year, variations in seasonal rainfall, well pumping, irrigation, and alterations to site drainage.

5.0 CORROSIVITY

5.1 Laboratory Corrosion Tests

Corrosion test results of soil samples obtained from the site were evaluated based on ASTM A888 methods. Table 1, Soil Test Corrosion Evaluation, from the ASTM procedure is presented below. If the summed points are equal to 10, the soil should be considered corrosive to cast iron pipe. If sulfides are present and low or negative redox potential results are obtained, three points shall be given for this range.

TABLE 1 – ASTM Soil Test Corrosion Evaluation

Soil Characteristics	Points
Resistivity, ohm-cm, based on single probe or water-saturated soil box.	
<700	10
700-1,000	8
1,000-1,200	5
1,200-1,500	2
1,500-2,000	1
>2,000	0
PH	
0-2	5
2-4	3
4-6.5	0
6.5-7.5	0
7.5-8.5	0
>8.5	5
Redox Potential, mV	
>+100	0
+50 to +100	3.5
0 to 50	4
Negative	5
Sulfides	
Positive	3.5
Trace	2
Negative	0
Moisture	
Poor drainage, continuously wet	2
Fair drainage, generally moist	1
Good drainage, generally dry	0

The results from a bulk sample collected from the upper five-feet of the site indicate a pH of 7.83, a resistivity of 1,590 ohm-cm, an electrical conductivity of 630 micro-ohm/cm, a chloride content of 113 ppm, a redox potential of +352 mV, and 0.027 ppm soluble sulfides. These results indicate the surficial soils have a relatively low potential to be corrosive to buried ferrous pipes.

5.2 Laboratory Water-Soluble Sulfate Tests

Water-soluble sulfate can affect the concrete mix design for concrete in contact with the ground, such as shallow foundations, piles, piers, and concrete slabs. The UBC, CBC, and IBC provide the following evaluation criteria:

TABLE 2 – UBC, CBC, IBC Water Soluble Sulfate Impacts to Cement

Sulfate Exposure	Sulfate Percent by Weight or (mg/kg)	Cement Type	Max. Water Cementitious Ratio by Weight	Min. Unconfined Compressive Strength, psi
Negligible	0.00-0.10 (0-1,000)	NA	NA	NA
Moderate	0.10-0.20 (1,000-2,000)	II, IP (MS), IS (MS)	0.50	4,000
Severe	0.20-2.00 (2,000-20,000)	V	0.45	4,500
Very Severe	Over 2.00 (20,000)	V plus pozzolan	0.45	4,500

The test results for the composite sample contained 138 ppm of water-soluble sulfate. Hence, the water-soluble sulfate content in the site soil has a negligible impact on buried concrete at the site, whether surficial or at depth. However, it should be pointed out that the water-soluble sulfate concentrations can vary due to the addition of fertilizer, irrigation, and other possible development activities. An appropriate expert should be contacted if a detailed evaluation is required.

6.0 GEOLOGIC HAZARDS

6.1 Seismic Induced Hazards

Seismic hazards resulting from the effects of an earthquake generally include ground shaking, liquefaction, lateral spreading, dynamic settlement, fault ground rupture and fault creep, and tsunamis and seiches. The site is not necessarily impacted by all of these potential seismic hazards. Nonetheless, potential seismic hazards are discussed and evaluated in the following sections in relation to the planned construction.

6.1.1 Ground Shaking

The site will likely experience severe ground shaking from a major earthquake originating from the major Bay Area faults, particularly the nearby Hayward (0.9 miles from the site), Calaveras (14.5 miles from the site) or San Andreas (17.4 miles from the site) faults.

6.1.2 Liquefaction Induced Phenomena

Research and historical data indicate that soil liquefaction generally occurs in saturated, loose granular soil (primarily fine to medium-grained, clean sand deposits) during or after strong seismic ground shaking and is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to flow as a liquid. However, because of the higher intergranular pressure of the soil at greater depths, the potential for liquefaction is generally limited to the upper 40 feet of the soil. Potential hazards associated with soil liquefaction below or near a structure include loss of foundation support, lateral spreading, sand boils, and areal and differential settlement.

Lateral spreading is lateral ground movement, with some vertical component, as a result of liquefaction. The soil literally rides on top of the liquefied layer. Lateral spreading can occur on relatively flat sites with slopes less than two percent under certain circumstances. Lateral spreading can cause ground cracking and settlement.

Our site investigation encountered medium stiff to very stiff clayey soil overlying relatively shallow weathered bedrock. The granular soil that was encountered was minor in nature and possessed too high of a clay content and was too dense to liquefy. Based on the subsurface soil encountered, it is our opinion that the likelihood of the site to experience liquefaction induced settlement and lateral spreading to be low to nil.

6.1.3 Dynamic Compaction (Settlement)

Dynamic settlement is a process in which unsaturated, relatively clean sands and silts are densified by the vibratory motion of a strong seismic event. The upper, unsaturated soil zone at the site is characterized by clayey fill and native soils. On the basis of the data developed for this investigation and cognizant that the site is subject to severe ground shaking, it is our opinion that the site soils should not be significantly affected by dynamic compaction of dry sands.

6.1.4 Fault Ground Rupture and Fault Creep

The State of California adopted the Alquist-Priolo Earthquake Fault Zone Act of 1972 (Chapter 7.5, Division 2, Sections 2621 – 2630, California Public Resources Code), which regulates development near active faults for the purpose of preventing surface fault rupture hazards to structures for human occupancy. In accordance with the Alquist-Priolo Act, the California Geological Survey established boundary zones or Earthquake Fault Zones surrounding faults or fault segments judged to be sufficiently active, well-defined and mapped for some distance. Structures for human occupancy within designated Earthquake Fault Zone boundaries are not permitted unless surface fault rupture and fault creep hazards are adequately addressed in a site-specific evaluation of the development site.

The site is not currently within a designated Earthquake Fault Zone as defined by the State (Hart and Bryant, 1997). The closest Earthquake Fault Zone is associated with the northwest striking Hayward fault, located less than one mile to the northeast of the site. Since the site is not within an Earthquake Fault Zone, the potential for fault ground rupture and fault creep hazards are judged to be very low.

6.1.5 Tsunamis and Seiches

Tsunamis are long-period sea waves generated by seafloor movements from submarine earthquakes or volcanic eruptions that rapidly displace large volumes of water. Coastal communities along the Pacific Ocean are particularly susceptible to such phenomena. However, the site is not susceptible to tsunami hazards due to its inland location, approximately 10 to 15 miles from the Pacific Coast.

Earthquake-induced waves generated within enclosed bodies of water are called seiches. The nearest body of water, San Francisco Bay, is located about one mile to the west and down gradient of the site. As such, the site has a low potential of experiencing seiche hazards.

6.2 Other Hazards

Potential geologic hazards other than those caused by a seismic event generally include ground failure and subsidence, landslides, expansive and collapsible soils, flooding, and soil erosion. These are discussed and evaluated in the following sections.

6.2.1 Landsliding

The site is not mapped by CGS or USGS sources as being located within an existing landslide or potential landslide area. Based on the underlying soil and bedrock conditions, and the local topography, the site is not considered prone to potential landsliding.

6.2.2 Expansive and Collapsible Soils

Highly expansive fine-grained soils were encountered during our subsurface exploration. The boring logs and laboratory test results are contained in the appendices of the report. The results of the laboratory testing performed on representative samples of the near-surface soils (less than 10 feet deep) indicate a Plasticity Indices of 21 and 41, and Liquid Limits of 33 and 59, indicative of soil with a high shrink/swell or expansion potential. Hence, mitigation for highly expansive soil conditions will be required for this site.

The subsurface deposits encountered during the drilling program generally consisted of medium stiff to very stiff, clayey soil overlying relatively near surface weathered bedrock. Blow counts for samples collected from these borings suggest relatively dense or stiff materials at depth. Therefore, the potential for collapsible soils underlying the site is considered to be low.

6.2.3 Flooding

According to recent Federal Emergency Management Agency mapping, the site area is not located within a recognized flood hazard zone. Because of the fact that there are no creeks crossing or in the immediate vicinity of the site, we conclude that the hazard of flooding at the site is low.

6.2.4 Soil Erosion

The surface soils at portions of the site have been disturbed during previous construction activities. Present construction techniques and agency requirements have provisions to limit soil erosion and resultant siltation during construction. These measures will reduce the potential for soil erosion at the site during the various construction phases. Long-term erosion at the site will be reduced by landscaping and hardscape areas, such as parking lots and walkways, designed with appropriate surface drainage facilities.

7.0 SEISMIC SETTING

7.1 Regional Faulting and Tectonics

Regional transpression has caused uplift and folding of the bedrock units within the Coast Ranges. This structural deformation occurred during periods of tectonic activity that began in the Pliocene and continues today. The site is located in a seismically active region that has experienced periodic, large magnitude earthquakes during historic times. This seismic activity appears to be largely controlled by displacement between the Pacific and North American crustal plates, separated by the San Andreas Fault zone, located approximately 17 miles west of the site. This plate displacement produced regional strain that is concentrated along major faults of the San Andreas Fault System including the San Andreas, Hayward, and Calaveras faults in this area.

7.1.1 Calaveras Fault

The Calaveras fault trends northwesterly approximately 123 km from near Hollister to the San Ramon/Dublin area. The Calaveras fault has been divided into three segments, the Northern, Central, and Southern segments. The site is located approximately 14.5 miles (23.2 km) from the north segment of the Calaveras fault. The slip rate on the north segment of the Calaveras fault is estimated to be about six mm/year and has been assigned a moment magnitude (Mmax) of 6.8 (CGS, 2003). The Working Group on California Earthquake Probabilities (WG99) has estimated that there is an 11 percent probability of at least one magnitude 6.7 or greater earthquake before 2030 along the Calaveras fault (USGS, 2003).

7.1.2 Hayward Fault

The Hayward fault trends northwesterly approximately 88 km from the Milpitas area to San Pablo Bay. The Hayward fault has been divided into two main segments, the Northern and Southern segments. The site is located approximately 0.9 miles (1.5 km) from the north segment of the Hayward fault. The slip rate on this segment of the Hayward fault is estimated to be about nine mm/year and has been assigned a moment magnitude (Mmax) of 6.4 (CGS, 2003). The Working Group on California Earthquake Probabilities (WG99) has estimated that there is a 27 percent probability of at least one magnitude 6.7 or greater earthquake before 2030 along the Southern segment of the Hayward fault (USGS, 2003).

7.1.3 San Andreas Fault

The northwest-trending San Andreas fault runs along the western coast of California extending approximately 625 miles from the north near Point Arena to the Salton Sea area in southern California (Jennings, 1994). The fault zone has been divided into 11 segments. The site is located approximately 17.8 miles (28.7 km) to the east of the Peninsula segment. The slip rate on the Peninsula segment of the San Andreas Fault is estimated to be about 17 mm/year and has been assigned a moment magnitude (M_{max}) of 7.1 (CGS, 2003). The Working Group on California Earthquake Probabilities (WG99) has estimated that there is a 15 percent probability of at least one magnitude 6.7 or greater earthquake before 2030 along the Peninsula segment of the San Andreas Fault (USGS, 1999).

A detailed, independent seismic analysis is contained in the attached "Supplemental Site Specific Ground Response & Seismic Hazard Evaluation Report" contained in Appendix C. This document contains 2007 CBC seismic design parameters, distances of major faults to the site, response spectra, and other seismic analysis parameters.

8.0 FIELD AND LABORATORY FINDINGS

8.1 Subsurface Conditions

We have prepared a typical subsurface profile based on the soils encountered in the borings and CPTs placed for this study. It should be noted that this is a generalized profile and is subject to local lateral and vertical variations. See the boring logs contained in Appendices A and B for specific information.

0 to 5 – 7 feet	FAT CLAY (CL), dark brown to black, sandy, medium stiff to very stiff, moist. Upper portions may be disturbed soil or artificial fill.
5-7 to 11-12 feet	LEAN CLAY (CL), orange and grayish-brown, sandy, medium stiff to hard, slightly moist to moist. Residual Soil.
11-12 to 17-18 feet	CLAYEY SAND (SC), POORLY GRADED SAND (SP), various brown, gravelly, medium to coarse, slightly moist to wet, dense to very dense. Residual Soil.
17-18 to 30 feet	BEDROCK, sandstone and siltstone/claystone, Franciscan Assemblage.

8.2 Laboratory Test Results

Laboratory test results are contained in Appendix B of this report. The results of the laboratory testing are summarized below:

8.2.1 Soil Characterization

Sieve Analysis testing was performed on a sample collected from approximately 15 feet deep in Boring 1. The sample, which had been obtained with a SPT blow count of 47 (dense) was found to contain approximately 23 percent materials passing the #200 sieve. Atterberg Limits testing was performed on representative samples of the near-surface soils (less than 10 feet deep). A sample collected within the upper five feet of the site (existing fill) was found to have a Liquid Limit of 59 and Plasticity Index of 41. Another sample collected from between approximately 5 to 10 feet deep was found to have a Liquid Limit of 33 and Plasticity Index of 21.

8.2.2 Strength

Four unconfined compressive strength tests were performed on samples within the upper five feet, with results ranging from about 1,500 to 4000 psf.

9.0 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based upon the analysis of the information gathered during the course of this study and our understanding of the proposed improvements.

9.1 Conclusions

The site is considered suitable from a geotechnical and geologic perspective for the proposed improvements provided the recommendations of this report are incorporated into the design and implemented during construction. The predominant geotechnical and geological issues that will need to be addressed at this site are summarized below.

Seismic Considerations – The site is located within a seismically active region and should be designed to account for earthquake ground motions, as described in this report.

Expansive Soils – The presence of near surface highly expansive soils within the upper 12-feet of the site will require at least 24 inches of over-excavation in building pad areas and 12 inches in the pool floor areas and replacement with non-expansive Select fill or treated soil. Moisture conditioning of the fill and upper processed cut surfaces will be necessary.

Demolition – The proposed development will necessitate demolition of the existing building and foundations, concrete flatwork, asphalt pavement and the removal of buried utilities. Demolition excavations should be backfilled with engineered fill.

Utility Connections – It is recommended that utility connections at building perimeters be designed for one inch of potential movement in any direction where the utility enters the buildings. This should accommodate potential differential movement during a seismic event and for settlement and heave of the expansive soil.

Winter Construction – If grading occurs in the winter rainy season, appropriate erosion control measures will be required and weatherproofing of the building pad, foundation excavations, and/or pavement areas should be considered. Winter rains will also impact foundation excavations and underground utilities.

Existing Undocumented Fill - In areas where loose fill is encountered, such as existing landscaping areas , the loose fill should be removed and replaced as necessary with engineered fill. The over-excavation for placement of the Select Fill layer should aid in exposing loose fill areas.

9.2 Site Grading

9.2.1 General Grading

Site grading is generally anticipated to consist of very minor cuts and fills, other than the excavations for the swimming pools. Existing expansive clay soil which will be excavated at the site should not generally be used for structural fill but may be used in landscaping areas. Imported Select Fill should be non-expansive, having a Plasticity Index of 12 or less, an R-Value greater than 40, and contain at least 10 percent fines so the soil can bind together. Imported materials should be free of organic materials and debris, and should not contain rocks or lumps greater than three inches in maximum size. Import fill materials should be approved by the Geotechnical Engineer prior to use on site. On-site soil can be treated with five percent lime by weight (assume 125-pcf weight) to produce a Select fill material if desired. If this method is chosen we suggest that a specialty lime-treatment subcontractor perform the treatment and that it be performed with up to 18 inches treated in place – for the 24 inch thickness this will require some treatment , removal and replacing. Also, existing baserock and sand which underlies pavement areas or floor slabs to be removed may be reused as Select Fill upon examination and approval by the Geotechnical Engineer.

Final grading should be designed to provide positive drainage away from structures. Soil areas within 10 feet of proposed structures should slope at a minimum of four percent away from the buildings. Roof leaders and downspouts should discharge onto paved surfaces sloping away from the structure or into a closed pipe system channeled away from the structure to an approved collector or outfall.

9.2.2 Project Compaction Recommendations

Table 3 provides the recommended compaction requirements for this project. Some items listed below may not apply to this project. Specific moisture conditioning and relative density requirements will be discussed individually within applicable sections of this report.

TABLE 3 - PROJECT COMPACTION REQUIREMENTS

Description	Percent Relative Compaction	Minimum Percent Above Optimum Moisture Content
Building Pads, Onsite Soil	90	5
Building Pads, Subgrade Soil	90	5
Building Pads, Class 2 Baserock	90	2
Building Pads, Imported Select Fill	90	2
Building Pads, Treated Soil	90	2
AC Pavement Areas, Subgrade, Onsite Soil	95	3 to 5
AC Pavement Areas, Class 2 Baserock	95	1
AC Pavement Areas, Select Fill, Subgrade	95	1
AC Pavement Areas, Treated Soil, Subgrade	95	2
Concrete Pavement Areas, Subgrade, Onsite Soil	90	5
Concrete Pavement Areas, Class 2 Baserock	95	1
Concrete Pavement Areas, Select Fill	95	1
Concrete Pavement Areas, Treated Soil	95	2
Concrete Hardscape, Subgrade Soil	90	5
Underground Utility Backfill, 5-Feet and Deeper	95	5
Underground Utility Backfill, Upper 5-Feet	90	5
Underground Utility Backfill, Upper Foot in Pavement Areas	95	5
Retaining Wall Backfill, 5-Feet and Deeper	95	5
Retaining Wall Backfill, Upper 5-Feet	90	5

9.2.3 Site Preparation and Demolition

Site grading should be performed in accordance with these recommendations. A pre-construction conference should be held at the jobsite with representatives from the owner, general contractor, grading contractor, and Geosphere prior to starting the stripping and demolition operations at the site.

The site should be cleared of loose fill, concrete, asphalt, vegetation, organic topsoil, debris, and other deleterious materials within the proposed development areas. The grading contractor should be aware of buried objects and underground utilities at the site which are to be removed or abandoned appropriately.

In general, buried objects and debris should be removed from the site. The resulting excavations should be backfilled with properly compacted fill or other material approved by the Geotechnical Engineer.

Existing underground utilities to be abandoned at the site should be properly grouted closed or removed as needed. If the utilities are removed, the excavations should be backfilled with properly compacted fill or other material approved by the Geotechnical Engineer.

9.2.4 Grading for Structures

The areas receiving interior slabs and the pools should be underlain by non-expansive, Select Fill (24 inches for interior slabs, 12 for pool floors) to compensate for expansive soil conditions. The subgrade should be over-excavated if necessary to accommodate the Select Fill layer. The over-excavation and replacement with Select Fill should extend at least five-feet outside the structure perimeter. If the exposed soil is expansive, the soil should be moisture conditioned to at least five percent over optimum moisture. The over-excavation bottom should be scarified to a depth of at least eight inches and compacted to 90 percent relative compaction as determined by ASTM D1557 (Modified Proctor). If loose or soft soil is encountered at the bottom of the over-excavation, these should be removed to expose firm soil and backfilled with Select fill.

Select fill and treated soil should be compacted and moisture conditioned. Engineered fill should be placed in maximum eight-inch thick, un-compacted lifts. The fill should be thoroughly mixed during placement to provide uniformity in each layer.

9.2.5 Grading Pavement and Hardscape Areas

Areas to receive pavements should be scarified to a depth of eight inches below existing grade or final subgrade whichever is lower. Scarified areas should be moisture conditioned and compacted. Where required, engineered fill should be placed to reach subgrade elevation. Once the compacted pavement subgrade has been reached, it is recommended that baserock in paved and on-grade concrete slab areas be placed immediately after grading to protect the subgrade soil from drying. Alternatively, the subgrade should be kept moist by watering until baserock is placed.

Rubber-tired heavy equipment, such as a full water truck, should be used to proof load exposed pavement subgrade areas where pumping is suspected. Proof loading will determine if the subgrade soil is capable of supporting construction equipment without excessive pumping or rutting.

9.2.6 Site Winterization and Unstable Subgrade Conditions

If grading occurs in the winter rainy season, unstable and unworkable subgrade conditions may be present and compaction of on-site soils may not be feasible. These conditions may be remedied using soil admixtures, such as lime. A five percent mixture of lime based on a soil unit weight of 125 pcf is recommended. Treatment should vary between 12 to 18 inches, depending on the anticipated construction equipment loads. Credit for lime-treatment can be given toward use as Select Fill. More detailed recommendations can be provided during construction. Stabilizing subgrade in small, isolated areas can be accomplished with the approval of the Geotechnical Engineer by over-excavating one foot, placing Tensar BX1100 or equivalent geogrid on the soil, and then placing 12-inches of Class 2 baserock on the geogrid. The upper six inches of the baserock should be compacted to at least 95 percent relative compaction.

9.3 Utility Trench Construction

In the building pad area, utility trenches should be backfilled with Select fill above the utility bedding and shading materials. In pavement areas, utility trenches may be backfilled with native soil above the utility bedding and shading materials. If rocks larger than four inches in maximum size are encountered, these should be removed from the fill prior to placement in the utility trenches. Utility bedding and shading compaction requirements should be in conformance with the requirements of the local agencies having jurisdiction and as recommended by the pipe manufacturers. Jetting of trench backfill is not recommended.

Pea gravel, rod mill (pea gravel with sand), or other similar self-compacting material should NOT be utilized at the site. This material may act as a conduit for subsurface moisture migration. Utility trenches should be sealed with concrete, clayey soil, sand-cement slurry, or controlled density fill (CDF) where the utility enters the building under the perimeter foundation. This would reduce the potential for migration of water beneath the building through the shading material in the utility trench.

If rain is expected and the trench will remain open, the bottom of the trench may be lined with one to two inches of gravel. This would provide a working surface in the trench bottom. The trench bottom may have to be sloped to a low point to pump the water out of the trench.

9.4 Temporary Excavations and Shoring

The contractor should utilize proper Cal OSHA methods during construction. Excavations in soil more than five feet deep and less than 10 feet deep should have side slopes constructed at 1H:1V (horizontal to vertical). The grading contractor should make selection of temporary side slopes based upon the materials encountered during the excavation. Maximum slope ratios provided above are assumed to be uniform from top to toe of the slope. Adequate provisions should be made to prevent water from ponding on top of the slope and from flowing over the slope face. Surcharge loads should not be permitted within 10 feet of the top of the slope. Desiccation or excessive moisture in the excavation could reduce stability and require shoring or laying backside slopes.

Lateral pressures on shoring for cut heights up to 12 feet for both horizontal adjacent conditions are contained in Figure 11.

9.5 Building Foundations

9.5.1 Shallow Foundations

The use of shallow conventional footings appears suitable for the planned structure. However, due to the highly expansive nature of the site soils, foundations for the structures should be a minimum of 36-inches below lowest adjacent finish soil grade. Based on a minimum Factor of Safety of 3.0, the recommended allowable bearing capacity for the proposed foundations is 2,700 pounds per square foot (psf) for DL + LL, with an increase of 200 psf for each additional foot of depth up to a maximum of 3500 psf. The allowable bearing capacity may be increased by 1/3 for temporary wind and seismic loading. Total settlement of building foundations is anticipated to be about one-inch or less with differential settlements of ½ inch or less. An allowable friction coefficient of 0.30 may be utilized.

Footing excavations should have firm bottoms and be free from excessive slough prior to concrete or reinforcing steel placement. The footing bottoms should be kept moist to prevent the development of shrinkage cracks. If shrinkage cracks develop in the footing bottoms, they should be moisture conditioned prior to the placement of concrete.

Where utility trenches are to be located adjacent to foundations, the bottom of the footing should be located below an imaginary 1:1 (horizontal to vertical) plane projected upward from the nearest bottom edge of the utility trench.

If construction occurs during the winter months, it is suggested that a thin layer of concrete be placed at the bottom of the footings. This will protect the bearing soil and facilitate removal of water and slough if rainwater fills the excavations. The foundation excavations should be observed by a representative of Geosphere prior to placement of reinforcing steel or concrete to evaluate the exposed soil conditions.

9.5.2 Lateral Resistance

Shallow foundations can resist lateral loads with a combination of bottom friction, side friction, and passive resistance. An allowable coefficient of friction of 0.30 between the base of the foundation elements and underlying material are recommended. Side friction for the perimeter foundations on the exterior side of buildings is an allowable uniform side shear resistance of 30 psf. Side friction resistance should only be utilized below a depth of two feet below finish floor. An allowable passive resistance equal to an equivalent fluid weighing 165 pounds per cubic foot (pcf) acting against the foundation may be utilized, using a factor of safety of 1.5. The friction between the bottom of the floor slab and the underlying soil should not be utilized to resist lateral forces.

9.6 Concrete Slabs-on-Grade

9.6.1 General Recommendations

Interior concrete slab-on-grade floors should be a minimum of five-inches in thickness. A modulus of subgrade reaction of 200 pci on an upper 24-inch cap of select import soil can be utilized. Interior floors sensitive to moisture should be underlain by a high quality vapor retarder meeting ASTM E1745 Class C requirements, such as Griffolyn Type 65, Griffolyn VaporGard, Moistop Ultra C, or equivalent. ASTM E1643 should be utilized as a guideline for the installation of the vapor retarder. A four–inch thick capillary rock layer or rock cushion is required beneath the floor slab in areas receiving moisture sensitive floor coverings. A sand layer is not required over the vapor retarder. If sand on top of the vapor retarder is required by the design engineers, the thickness should be minimized to less than one-inch. If construction occurs in the winter months, water may pond within the sand layer since the vapor retarder may prevent the vertical percolation of rainwater.

The building pad subgrade is expected to have numerous utility and foundation excavations and the subgrade ground surface will become disturbed from construction traffic. Appropriate compactive effort will be required when backfilling utilities and around foundations after the concrete has set. The subgrade surface should be compacted and smoothened with light construction equipment prior to placement of the vapor barrier

9.6.2 Exterior Concrete Flatwork

Due to the very highly expansive soil, exterior concrete flatwork should be placed on 12-inches of Select Fill. The subgrade beneath all flatwork should be compacted to 90 percent and moisture conditioned to a minimum of five percent over optimum. Flatwork should be reinforced with at least #4 reinforcing steel on 18-inch centers. Where flatwork is contiguous to the building at doorways and entrances, the flatwork should be doweled into the building foundation to reduce potential tripping hazards.

9.7 Swimming Pool Parameters

The site soils are very highly expansive and the pools should be designed for correspondingly high lateral pressures and should also receive mitigative measures relative to the potential for high uplift pressures which could result from moisture increase in the underlying soils. We recommend that an active lateral pressure of at least 60 pcf be utilized or, if the at-rest condition is considered to apply, then a value of 75 pcf in Equivalent Fluid Pressure terms should be applied. The use of a back-drainage system such as MiraDrain panels should be employed, as well as placement of an under-pool subdrain. The subdrain system drain system should include a manifold of four-inch minimum diameter perforated pipe in a12-inch thick blanket of CalTrans Class 2 Permeable Material or one drain in the rock if the pool soil subgrade is graded to drain to the pipe. The perforated pipes should be routed into a solid pipe connected into a suitable collector such as a storm drain line (if allowed) or the sewer line (if backflow design is applied). The Modulus of Subgrade Reaction for the compacted permeable base underlying the pools may be considered to be 200 psi/in.

9.8 Dewatering

There may be some need for dewatering on the deeper pool and utility excavations but trenching to a shallow sump will probably be adequate. While groundwater conditions can be variable, we do no anticipate a need for an extensive dewatering system such as well-points.

9.9 Plan Review

It is recommended that Geosphere be provided the opportunity to review the shoring, foundation, grading, and drainage plans prior to construction. The purpose of this review is to assess the general compliance of the plans with the recommendations provided in this report and the incorporation of these recommendations into the project plans and specifications.

9.10 Observation and Testing During Construction

It is recommended that Geosphere be retained to provide observation and testing services during site preparation, site grading, utility construction, foundation excavation, and to observe final site drainage. This is to observe compliance with the design concepts, specifications and recommendations, and to allow for possible changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

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LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report are based upon the soil and conditions encountered in the borings. If variations or undesirable conditions are encountered during construction, Geosphere should be contacted so that supplemental recommendations may be provided.

This report is issued with the understanding that it is the responsibility of the owner or his representatives to see that the information and recommendations contained herein are called to the attention of the other members of the design team and incorporated into the plans and specifications, and that the necessary steps are taken to see that the recommendations are implemented during construction.

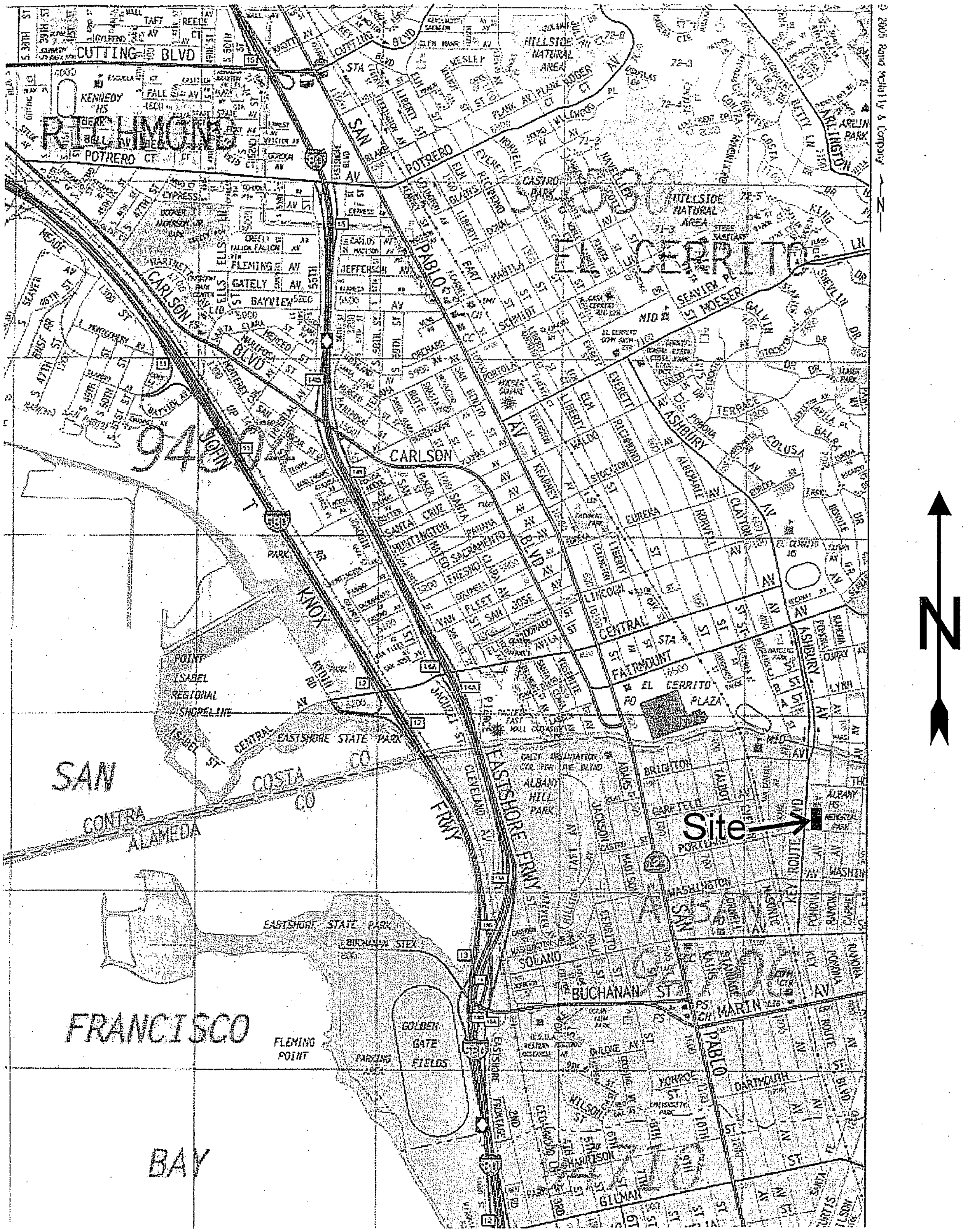
The findings and recommendations presented in this report are valid as of the present time for the development as currently proposed. However, changes in the conditions of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Accordingly the findings and recommendations presented in this report may be invalidated, wholly or in part, by changes outside our control. Therefore, this report is subject to review by Geosphere after a period of three (3) years has elapsed from the date of issuance of this report. In addition, if the currently proposed design scheme as noted in this report is altered, Geosphere should be provided the opportunity to review the changed design and provide supplemental recommendations as needed.

Recommendations are presented in this report which specifically request that Geosphere be provided the opportunity to review the project plans prior to construction and that we be retained to provide observation and testing services during construction. The validity of the recommendations of this report assumes that Geosphere will be retained to provide these services.

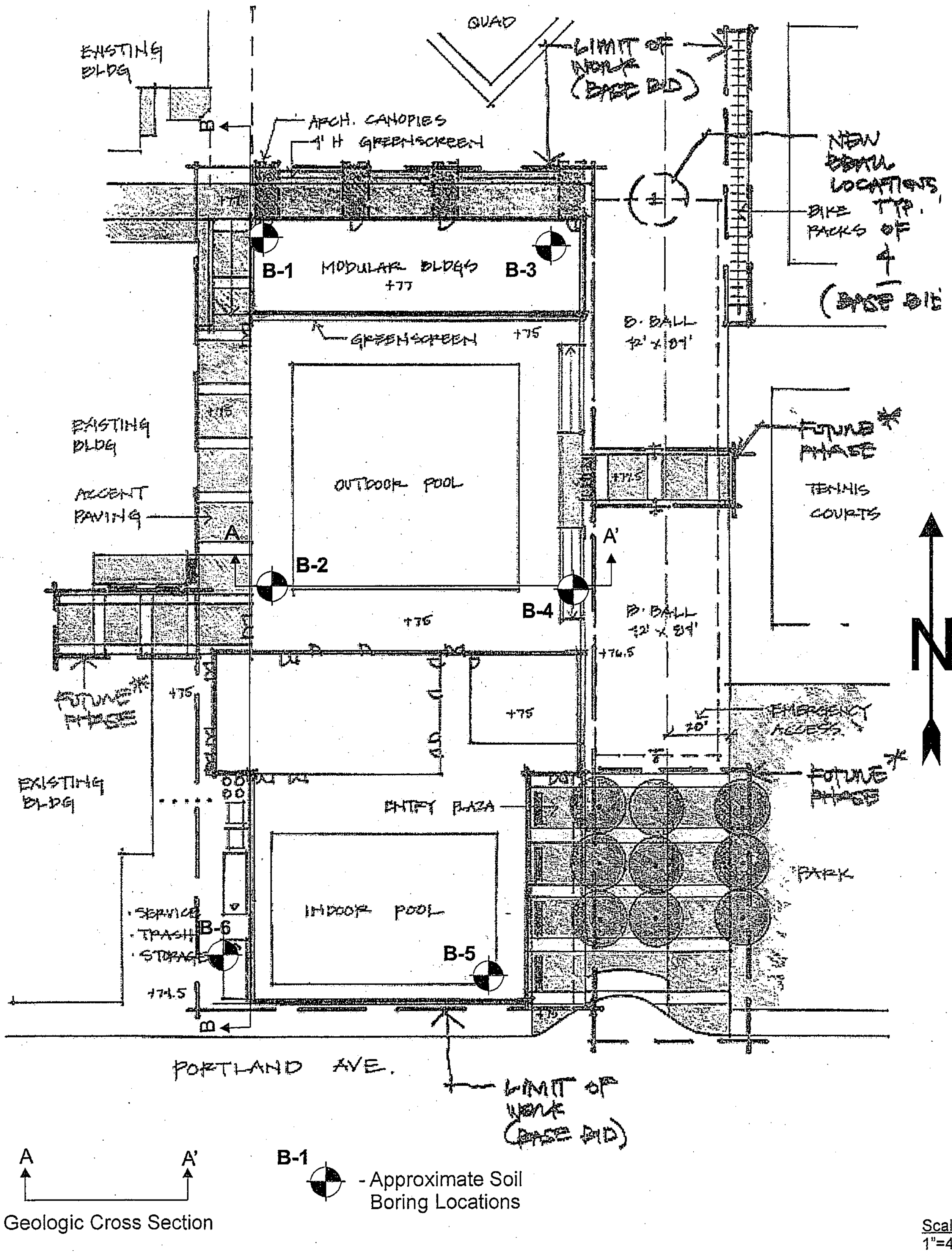
This report was prepared upon your request for our services, and in accordance with currently accepted geotechnical engineering practice. No warranty based on the contents of this report is intended, and none shall be inferred from the statements or opinions expressed herein.

The scope of our services for this report did not include an environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on, below or around this site. Any statements within this report or on the attached figures, logs or records regarding odors noted or other items or conditions observed are for the information of our client only.

FIGURES

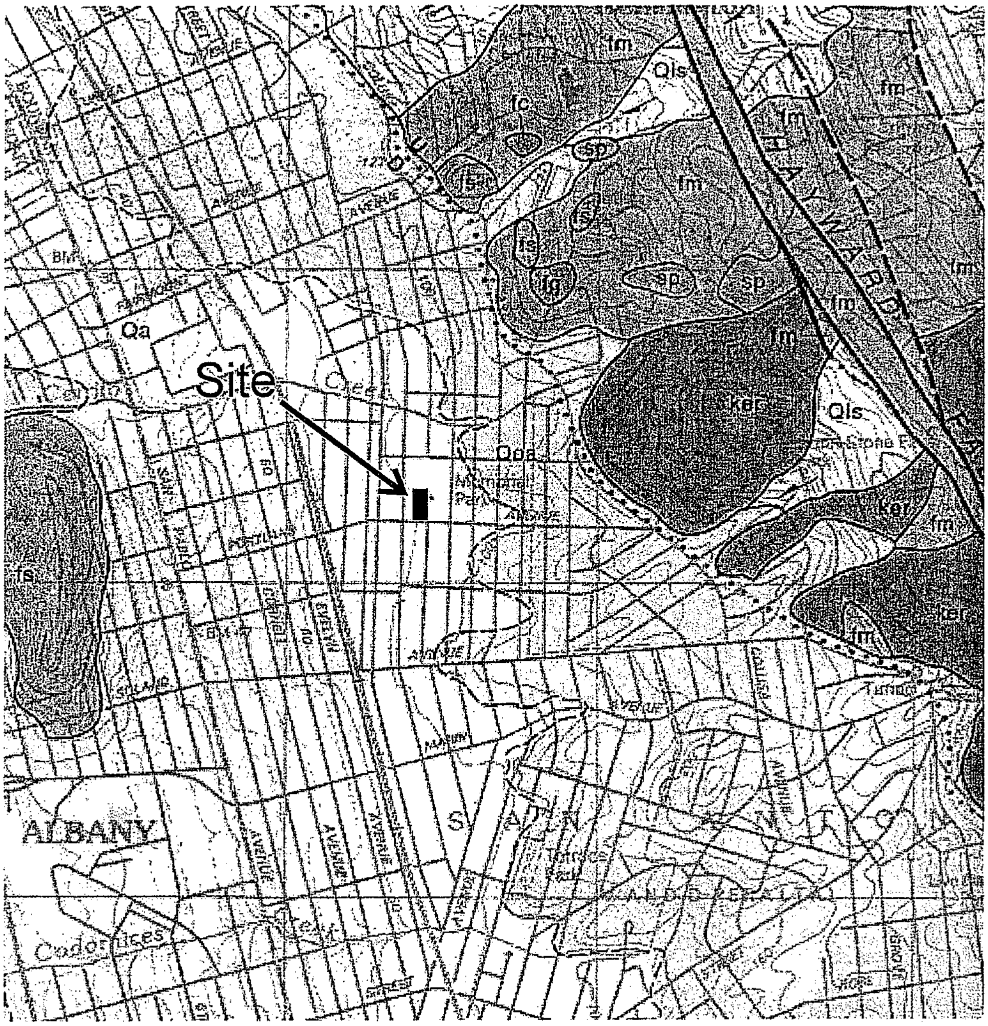


Albany High School - Aquatic Center	91-02320-A	December 2008
Geosphere Consultants, Inc.	Site Vicinity Map	Figure 1



Albany High School - Aquatic Center	91-02320-A	December 2008
Geosphere Consultants, Inc. AN ECS COMPANY Geotechnical Engineering · Engineering Geology Environmental Management · Water Resources	Site Plan	Figure 2

Source: Santa Barbara Museum of Natural History - Dibblee Geology Center, Geologic Map of the Richmond Quadrangle, Dibblee 2005.

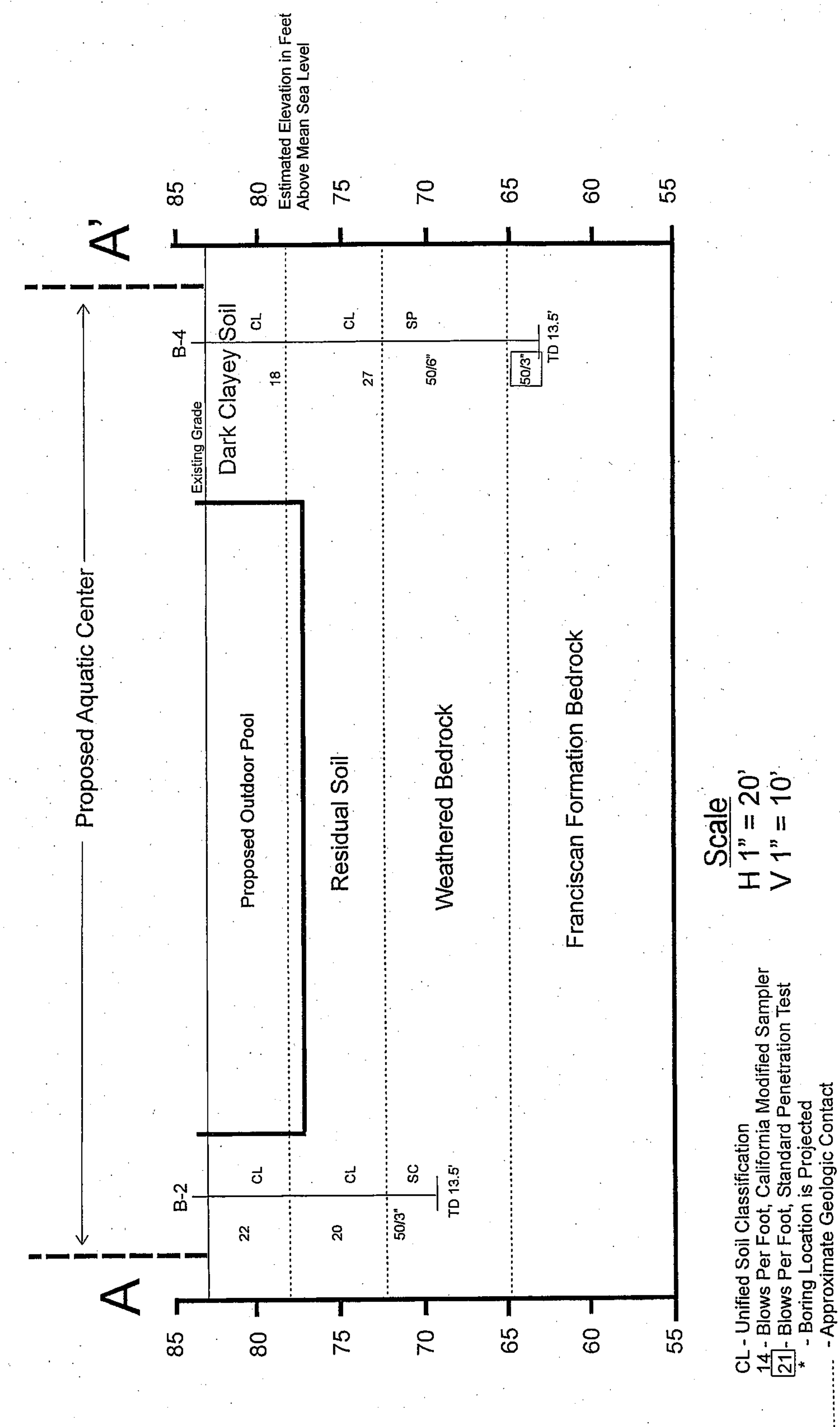


N

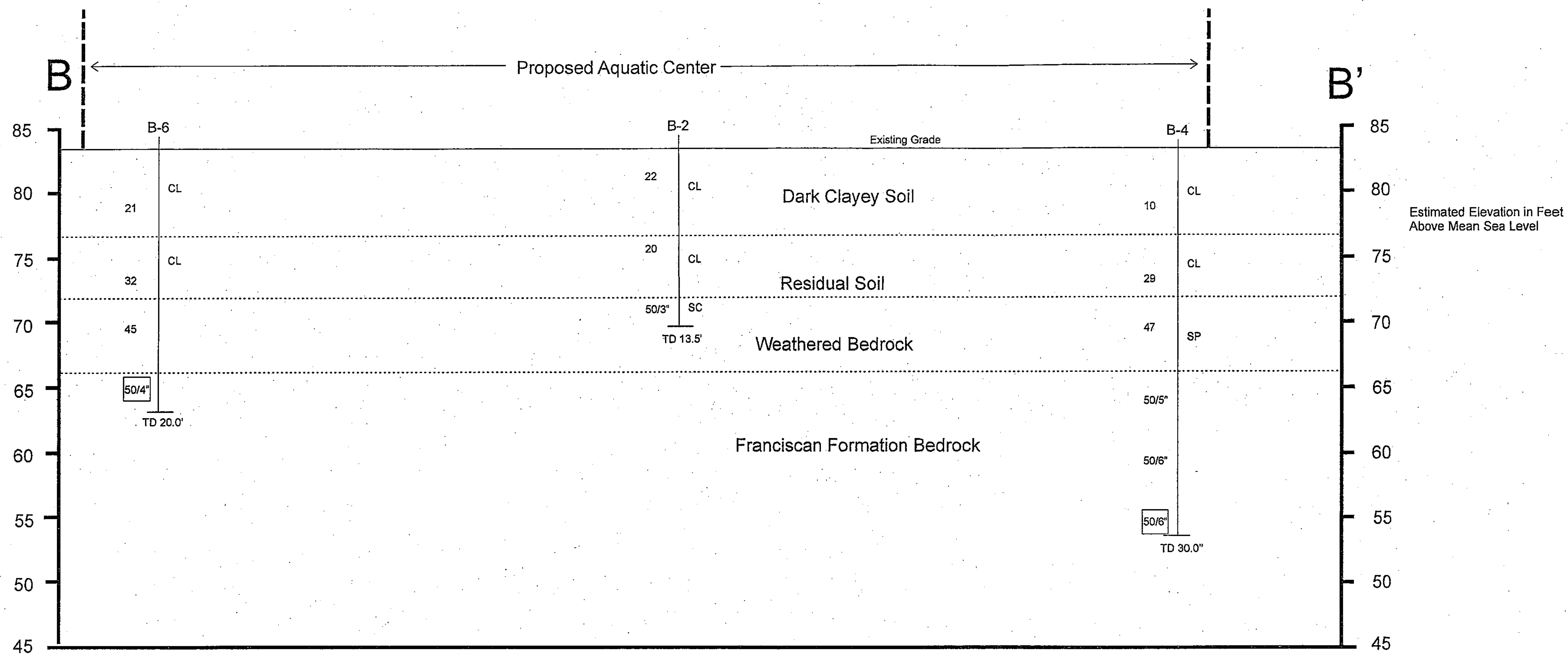
Qoa - Older Surficial Deposits
Qa - Surficial Deposits
ker - Volanic Rocks, Andesitic
fm - Franciscan Melange, claystone and graywacke

No Scale

Albany High School - Aquatic Center	91-02320-A	December 2008
Geosphere Consultants, Inc. AN EIS COMPANY Geotechnical Engineering - Engineering Geology Environmental Management - Water Resources	Site Vicinity Geology Map	Figure 3



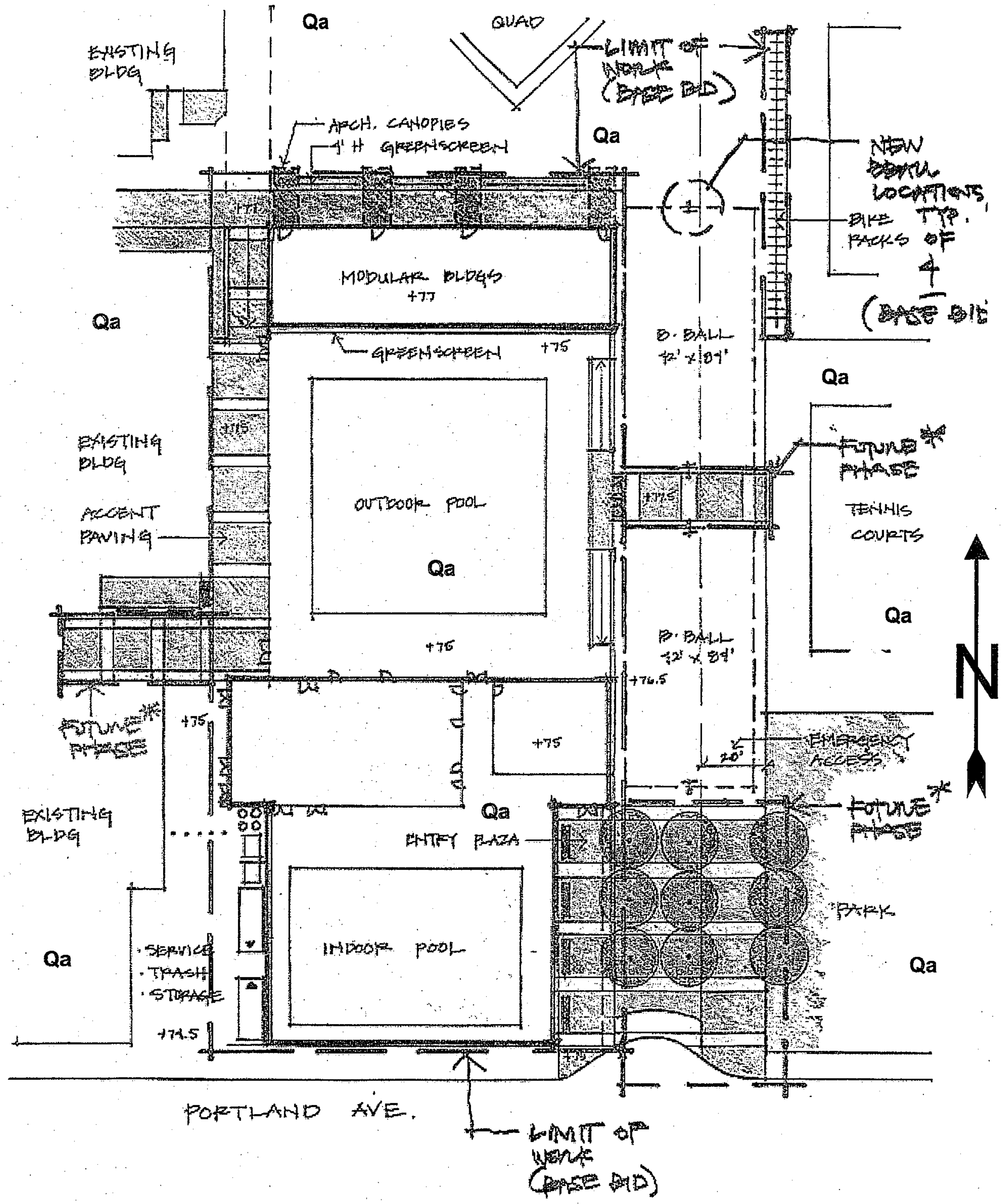
Albany High School - Aquatic Center	91-02320-A	December 2008
Geosphere Consultants, Inc. AN EFS COMPANY Geotechnical Engineering • Engineering Geology Environmental Management • Water Resources	Schematic Geologic Cross-Section A-A'	Figure 4a



<u>Scale</u>
H 1" = 30'
V 1" = 10'
3 X Vertical Exaggeration

CL - Unified Soil Classification
14 - Blows Per Foot, California Modified Sampler
$\boxed{21}$ - Blows Per Foot, Standard Penetration Test
* - Boring Location is Projected
.... - Approximate Geologic Contact

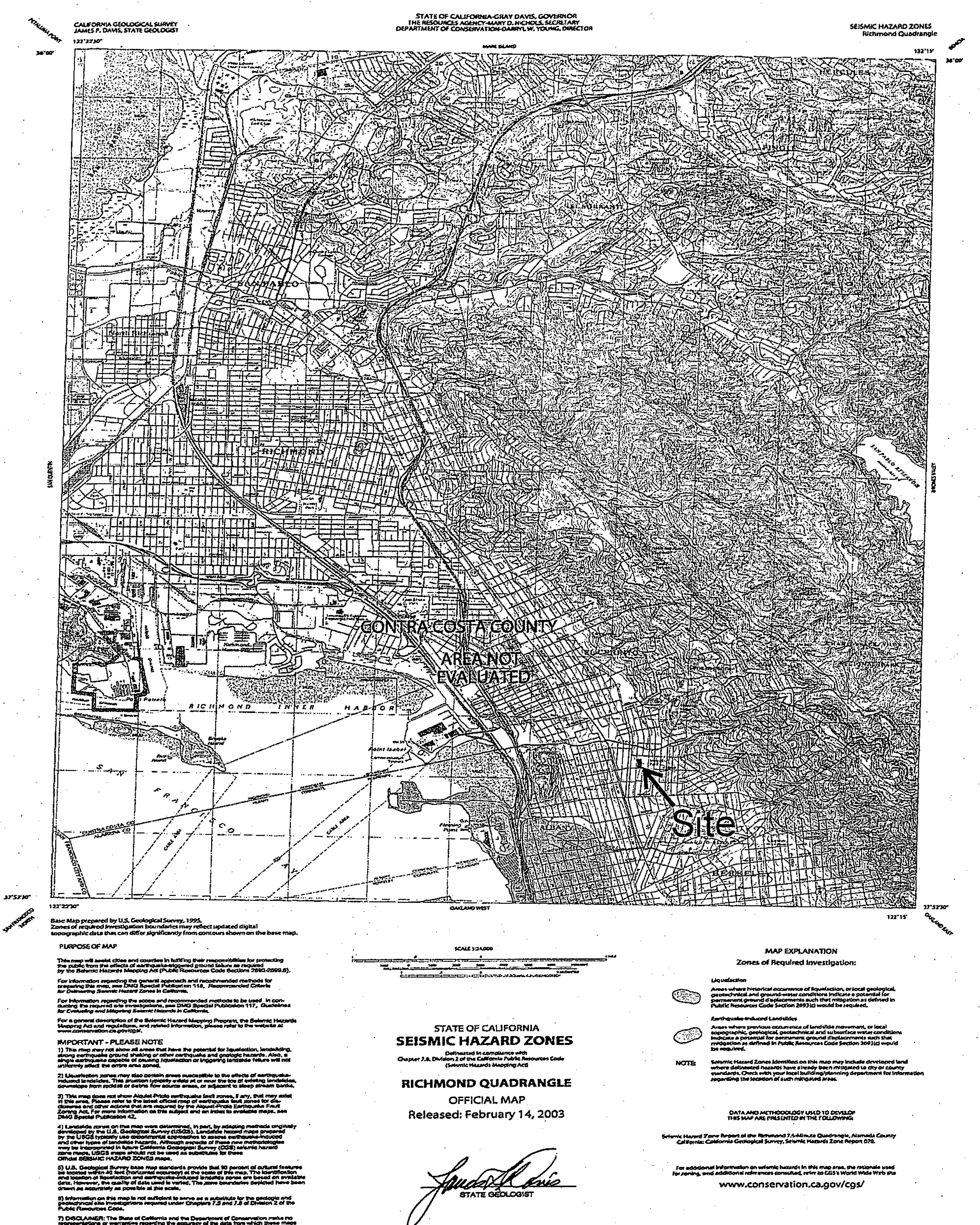
Albany High School- Aquatic Center	91-02320-A	December 2008
Geosphere Consultants, Inc.	Schematic Geologic Cross-Section B - B'	Figure 4b



Qa - Surficial Deposits, Alluvium

Scale
1"=48'

Albany High School - Aquatic Center	91-02320-A	December 2008
Geosphere Consultants, Inc. AN EFS COMPANY Geotechnical Engineering • Engineering Geology Environmental Management • Water Resources	Site Geology Map	Figure 5



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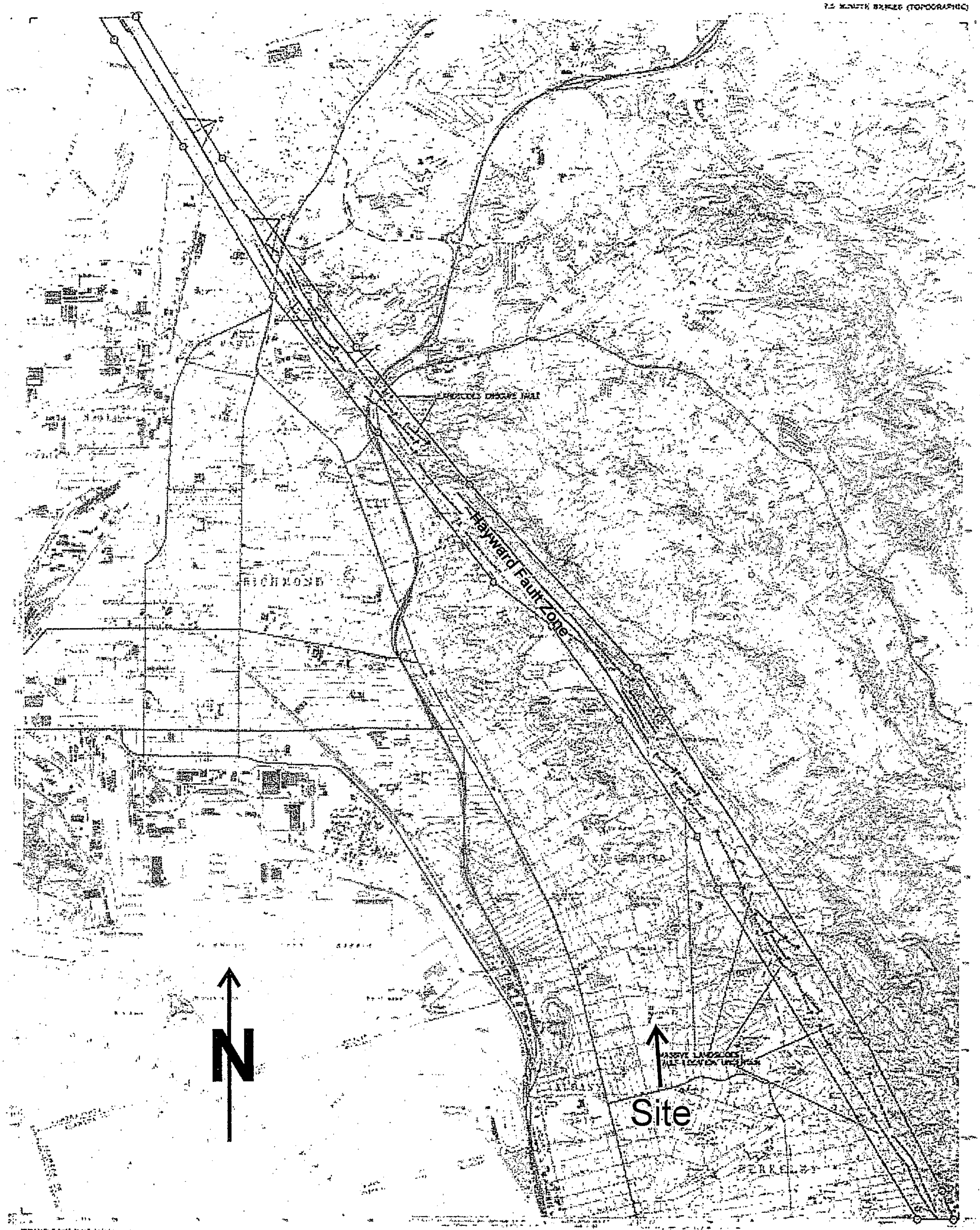
Albany High School - Aquatic Center	91-02320-A	December 2008
Geosphere Consultants, Inc. AN ETS COMPANY Geotechnical Engineering • Engineering Geology Environmental Management • Water Resources	Liquefaction Susceptibility Map	Figure 6

Source: State of California Special Studies Zones - Richmond, Revised Official Map, Effective January 1, 1982.



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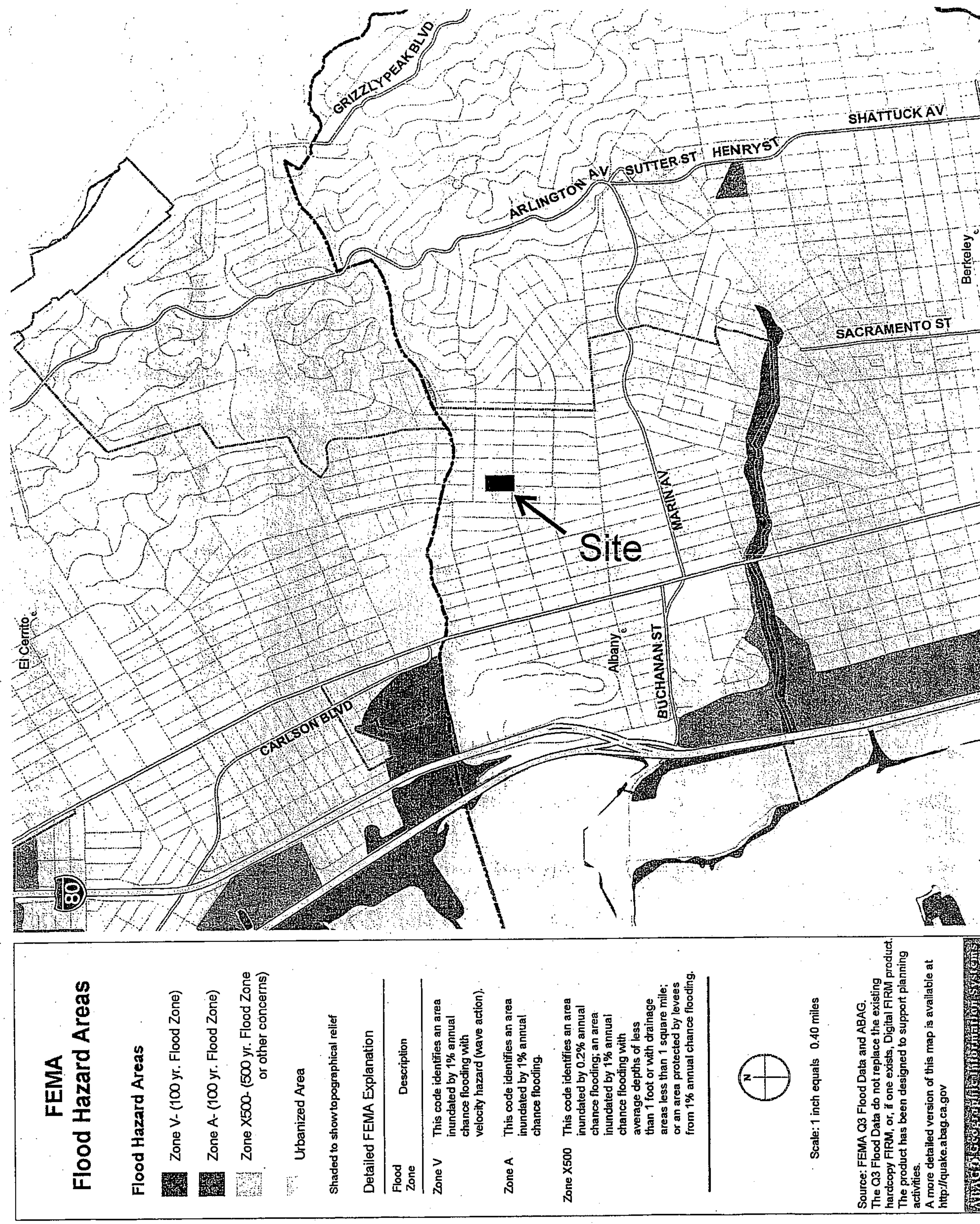
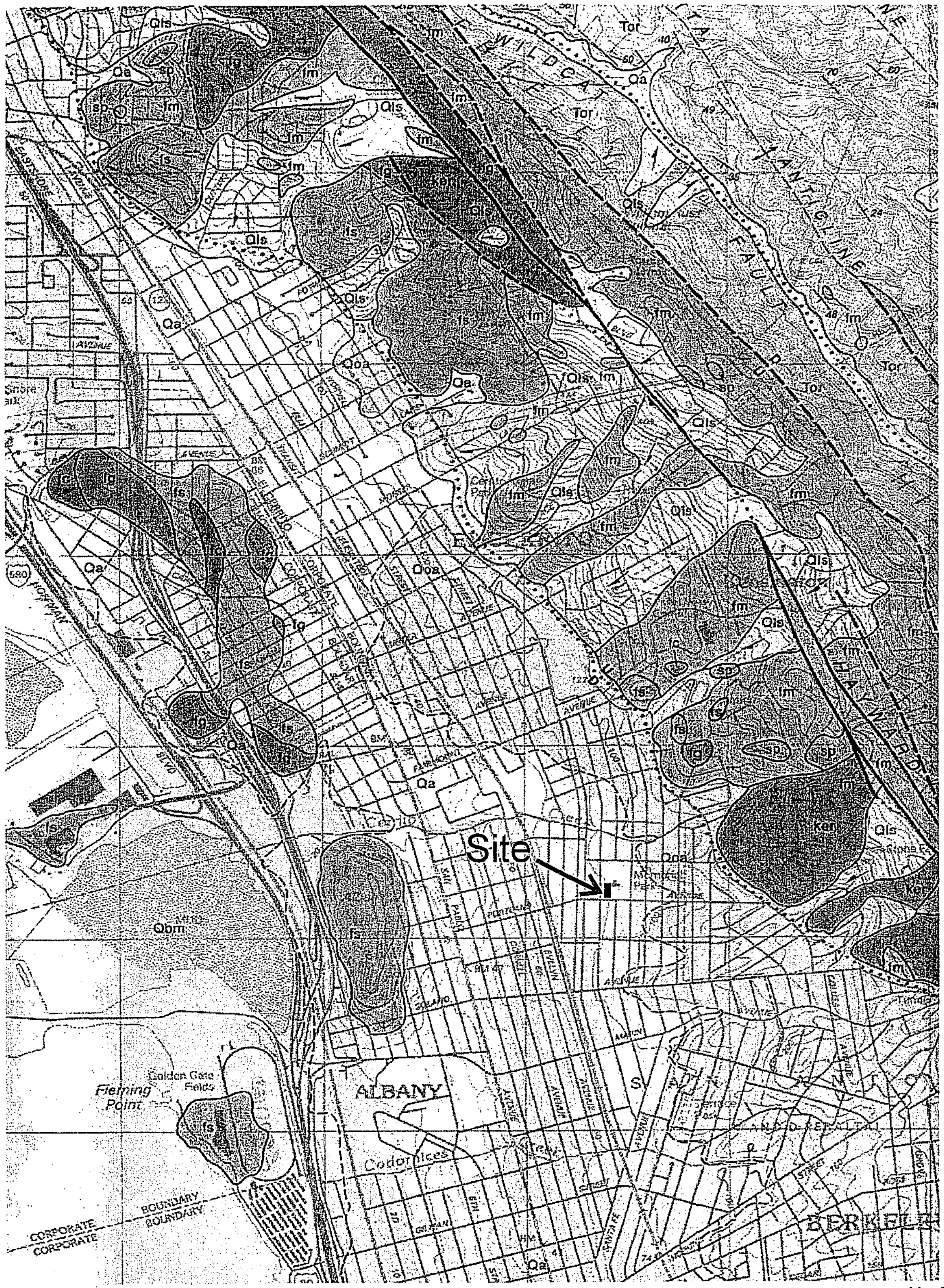
Albany High School - Aquatic Center	91-02320-A	December 2008
Geosphere Consultants, Inc.	Regional Fault Map	Figure 7



No Scale

Albany High School - Aquatic Center	91-02320-A	December 2008
Geosphere Consultants, Inc. An ECS Company. Geotechnical Engineering • Engineering Geology • Environmental Management • Water Resources	Flood Hazard Map	Figure 8

Source: Santa Barbara Museum of Natural History - Dibblee Geology Center, Geologic Map of the Richmond Quadrangle, Dibblee 2005.



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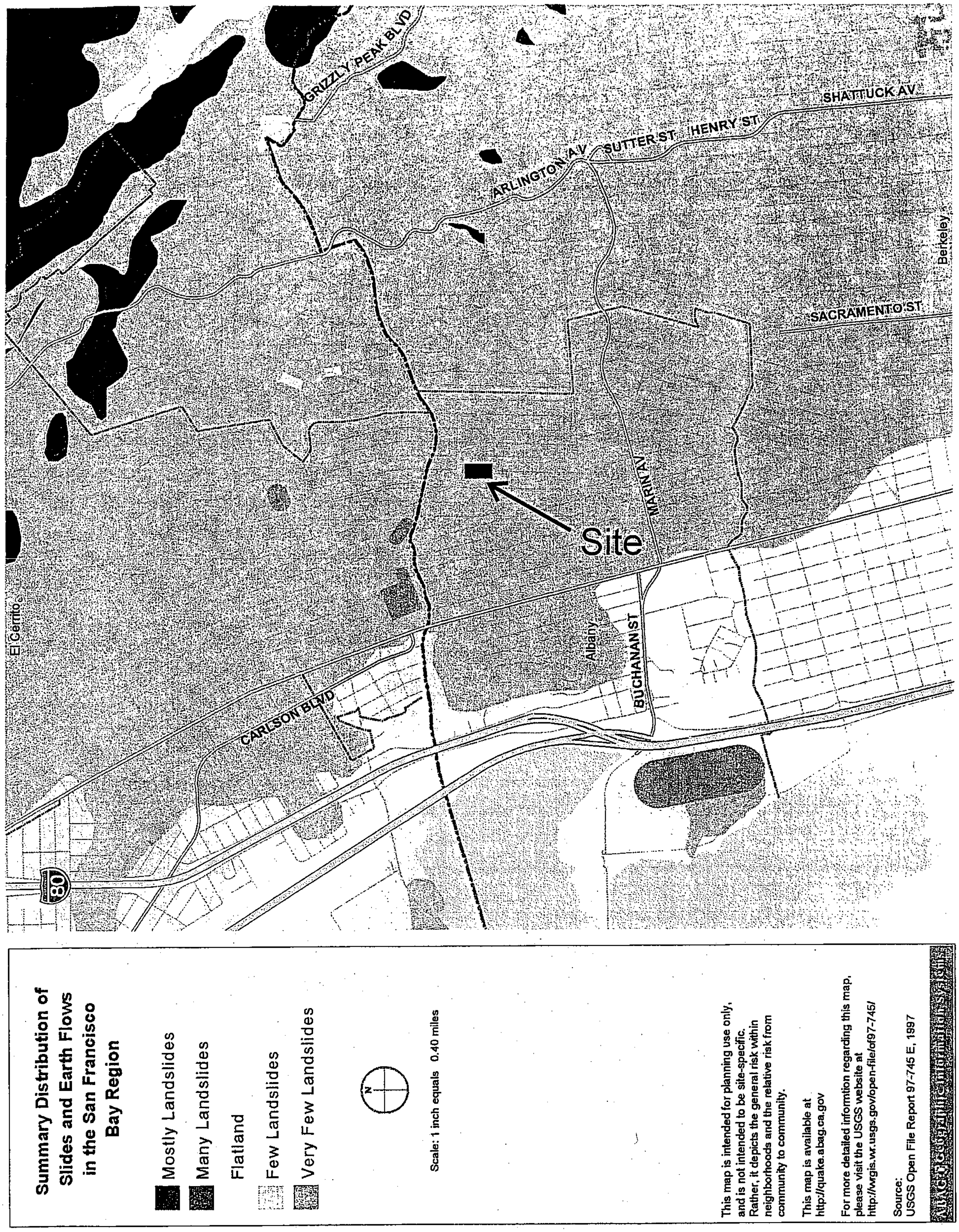
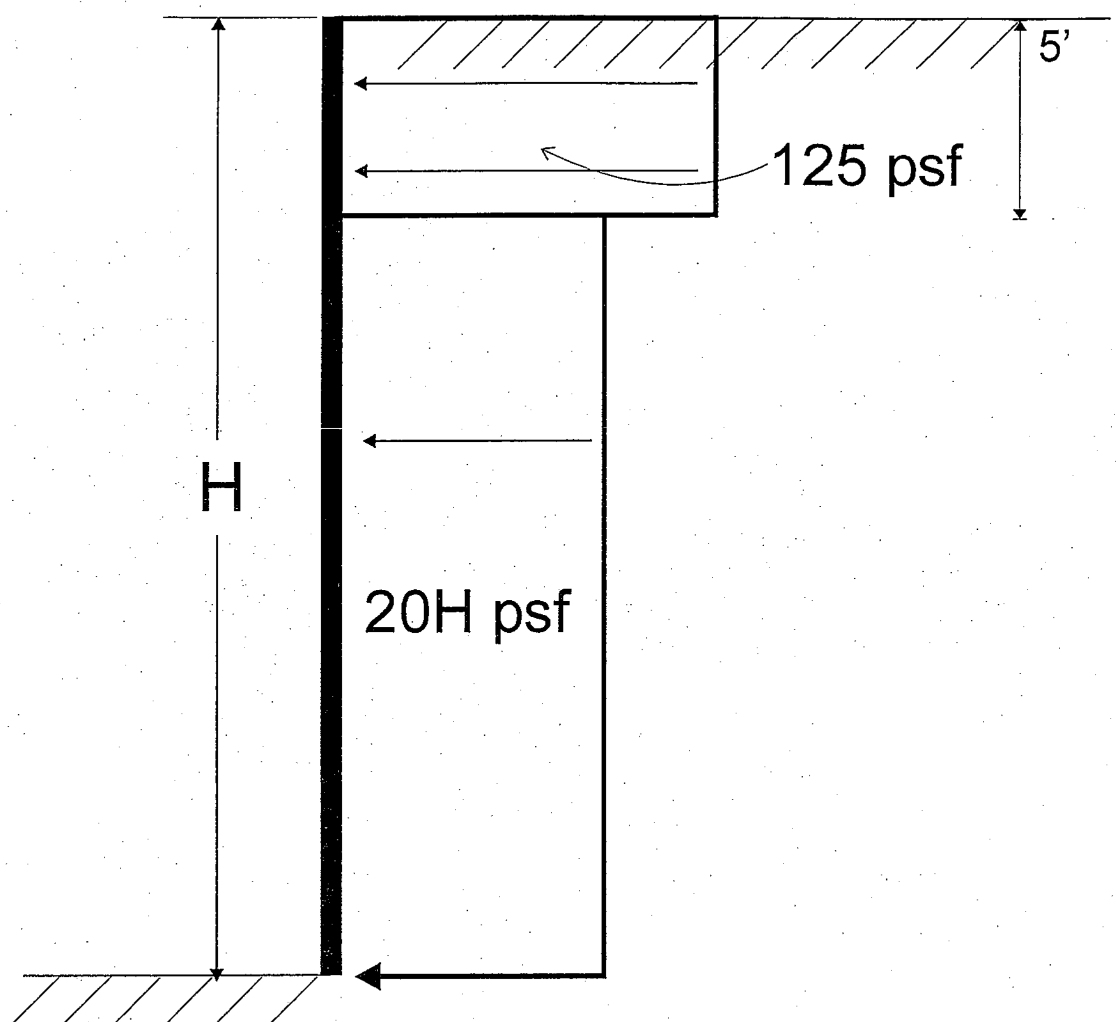
Albany High School - Aquatic Center	91-02320-A	December 2008
Geosphere Consultants, Inc. AN ETS COMPANY Geotechnical Engineering • Engineering Geology Environmental Management • Water Resources	Regional Geologic Map	Figure 9



No Scale

Albany High School - Aquatic Center	91-02320-A	December 2008
Geosphere Consultants, Inc. AN SES COMPANY Geotechnical Engineering · Engineering Geology Environmental Management · Water Resources	Existing Landslide Map	Figure 10



Not to scale

Lateral earth pressures for temporary shoring design should be taken as 125 psf for $0<H<5$ feet and should be calculated as 20H below existing ground thereafter.

Albany High School - Aquatic Center	91-02320-A	December 2008
Geosphere Consultants, Inc. AN ETS COMPANY		Figure #11 Temporary Shoring Pressures

APPENDIX A

Boring Logs