GEOMATRIX

GEOTECHNICAL STUDY
WOODFIN SUITE HOTEL
Emeryville, California

Prepared for:

Hardage Suite Hotels
9255 Town Center Drive, Suite 900
San Diego, California 92121

July 14, 1997
Project No. 4105

Geomatrix Consultants

100 Pine Street, 10th Floor
San Francisco, CA 94111
(415) 434-9400 • FAX (415) 434-1365

GEOMATRIX

July 14, 1997
Project: 4105

Mr. Peter Kruse
Hardage Suite Hotels
9255 Town Center Drive, Suite 900
San Diego, California 92121

Dear Mr. Kruse:

We are pleased to present the results of our geotechnical study for the proposed Woodfin Suite Hotel in Emeryville, California. The study including drilling exploratory borings and excavating test pits, testing and examining selected soil samples, and developing foundation and earthwork recommendations for the proposed 12-story building. The accompanying report describes the field exploration and laboratory testing programs conducted for this study and presents our geotechnical recommendations for design and construction of the new hotel.

A draft version of this report was previously sent to you on May 23, 1997. Questions regarding settlement of lightly-loaded ancillary buildings at the site have been addressed in the enclosed revised report.

We have appreciated the opportunity of working with you and your design team on this project. Please contact either of the undersigned if you have any questions about this report or if we can be of further service.

Sincerely,

GEOMATRIX CONSULTANTS, INC.

Laura Varner
Senior Engineer

Carl Basore
Principal Engineer

LV/ckk
PROJ\4105-LTR.DOC

Enclosure

cc: Pat Harrigan, Skilling Ward Magnusson Barkshire, Inc.

GEOMATRIX

GEOTECHNICAL STUDY
WOODFIN SUITE HOTEL
Emeryville, California

Prepared for:

Hardage Suite Hotels
9255 Town Center Drive, Suite 900
San Diego, California 92121

July 14, 1997
Project No. 4105

Geomatrix Consultants

GEOMATRIX

TABLE OF CONTENTS

GEOMATRIX

TABLE OF CONTENTS
(continued)

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

GEOTECHNICAL STUDY
WOODFIN SUITE HOTEL
Emeryville, California

1.0 INTRODUCTION

A new 12-story hotel is planned for construction at the south end of the Market Place in Emeryville, California. The new hotel will be constructed on Shellmound Street in a paved parking area previously occupied by industrial buildings. An outdoor swimming pool and deck area will be constructed adjacent to the hotel building. New parking areas will be provided around the perimeter of the hotel.

Geomatrix Consultants, Inc., (Geomatrix), previously performed geotechnical studies at the site for buildings that were not constructed. Results of the previous studies were presented in the following three reports:

1. Geotechnical Study, Movie Theater, Emeryville, California, dated June 23, 1987. Prepared for The Martin Company.
2. Preliminary Exploration Program, Emeryville Hotel, Emeryville, California, dated September 3, 1987. Prepared for The Martin Company.
3. Geotechnical Study, Hawthorne Suites Hotel, Emeryville, California, dated April 20, 1988. Prepared for Another Tree Development Corporation.

The geotechnical study presented in this report was performed in accordance with the Scope of Services outlined in our proposal to Hardage Suite Hotels dated March 24, 1997. The purpose of this study was to perform a supplementary field exploration program to assess subsurface soil conditions and explore for buried walls, footings, and obstructions at the new hotel site and develop recommendations and design criteria for foundation support and earthwork construction for the planned project. The following information, recommendations, and design criteria for the planned hotel development are presented in this report:

- a site plan showing the location of exploratory borings and excavations performed at the site, as well as buried foundations encountered in the excavations
- description of subsurface conditions encountered in the borings and logs of the borings
- results of laboratory tests performed on soil samples
- recommendations for design of foundations, including spread footings for light ancillary structures and retaining walls and driven piles for the hotel building
- estimated settlement of structures
- lateral earth pressures for design of low retaining walls, including active and passive pressures and coefficient of friction value to resist sliding
- recommendations for subsurface drains, if required
- recommendations for site preparation and earthwork construction, including the suitability of onsite soils for use as fill
- recommendations for design of asphalt concrete pavements and subgrade reaction coefficient for concrete pavements
- lateral earth pressures and construction considerations for the swimming pool.

2.0 PROJECT DESCRIPTION

The new Woodfin Suite Hotel is planned for construction on approximately 3.6 acres of property just north of the Powell Street overpass and west of the Amtrak Station in Emeryville, California. Architectural plans prepared by Austin Design Group indicate that the new structure will be 12 stories high and measure approximately 60 feet by 270 feet in plan dimension. A one-story conference center is planned along the west side of the building facing Shellmound Street, as well as the Porta Cochere entrance structure. A covered driveway will also be constructed along the east side of the building.

An outdoor swimming pool will be constructed on the west side of the building, and will be enclosed by a 6-foot-high fence and trellis. New parking lot construction and landscaping are planned for the perimeter of the building and will occupy the remainder of the site. Landscaping plans may require low retaining walls, and several 8- and 9-foot-high concrete screen and security walls are also planned at the northern building perimeter and on the south and east property lines.

A preliminary building plan and structural design information provided by Skilling Ward Magnusson Barkshire Inc. indicate that the new structure will be a reinforced concrete frame building with typical column spacing of 17 feet by 22 feet. Interior column loads are expected to be 1250 kips, and exterior column loads will be 625 kips. These building loads are for combined dead and live loads. The floor slabs will be constructed with lightweight concrete and post-tensioned concrete slabs. Lateral loads will be resisted with concrete shear walls. One-story structures are expected to have wall loads ranging from 0.2 kips/ft (exercise room) to 0.5 kips/ft (meeting room).

The site is presently a paved relatively level parking area. Existing grade at the site varies between elevation 7.5 feet to elevation 9.7 feet. It is our understanding that ground floor level in the new hotel building will be at approximately elevation 10.4 feet. Cuts and fills required to bring the site to grade and prepare the new building pad and provide surface drainage for the new parking areas are not expected to exceed 2½ feet.

An existing storm drain which crosses the hotel site will be removed and replaced with a new drainage pipe on the east side of the site. A new catch basin also will be installed at the southwest corner of the property and will tie into an existing storm drain.

3.0 FIELD EXPLORATION AND LABORATORY TESTING

General descriptions of the field exploration and laboratory testing programs performed for this study are presented in the following text. More detailed explanations of the field exploration and laboratory testing programs are presented in Appendices A and B. Subsurface information obtained during previous studies at the site by Geomatrix during 1988 and are presented in Appendix C.

3.1 FIELD EXPLORATION

The field exploration program performed for the new Woodfin Suite Hotel consisted of drilling two soil borings to obtain additional subsurface information to supplement information obtained in two deep borings drilled during the 1988 study for the Hawthorn Suites Hotel at the site and to better define and characterize bearing soils for design of driven pile foundations. The borings were drilled within the footprint of the proposed hotel at the locations shown on Figure 1. Eleven exploratory test pits were also excavated at the locations shown on Figure 1 along proposed building lines to explore for the presence of buried footings, walls, or slabs remaining beneath the existing parking lot after demolition of industrial buildings in the vicinity of the hotel site.

The two soil borings were drilled to depths of 85 to 90 feet using rotary wash drilling procedures. The borings were logged in the field by examining drill cuttings and retrieved samples. Soil samples were obtained from each boring and delivered to the laboratory for further examination and testing. Final logs were prepared based on the field logs, examination of samples in the laboratory, and laboratory test results and are presented in Appendix A. A boring log explanation sheet is also presented.

Soil samples were generally collected at 5-foot intervals using either a modified California drive sampler, or a Standard Penetration Test sampler. Samples were typically collected at smaller intervals at depths shallower than 10 feet and at greater intervals at depths greater than 30 feet.

Cuttings and fluids generated during drilling were placed in 55-gallon drums stored on the south end of the site. The borings were backfilled with cement-bentonite grout in accordance with procedures required by Alameda County.

The exploratory test pits were excavated to a maximum depth of 7 feet. Several of the test pits, T-2, T-6, T-7, T-8, and T-9 were terminated at shallow depths where obstructions were encountered. The excavations were logged in the field, and final logs were prepared based on the field logs, and are presented in Appendix A. An explanation of terms used on the test pit logs is provided on the boring log explanation sheet. One bulk soil sample was collected from one of the test pits for laboratory testing.

3.2 LABORATORY TESTING

Selected soil samples from the exploratory borings were delivered to the laboratory for examination and geotechnical testing to evaluate their physical characteristics and engineering properties. Samples were tested to measure their moisture content and unit weight, grain size distribution, and unconfined compressive strength. Results of laboratory tests are presented adjacent to the corresponding samples on the boring logs presented in Appendix A. Results of grain size distribution curves are also presented in Appendix A. A bulk sample of existing subgrade soil retrieved from a boring drilled in an area of future pavement construction was tested for Resistance value (R-value) in the laboratory. Results of the R-value test are presented in Appendix A.

Corrosivity tests were also performed on two samples of the near surface soils at the site by ConCeCo Engineering, Inc. Samples retrieved from one boring and one exploratory test pit which encountered conditions representative of the site were tested to measure their electrical resistivity, redox potential, and pH. The sulfide, chloride, sulfate and ammonia content of the soil samples were also measured. Samples tested were retrieved from Boring H-1 at a depth of 3.5 feet, and Test Pit T-5 at a depth of 0.75 to 5 feet. Results of the tests are presented in Appendix B, along with an evaluation of the corrosivity of the near surface soils at the site.

4.0 SITE AND SUBSURFACE CONDITIONS

4.1 SITE CONDITIONS

4.1.1 Surface Conditions

The proposed hotel site is located at the south end of an existing asphalt concrete paved parking lot southeast of the Market Place in Emeryville, California. The site is relatively level and ranges from approximately elevation 7.5 feet in the northwest portion of the property, to about elevation 9.7 feet at the southeast corner of the property. Several landscaping islands are present in the existing parking area, as well as light poles. The eastern side of the site is currently being used as a construction laydown area and some debris and soil stockpiles are present in this area. A line of concrete highway barriers approximately 75 feet west of the eastern property line, separates the construction yard area from the rest of the site.

4.1.2 Historical Site Development

The historical bay shoreline was located approximately 100 to 200 feet west of the proposed hotel site. The majority of the marshland and tidal flats east of the historical bay shoreline was filled by 1910. Additional fill was placed during grading for the existing parking lot.

The Market Place site was once occupied by a large manufacturing plant. Construction of the industrial facility began in the late 1800's, and was essentially complete by 1930. By the late 1950's, several additions to the plant had been constructed. The plant was disassembled and all but two of the buildings were removed between the early 1960's and about 1974. Information regarding previous construction at the site indicates that several of the former buildings were located adjacent to or within the limits of the proposed hotel. One or more of these buildings may have had basements. The approximate locations of the previously demolished structures are shown on Figure 1. Two of the brick industrial buildings were remodeled for commercial use in 1988 and are now the Market Place. The surrounding area was graded and paved to its present elevations in 1988 to provide parking for the commercial buildings.

4.2 SUBSURFACE CONDITIONS

4.2.1 Soil Conditions

Subsurface conditions at the proposed hotel site consist of approximately 3 to 9 feet of fill overlying 4 to 10 feet of bay sediments. The fill is generally comprised of a heterogeneous mixture of clay and sand with debris including wood, concrete and miscellaneous building materials. Bay sediments underlying the fill consist of soft to medium stiff and stiff silty and sandy clays. The combined thickness of fill and sediments encountered in explorations performed within the limits of the proposed hotel varies from 7 to 14 feet.

Stiff to very stiff silty and sandy clay with occasional layers of silty and clayey sand was encountered to a depth of 50 feet in Boring B2 and about 65 feet of Borings B1, H1 and H2. Very stiff to hard silty and sandy clay interbedded with layers of dense to very dense clayey gravel and sand with gravel was encountered below these materials to the depth of completion of the borings at about 91 feet. In Boring B2, the soil below a depth of 50 feet was more granular than encountered in the other three borings.

Groundwater was encountered at depths of 4 to 5.5 feet in several of the exploratory test pits excavated at the proposed hotel site during the current study and during the study performed at the site in 1988. Groundwater was encountered at depths of 5 to 8.5 feet in four of the borings drilled on the site during 1988. Circulation of drilling fluids in the remainder of the soil borings drilled on the site during the current study and during 1988 prevented the measurement of additional water levels.

4.2.2 Obstructions Encountered

Exploratory test pits excavated during the current study and during previous studies encountered concrete obstructions in areas of proposed hotel construction. Concrete obstructions were also encountered in exploratory pits and borings excavated at the site during explorations undertaken in 1987 and 1988. Obstructions encountered include concrete slabs and foundations, abandoned utility lines, and wood debris. Obstructions encountered within the limits of new building construction include concrete slabs or foundations along the east wall of the proposed elevator tower and the north and west walls of the hotel, and at four locations beneath the proposed building. The obstructions are typically present at depths of ½ to 2 feet. A list of the locations where obstructions were encountered at the site is provided in Table 1. Descriptions of the obstructions and depths at which they were encountered are also given in the table. Locations where concrete obstructions and buried utility lines were encountered in exploratory pits are shown on Figure 1.

Existing 10- and 12-inch diameter storm drain pipes provide drainage of the parking lot. The locations of the existing storm drains are shown on Figure 1. The 10-inch diameter pipe passes beneath the proposed hotel building. Active irrigation and electrical street lighting lines are also present at the hotel site.

5.0 GEOTECHNICAL ASSESSMENT OF SITE

5.1 FOUNDATIONS

5.1.1 Proposed Hotel Building

The fill and bay sediments at the site are loose, soft and compressible and have only moderate strength and are not considered suitable for support of the proposed 12-story hotel on shallow foundations. The thickness of these materials varies across the site, and this variation could result in significant differential settlement beneath the planned building. It is recommended that the columns and walls supporting the 12-story hotel building be supported on driven pile foundations extending through the mixed fill and soft bay sediments and into the underlying firm soils. To preclude excessive differential settlement which will adversely affect the performance and appearance of the ground-level concrete floor slab, the slab should also be supported on driven pile foundations.

Prior to driving piles, it will be necessary to predrill holes through the fill at each pile location to clear obstructions. Overexcavation and removal of existing foundations or portions of concrete slabs that cannot be removed during predrilling will be required. Additional removal of buried structures also may be required if obstructions are encountered during the excavation for the pile caps.

Pile foundations at the site will develop primarily skin friction support in the firm bearing soils underlying the bay sediments. Increased driving resistance is anticipated below a depth of 65 feet over most of the site as the clays become very stiff and the strata of dense granular soil become thicker. In the vicinity of Boring B2, piles may encounter substantial end bearing support in the thick strata of dense granular materials encountered below a depth of 50 feet.

We recommend that an indicator pile program be implemented at the site to better estimate the variation in pile lengths across the building site prior to casting and installing the pile foundations. The design length of piles may vary from 55 to 65 feet, depending on the results of the indicator pile program. Further identification of the location of existing obstructions beneath the site can also be provided during the predrilling and excavation of pile locations during an indicator pile program.

5.1.2 Light Ancillary Structures

Support of light ancillary structures planned for construction around the perimeter of the proposed 12-story building can be provided on shallow spread footing foundations bearing on a layer of compacted select fill, provided that the small structures can tolerate some settlement. However, light structures which are connected to the pile-supported hotel building, such as the planned meeting room, should be supported on driven pile foundations to prevent differential settlement between adjacent building elements that would be supported on different types of foundations. Provided that connections between the 12-story building and adjacent roof structures over covered driveways such as the Porta Cochere can be designed to tolerate some differential settlement, columns supporting the light roofs can be supported on shallow foundations bearing on compacted select fill. We also recommend that entrance ramps and utility connections be designed to accommodate settlement at connections to the pile-supported buildings. A more detailed discussion of settlement at the site is presented below.

5.2 SETTLEMENT

Settlement of structures supported on shallow foundations and floor slabs supported on grade will result from compression and consolidation of the loose heterogeneous fill and soft bay sediments at the site. Shallow foundations supporting light structures are expected to settle beneath the building loads and the weight of new fill placed to raise site grade. Ancillary one-story buildings connected to the hotel structure are expected to experience significant differential settlement between the pile-supported elements of the main building and the adjacent lightly loaded building columns supported on shallow foundations.

The post-construction settlement of the meeting room connected to the hotel structure is estimated to be less than 1½ inches. The differential settlement between the perimeter wall of the meeting room and the hotel structure should be considered equal to the total settlement of the meeting room.

Shallow foundations supporting the separate exercise room structure are expected to settle less than 1 inch. Differential settlement between adjacent load bearing elements is not expected to exceed ½ inch.

Settlement of pile foundations supporting the hotel structure is expected to be less than ½ inch. Differential settlement between adjacent load bearing walls should also be less than ½ inch.

Floor live loads combined with the weight of new fill placed to raise site grades are expected to cause some settlement of the building floor slabs if they are supported directly on grade. As a result of variations in thickness and compressibility of fill and bay sediments at the site, settlement will be uneven across the floor slab. In addition, uneven settlement will occur if the ground floor slab in the pile supported hotel building is supported on grade. Specifically, it is estimated that up to 1 inch of settlement could occur if the ground floor in the hotel is constructed at elevation 10.4 feet. Differential settlement across a slab-on-grade floor in the pile-supported hotel structure could essentially equal the total settlement in magnitude. In view of the relatively poor subsurface conditions at the site and the importance of maintaining a level floor for aesthetic appearances and for maintenance considerations, it its recommended that the first floor of the hotel building be a structural floor system supported on pile foundations.

Uneven settlement of the site should be accounted for in design of the surface drainage system, utility lines leading into the pile-supported building, and entrances to the building. Utility line connections leading into the building may need to be flexible to accommodate settlement of the ground relative to the pile-supported building. Also, if the utility lines are suspended from the concrete floor slab, the hangers should be designed to support the weight of pipe and soil backfill over the pipe. Use of loose sand backfill over the suspended pipelines will reduce the earth loads on the pipeline hangers.

At building entrances, one end of concrete approach slabs should be supported on the pile-supported building and the other end on grade. The approach slabs should be designed to span between the two ends of the walkway.

6.0 CORROSIVITY EVALUATION

Based on the results of laboratory tests performed on near-surface soils by ConCeCo Engineering (ConCeCo), the existing fill at the proposed hotel site is considered to be "very corrosive" to concrete, buried steel, iron, and copper structures, and concrete encased steel. Protective measures recommended by ConCeCo include cathodic protection of all buried steel, iron and copper pipe at the site. Steel reinforcement in concrete should be protected by concrete cover greater than 3 inches thick. The sulfate content measured in one soil sample was at the upper limit for Type II cement (0.20 percent). ConCeCo recommends using Type V cement, but the standard practice in the Emeryville area is to use Type II cement. More detailed recommendations, including encasement of iron pipe and provision of insulating elements between above and below grade piping, are provided in the Corrosion Investigation report prepared by ConCeCo dated May 7, 1997 and included in Appendix B of this report.

7.0 RECOMMENDATIONS

7.1 FOUNDATIONS

7.1.1 Pile Foundations

On the basis of subsurface conditions encountered at the site and the anticipated structural loads for the hotel building, it is recommended that the structure and the ground level concrete floor slab be supported on concrete pile foundations. In our opinion, 12-inch square, prestressed concrete piles are appropriate for support of the planned structure.

Piles in groups should be spaced at least 4 feet apart, measured from the centers of adjacent piles. A minimum group of two piles should be used to support individual column loads. However, a single line of piles may be used to support load bearing walls. Piles subject to transient uplift loads should be adequately tied into the pile cap using either the pile prestressing strands or reinforcing steel dowels. Specific foundation recommendations and design criteria for vertical and lateral load resistance of piles are given below.

7.1.1.1 Vertical Resistance. The vertical load capacity of 12-inch-square prestressed concrete piles is given on the design curve shown on Figure 2. To resist building seismic forces, the pile capacity values shown on the curves provided in Figure 2 can be increased 33 percent to resist downward transient (wind or seismic) loads. The capacity of piles to resist transient uplift loads is the same as the capacity of piles to resist dead and live compression loads. The capacity of 12-inch square prestressed concrete piles is usually limited to 100 tons even though the structural capacity of the piles is slightly higher. Shorter, lower capacity piles may be used to support the structural floor slab and the load-bearing elements of the meeting room structure.

The pile design curve for compression loads is based on developing skin frictional resistance below a depth of 15 feet from existing grade. In addition, some end bearing support is expected to develop below a depth of 65 feet. The depth shown on the pile capacity graph is measured from the existing grade. In determining the required pile lengths, the depth of the pile cap can be deducted from the lengths shown.

Because finished grade will be raised two or more feet above existing grade, sufficient settlement is expected to occur to impose downdrag loads on the foundation piles. It is recommended that the following downdrag loads be added to the structural loads to be resisted by each pile:

Thickness of New Fill (feet)	Design Downdrag Loads (kips)
0	0
2	10
4	12

7.1.1.2 Lateral Load Resistance. Transient lateral loads can be resisted by passive earth pressure acting against the sides of pile caps and grade beams. For design purposes, a passive earth pressure equal to a fluid weighing 400 pounds per cubic foot (pcf) is recommended for use against the face of the pile caps or grade beams which are in direct contact with the soil. Resistance to lateral loads can also be developed by adhesion between soil and the sides of grade beams oriented in the direction of load. A uniform adhesion value of 400 pounds per square foot (psf) acting on the sides of the grade beams can be used to resist lateral loads. Adhesion along the bottom of pile supported grade beams should be neglected, since any settlement of the fill would reduce or eliminate soil adhesion on the bottom of grade beams. If additional lateral resistance is required, the lateral load capacity of foundation piles is commonly taken into account.

Resistance to lateral seismic loads can be provided by bending of the foundation piles. To estimate the magnitude of load that a single 12-inch-square, prestressed concrete pile can take in bending, the computer program LPILE, which takes the nonlinear behavior of soil into account, was used. The lateral load resistance of piles increases with increasing deflection of the pile. For purposes of this analysis, the lateral load causing ½ inch deflection of the pile head for both free head and fixed head conditions was calculated. Increased lateral resistance can be developed if greater pile deflection is allowed. However, ½ inch seems reasonable for short-term loading associated with wind or seismic forces. Results of the analysis are as follows:

Pile Head Condition	Lateral Load (kips)	Maximum Bending Moment (inch-kips)
Free	11	400
Fixed	21	1000

The above lateral load capacity values are for a single pile. Because of interaction between adjacent piles, the capacity of pile groups to resist lateral loads is less than the sum of the capacity of individual piles. Accordingly, the lateral resistance of piles in groups should be reduced, depending on the spacing between adjacent piles. Reduction factors for lateral resistance of piles in groups are given below:

Spacing Between Piles (feet)	Reduction Factor on Single Pile Capacity (percent)
4	60
6	80
8	100

7.1.1.3 Indicator Piles. To better evaluate variations in pile lengths across the building site caused by variations in depth and thickness of granular strata, it is recommended that an indicator pile program be performed at the site prior to casting piles for production pile driving. Based on the size of the building and subsurface conditions encountered in the four deep borings drilled at the site, it is recommended that at least 20 indicator piles be driven at the site. The indicator piles should be located at actual foundation pile locations and spaced to give coverage across the entire building. The indicator piles should be cast 5 feet longer than design length to allow the piles to be driven deeper into the bearing soils, if necessary.

7.1.2 Spread Footing Foundations

Separate, light, one-story structures and walls may be supported on shallow spread footings founded on a pad of compacted select fill. The fill should extend at least 2 feet below the bottom of the footing and 2 feet beyond the edge of footing. Footings bearing on select compacted fill should extend 2 feet below the lowest adjacent finished grade.

Footings meeting the foregoing requirements for bearing on compacted select fill may be designed for the following bearing pressures:

Dead load	2000 psf
Dead plus live loads	2500 psf
All loads, including wind or seismic	3500 psf

Resistance to lateral loads can be provided by friction between the bearing soil and the bottom of spread footing foundations and by passive pressure acting on the face of the foundation pile caps. For frictional resistance, a coefficient of friction of 0.35 between concrete and soil is recommended. If additional lateral resistance is required, a passive pressure equivalent to a fluid weighing 400 pcf can be assumed to act on footings. Resistance offered by the upper 1 foot of soil should be neglected to account for seasonal changes in moisture content and resulting loss of strength.

7.2 LATERAL EARTH PRESSURES

Lateral earth pressures are provided below for low landscaping walls and for design of the proposed swimming pool.

7.2.1 Landscaping Walls

Lateral earth pressures acting on retaining structures are dependent on whether the wall is free to deflect at the top or is restrained and the type of soil backfill placed behind the wall. Since both types of walls may be constructed at the site, design earth pressures have been developed for both wall conditions.

Type of Wall	Lateral Earth Pressure (pcf)
Cantilever	35
Restrained	55

The lateral earth pressure values are given in terms of pounds per cubic foot and are equivalent fluid pressures that increase linearly with depth. These lateral earth pressures were developed assuming that select backfill material is placed and compacted in the space between the back of the retaining wall and an imaginary plane extending up from the heel of the wall footing at a 45 degree angle.

The foregoing design earth pressures also assume no buildup of hydrostatic pressure occurs behind the walls. To prevent the development of hydrostatic pressure behind retaining walls 3 feet or more in height, it is recommended that a subsurface drainage system, consisting of granular filter material and perforated subdrain pipe, be installed. A 12-inch-thick layer of granular filter material should be placed against the wall and extended to within 12 inches of the backfill surface. Compacted soil should be placed over the layer of filter material to minimize infiltration of surface water into the subdrain system. The granular filter material should be a clean, well-graded mixture of sand and gravel meeting the following grading requirements:

Sieve Size	Percentage Passing Sieve
1"	100
¾"	90 - 100
⅜"	40 - 100
No. 4	25 - 40
No. 8	18 - 33
No. 30	5 - 15
No. 50	0 - 7
No. 200	0 - 3

An alternative to graded filter material is to use clean gravel (¾-inch size) with a geotextile placed between the gravel and adjacent clayey soil. The geotextile should be Mirafi 140NC or similar material.

A perforated subdrain pipe should be installed at the bottom of the wall. The pipe should be at least 4 inches in diameter and lead to a free outlet. The perforations should be placed down. The pipe should be surrounded with granular material. Three-inch-diameter weep holes, spaced 8 feet or less on centers, may be used in lieu of perforated subdrain pipe.

7.2.2 Swimming Pool

The swimming pool should be designed to support lateral earth pressures, pressures due to surcharge loading from the adjacent deck slab, and hydrostatic and buoyant pressures below the design elevation for the groundwater table. We have assumed that the pool shell will have a maximum height of approximately 10 feet. Lateral pressures on the pool have been developed for above and below groundwater level and represent at rest pressures because of limited deflection of the pool walls:

Above groundwater	50 pcf
Below groundwater	80 pcf

Pressure due to surcharge loading should be taken as a rectangular distribution over the entire depth of the pool with a value equal to approximately one-third of the surcharge loading. These pressures should be assumed to occur whether the pool is empty or full.

In the vicinity of the proposed pool, groundwater was encountered in the borings at depths ranging from approximately 4 to 8½ feet below the ground surface. Because the local groundwater levels can fluctuate depending on factors such as seasonal rainfall, groundwater withdrawal, irrigation, and construction activities on this or adjacent properties, we recommend that for the design of the pool shell, the depth to groundwater be assumed at 3 feet below the existing ground surface. Because the pool extends below the design elevation of the groundwater table, a system to resist buoyant pressures or a relief valve at the pool bottom will need to be provided to relieve hydrostatic pressures.

The structural floor system in the hotel building is expected to be a reasonably good barrier to moisture migration from the soil into the building. However, positive control of moisture for structural slabs, and in areas where dampness of slabs-on-grade in ancillary buildings would be undesirable, can be obtained by placing a layer of open-graded gravel at least 4 inches thick on the subgrade to form a capillary break. A moisture-proof membrane should be installed over the gravel layer and covered with 2 inches of sand to protect the membrane from damage during construction. The gravel and sand can be considered as the upper 6 inches of select fill under the slabs-on-grade.

7.4 PAVEMENT DESIGN AND CONSTRUCTION

Pavement construction at the Woodfin Suite Hotel site includes a new parking lot, driveways, sidewalks, and patios. Structural design of flexible (asphalt concrete) pavements for the parking area and driveways are provided below. Subgrade reaction coefficients for design of the rigid (Portland cement concrete) pavements, including sidewalks, patios and the pool deck are also provided.

Structural design of flexible pavements is based on the strength of the subgrade soil, strength of the pavement materials, and assessment of vehicle traffic (both vehicle weight and frequency). The Caltrans method of pavement design uses the resistance-value (R-value) test to evaluate the strength of subgrade soil and pavement materials. One sample of the potential subgrade soils within the development was collected to measure its R-value. The sample was collected from Test Pit T-5 along the northeast perimeter of the planned hotel building. An R-value of 35 was measured on the composite sample of granular fill and underlying silty clay. The sample tested was comprised of soils that are representative of typical subgrade conditions at the site. The measured R-value was used for design of new asphalt concrete pavements at the proposed hotel.

The Traffic Index (T.I.) is used to designate the volume of traffic and weight of vehicles expected to travel on the new pavement. The T.I. is usually based on an estimated traffic volume projected over the economic life of the pavement (usually 20 years) and the expected mix of cars and trucks. Traffic index values have not yet been developed for the new parking areas. However, to provide information for evaluating final pavement sections, the following structural asphalt concrete pavement sections have been prepared for parking areas and driveways for a range of Traffic Index values:

	Pavement Component Thickness (feet)		
Traffic Index (T.I.)	Asphalt Concrete	Class 2 Aggregate Base	Class 2 Aggregate Subbase
4	0.35	----	----
	0.20	0.50	----
5	0.45	----	----
	0.20	0.50	----
6	0.25	0.60	----
	0.25	0.55	0.35

A traffic index of 5 or more is suggested to provide a more durable pavement. A traffic index of 5 or 6 should be used where truck deliveries are made and where traffic is heavy.

Concrete pavements can be designed using a coefficient of subgrade reaction of 150 pounds per cubic inch, and should be underlain by a minimum of 6 inches of aggregate base. Concrete pavements which are not subject to vehicular traffic such as exterior sidewalks should be underlain by at least 4 inches of compacted select fill placed on a prepared subgrade. It is recommended that concrete slabs at large patio and decorative areas be supported on at least 8 inches of compacted select fill to further reduce the potential for slab cracking and movement. As recommended above, the pool deck should be underlain by a minimum of 12 inches of compacted select fill.

The pavement materials should conform to the following sections of the Caltrans Standard Specifications, latest edition:

Pavement Material	Type of Material	Standard Specification
Asphalt Concrete	Type B	39
Aggregate	½ inch maximum, medium gradation	39
Asphalt	AR 4000	92
Aggregate Base	Class 2, ¾ inch gradation	26
Aggregate Subbase	Class 2	25
Portland Cement Concrete Pavement	Portland Cement Concrete Pavement	40

The upper 6 inches of subgrade soil and the aggregate base and any subbase materials beneath pavement subject to vehicular traffic should be compacted to a minimum of 95 percent compaction as determined by ASTM D1557. Subgrade beneath sidewalks and patios should be compacted to at least 90 percent compaction. In addition, the subgrade soil should be compacted at a moisture content at least 1 percent above optimum. Soft or wet areas should be subexcavated to firm soil.

The fill material that underlies the new parking and roadway areas is heterogeneous and contains some debris. In general, the fill becomes less dense and weaker with depth. To provide a firm subgrade for new pavements, it is recommended that new grades not be lowered significantly below existing pavement grades. If grades are lowered, there is the possibility that the deeper subgrade soil will be soft and pumping. To provide a stable subgrade for pavement construction, it may be necessary to subexcavate 1 to 2 feet of soft soil, place a strong geotextile and backfill with crushed gravel. More specific recommendations for stabilizing soft subgrade conditions are given in Section 7.6.1.

7.5 EARTHWORK

All existing pavement, curbs and landscaping islands which will not be incorporated in new site development should be removed from the planned construction area. Concrete and asphalt surfacing may be either hauled off site to a suitable disposal area or pulverized and reused for pavement construction or for fill. Vegetation which is stripped and soil removed from landscaping areas may be stockpiled and reused in future planting areas.

All active or inactive utilities within the building area should be relocated or abandoned, or appropriate protective measures should be taken to avoid damage to the utilities or accidents that could result in injury. Pipelines to be abandoned in place should be filled with a sand-cement slurry in exterior areas. Pipes under the pile supported building can be capped but do not need to be filled. If existing utilities are removed, the resulting excavation should be backfilled with well compacted fill.

Existing foundations and concrete slabs present beneath the hotel building and ancillary structures site should be removed from the limits of new underground construction when encountered during predrilling for new pile foundations and excavating for pile caps and footing foundations and the swimming pool. All excavations resulting from the removal of buried obstructions should be backfilled with compacted fill.

After the site has been cleared, the building and pavement areas should be brought to grade by excavating and placing and compacting fill. Soil will be excavated to construct new foundations and the swimming pool. Areas to receive fill should be scarified, moisture conditioned to at least 1 percent above optimum moisture content, and compacted to the requirements for fill presented below in this section of the report.

Soils to depths of 5 to 8 feet are a heterogeneous mixture of granular materials and debris. The granular soil can be used as fill providing the debris and organic materials are removed prior to compaction. Extra effort will be required to remove debris when soil from onsite excavations are used for fill. The underlying soft bay sediments are clayey and wet and difficult to compact and are not suitable for use as fill. All fill material should be a soil or soil-rock mixture free of organic material, debris, and other deleterious substances. The soil should contain no rocks larger than 4 inches in greatest dimension nor more than 15 percent larger than 2½ inches. All imported soil should be a select material meeting the forgoing requirements for general fill as well as the following quality requirements:

Maximum plasticity index	15
Percentage passing No. 200 sieve	50 maximum, 10 minimum

The requirement that at least 10 percent pass the number 200 sieve is to preclude the use of sand or gravel as select fill. All fill and backfill materials should be observed and tested by the geotechnical engineer prior to use in order to evaluate their suitability.

All fill and backfill necessary to bring the site to grade, backfill excavations, or to support foundations should be placed in uniform lifts not exceeding 8 inches in uncompacted thickness. Each lift should be brought to a uniform moisture content prior to compacting by either spraying the soil with water if it is too dry or aerating the material if it is too wet. Fill should be compacted to the following degree of compaction as determined by ASTM D-1557:

Fill Location	Degree of Compaction
General site fill	90
Utility trench backfill	90
Fill below foundations and floor slabs	90
Upper 6 inches of fill and backfill below pavements	95

7.6 CONSTRUCTION CONSIDERATIONS

7.6.1 Site Preparation and Grading

Site preparation and fill and backfill placement should be observed by a representative of our firm to observe whether any undesirable material is encountered in the construction area and confirm that the exposed soils are similar to those encountered during the field exploration programs at the site. Site preparation activities to be observed include site excavation and scarification and compaction of areas to receive fill. Placement and compaction of select fill in building areas also should be observed.

The time of year when earthwork is undertaken will greatly influence the time and effort required to complete the work. Site preparation and grading will be difficult during winter or early spring when surface soils are saturated and wet. Therefore, to minimize delays in the project, the earthwork should be scheduled for late spring, summer, or early fall.

Soft and pumping subgrade conditions may be encountered during site grading for pavements and building areas. The deeper the excavations the greater the potential for encountering soft and wet soils. When soft subgrade conditions are encountered the areas can be ripped, scarified, and reworked to reduce the moisture of the soil. If an isolated area of debris or wet clay is encountered, it would be more direct to subexcavate the soft and wet soil and replace it with dry fill. If time is not available to aerate the soft wet soil, the area can be subexcavated 1 to 2 feet (depending on how soft the soil is), a strong geotextile placed over the area, and the excavation backfilled with crushed gravel or aggregate base.

7.6.2 Pile Installation

The presence of existing obstructions, including concrete foundations and slabs, pipelines, and other debris, requires that each pile location be predrilled to a depth of 15 feet. The diameter of the predrill auger should be 14 inches. Shallow footings and slabs such as those encountered in Test Pits 2, 4, 5, 6, 7, and 9 should be removed from the area of new foundation construction. If removal extends outside or below the limits of new construction, these portions of the excavation should be backfilled with compacted select fill.

The approach used to install piles at similar sites where obstructions and debris were present in the fill has been as follows:

1. Predrill each pile location using heavy duty drilling equipment.
2. Install and drive piles at all successful predrilled locations.
3. At locations where refusal to predrilling is encountered, an excavator is used to remove the obstruction. After the obstruction is removed, the pile is set in the excavation and

driven and the excavation backfilled. An alternative is to backfill the excavation prior to setting and driving the pile.

An alternative approach would be to subexcavate the building site to a depth of 4 feet and remove all the concrete and debris encountered. Predrilling would still be required, but most of the obstructions would have been removed by the excavation. A few deeper obstructions may still be encountered, but it appears that most of the obstructions encountered in the excavations performed at the site were relatively shallow.

The pile contractor should select a hammer that is capable of driving the piles to their design tip elevations without overstressing the concrete in either compression or tension. It is recommended that the piles be driven with a hammer having a rated energy of at least 50,000 foot-pounds.

Preliminary pile driving criteria, consisting of minimum and refusal blow counts, have been developed for two different hammer energies. The criteria are intended to be used as a guide for driving the indicator piles. The driving criteria should be reviewed and modified as necessary after the indicator pile program has been completed and before production pile driving begins.

Rate Hammer Energy (foot-pounds)	Pile Capacity (tons)	Minimum Blow Count (blows/foot)	Refusal Blow Count (blows/foots)
50,000	50	12	40
	100	25	75
70,000	50	10	30
	100	20	60

The general driving criteria for installation of piles are as follows:

1. Drive piles to their design tip elevation.

2. If driving resistance is below the minimum blow count, continue driving the pile until the minimum blow count criteria is met.

3. If hard driving resistance is encountered above the design tip elevation, driving can stop provided that pile tip is within 5 feet of design tip elevation and the driving resistance meets the refusal blow count criteria.

The installation of indicator and production pile foundations should be observed by a representative of our firm to compare driving conditions encountered with those revealed by the exploratory borings drilled at the site. Based on the results of the indicator pile program, final driving criteria will be developed for installation of the foundation piles at the building site.

7.6.3 Footing Foundations

Excavations for spread-footing foundations should be observed by the geotechnical engineer prior to placement of reinforcing steel and concrete to confirm that the bearing soils are firm and consistent with conditions encountered in the exploratory borings. If loose or soft soils are exposed in any of the excavations, the footings should be deepened or the loose or soft soils excavated and replaced with lean concrete or compacted select fill or gravel.

7.7 BASIS FOR RECOMMENDATIONS

The recommendations made in this report are based on the assumption that the soil conditions do not deviate appreciably from those disclosed in the two deep exploratory borings drilled at the Woodfin Suite Hotel site during the current study, and from those encountered in the two deep and 14 shallow exploratory borings drilled at the site during previous studies. If any variations or undesirable conditions are encountered during construction, the effects of these conditions on the recommendations presented herein should be evaluated and, if necessary, supplemental recommendations developed. The recommendations are also made for the proposed Woodfin Suite Hotel project described in this report. Significant changes in location, type of structures, or loading conditions should be evaluated as to their effects on the recommendations.

It is recommended that we review the foundation and grading plans and specifications to determine that the intent of the recommendations presented herein has been properly interpreted and incorporated into the contract documents. In addition, a representative of our firm should observe the site grading and foundation excavations and installation of driven piles to verify that the subsurface conditions used as a basis for the recommendations are encountered throughout the site.

GEOMATRIX

TABLE 1
SUMMARY OF OBSTRUCTIONS ENCOUNTERED IN BORINGS AND TEST PITS
WOODFIN SUITE HOTEL
Emeryville, California

Boring/Test Pit No.[1]	Depth of Boring/Test Pit (ft)	Obstruction Encountered	Depth to Obstruction (ft)	Thickness of Obstruction (ft)	Description
T-1	7	No	---	---	---
T-2	0.5	Yes	0.5	[3]	Concrete
T-3	7	No	---	---	---
T-4	6.5	Yes	1	1	1-ft.-wide concrete strip in north end of pit
T-5	6	No[4]	---	---	---
T-6	1.3	Yes	1.3	[3]	Concrete
T-7	1.5	Yes	1.5	[3]	Reinforced Concrete
T-8	1.25	No[5]	---	---	---
T-9	1.25	Yes	1.25	[3]	Concrete
T-10	6.5	No[4]	---	---	---
T-11	6.5	No	---	---	---
H-1	91.5	No	---	---	---
H-2	86.5	No	---	---	---
TP#1	9	Yes	2	2-3	Reinforced concrete footings
TP#1	9	Yes	2	9+	15-ft.-diameter concrete tank
TP#1	9	Yes	2	9+	1-ft.-wide concrete wall, ties into concrete wall encountered in TP #3 & TP #4
TP#2	9	Yes	2	0.5	Concrete slab
TP#3 TP#4	10	Yes	2	9+	Concrete wall, varies from 0.5-feet-wide at top to 2-ft.-wide at depth of 4 feet.

TABLE 1
SUMMARY OF OBSTRUCTIONS ENCOUNTERED IN BORINGS AND TEST PITS
WOODFIN SUITE HOTEL
Emeryville, California
(continued)

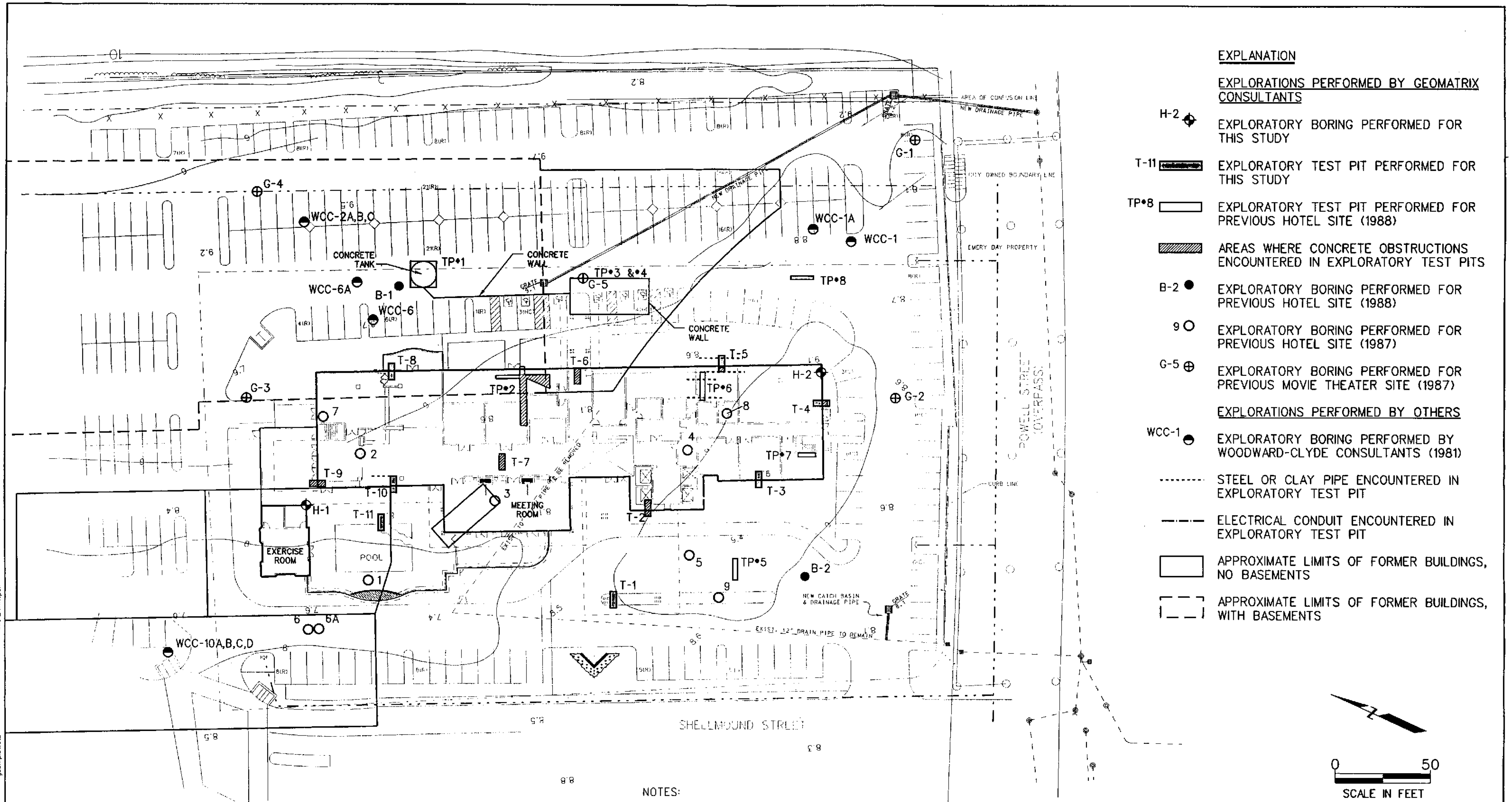
Boring/Test Pit No.[1]	Depth of Boring/Test Pit (ft)	Obstruction Encountered	Depth to Obstruction (ft)	Thickness of Obstruction (ft)	Description
TP#5	10	No	---	---	---
TP#6	5	No[4]	---	---	---
TP#7	8.5	No	---	---	---
TP#8	8	No	---	---	---
G-1	31.5	No	---	---	---
G-2	21.5	No	---	---	---
G-3	31	Yes	2	1	Concrete
G-4	21.5	No	---	---	
G-5	12.5	Yes	13	3	Concrete
1	21.5	No	---	---	---
2	21.5	No	---	---	---
3	21.5	No	---	---	---
4	21.5	No	---	---	---
5	21.5	Yes	4	1/2	Concrete or brick
6	6	Yes	2.5	>3.5	Wood
6A	20	No	---	---	---
7	20	Yes	2.5	1/2	Concrete
8	20	Yes	5	1	Large rocks
9	20	No	---	---	---
WCC-1	5.5	No	---	---	---
WCC-1A	9	No	---	---	---
WCC-2	11	Yes	6	1/2	Wood

TABLE 1
SUMMARY OF OBSTRUCTIONS ENCOUNTERED IN BORINGS AND TEST PITS
WOODFIN SUITE HOTEL
Emeryville, California
(continued)

GEOMATRIX

Boring/Test Pit No.[1]	Depth of Boring/Test Pit (ft)	Obstruction Encountered	Depth to Obstruction (ft)	Thickness of Obstruction (ft)	Description
WCC-2A, B&C	5.5	Yes	5.5	(3)	Concrete
WCC-6	11	No	---	---	---
WCC-6A	8	No	---	---	---
WCC-10	6.5	Yes(2)	6.5	(3)	Large rocks
WCC-10A	4.5	Yes	4.5	(3)	Unknown
WCC-10B	3.5	Yes	3.5	(3)	Unknown
WCC-10C	10.5	No	---	---	---
WCC-10D	16	No	---	---	---

1 Borings with letter suffix were drilled within a few feet of the initial numbered borings, e.g., borings 2A, 2B, and 2C were drilled within a few feet of Boring 2.
2 Refusal encountered on large rocks (approximately 8" diameter cobbles)
3 Thickness of obstruction not determined.
4 Abandoned utility pipe(s) encountered. See log of test pits for depth encountered and description of utility.
5 Existing utility conduit encountered.



NOTES:

1. Basemap derived from Conceptual Drainage Plan provided by Austin Design Group, April 1997.
2. Locations of exploratory borings and test pits performed by Geomatrix Consultants Inc., are approximate.
3. Locations of former buildings are based on information obtained from historical aerial photographs, Pacific Aerial Survey, 1947 and 1949

SITE AND BORING LOCATION PLAN

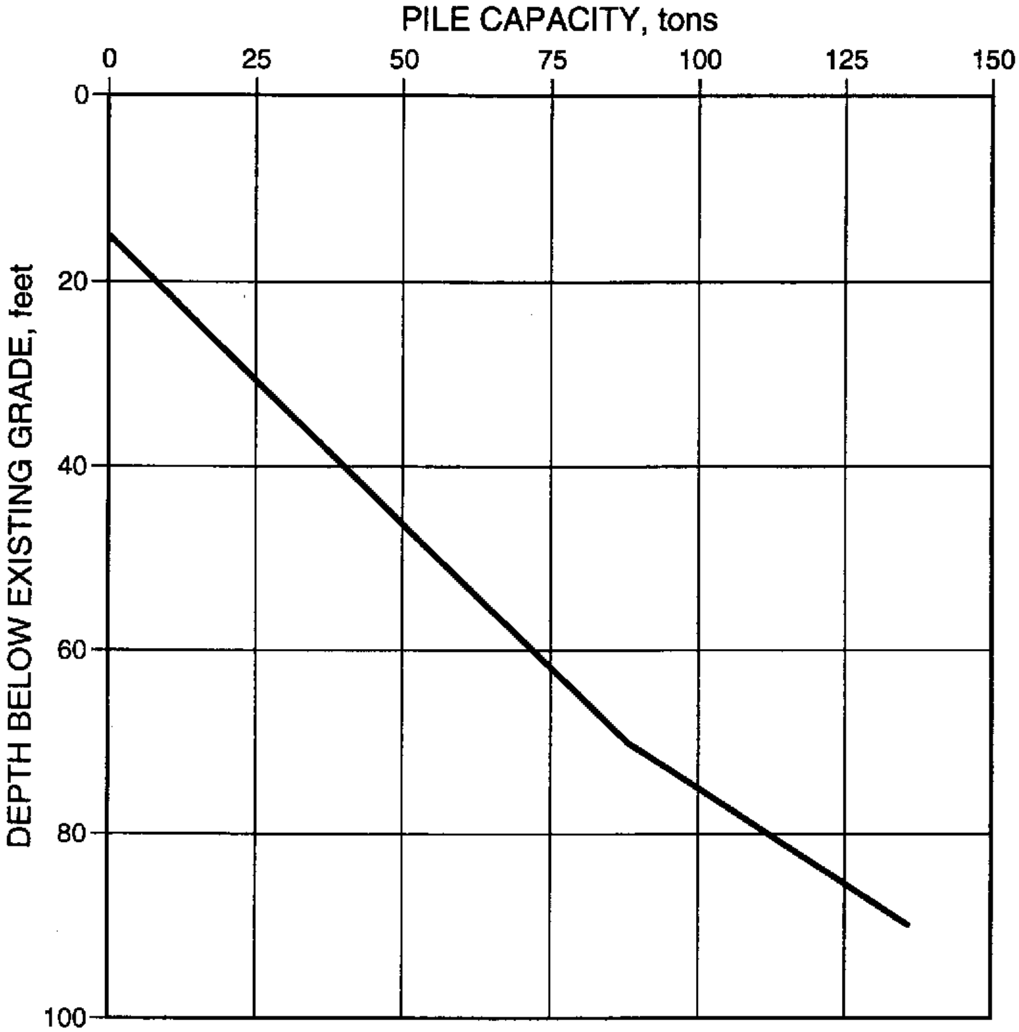
Woodfin Suite Hotel
Emeryville, California

GEOMATRIX

Project No. 4105

Figure 1

23-MAY-1997 12:48 s:\4100s\4105\fig_01.dgn \\PRINTSRV\SF4MV2B masaria geomplot.clb MAP_4mv.pen CHECKED:



Note

Pile capacity is design capacity for 12-inch square prestressed concrete pile for combined dead and live loads.

GEOMATRIX	PILE CAPACITY DESIGN CURVE Woodfin Suite Hotel Emeryville, California	Figure 2 Project No. 4105

APPENDIX A

FIELD EXPLORATION AND LABORATORY TESTING

FIELD EXPLORATION PROGRAM

In accordance with federal law, a site-specific health and safety plan was prepared prior to performance of the field exploration program to identify potential health and safety issues at the site and to outline procedures to protect Geomatrix personnel. This plan was developed based on our review of available environmental data in the vicinity of the project site. The plan was reviewed by all site personnel, including subcontractors, before work began at the site.

Prior to commencing our field exploration program, the necessary permits were obtained from Alameda County Zone 7 for drilling borings in the County, and a business license for drilling borings in the City of Emeryville. We contacted Underground Services Alert (USA) to help locate utilities at the site prior to performing our field exploration program and a private utility locator also was hired to clear existing utility locations in the vicinity of the explorations.

Two borings were drilled and sampled for this study at the locations shown on the site plan, Figure 1. The borings were drilled by Pitcher Drilling of Palo Alto, California, using a truck-mounted rotary wash drill rig between April 11 and 14, 1997. During drilling operations, Geomatrix personnel maintained a record of field activities, classified the soils encountered, and prepared continuous logs of the borings. Soil samples were collected from the borings to aid in characterizing the subsurface conditions and for subsequent geotechnical laboratory testing. Final boring logs were developed from geotechnical laboratory classification and conditions recorded on the field logs and are presented on logs of borings included in Figures A-2 and A-3. A boring log explanation sheet is included on Figure A-1.

Samples of soil were obtained from the borings by either a modified California drive sampler (2-inch ID, 2-1/2-inch OD) or a Standard Penetration Test (SPT) sampler (1.375-inch ID, 2-inch OD). The modified California sampler was lined with thin, segmented brass tubes. This sampler and the SPT sampler were driven into the soil with a 140-pound hammer falling 30 inches. The samplers were driven 18 inches, in most cases, and the blow counts recorded for the final 12 inches of driving, or a portion thereof, are given at the corresponding sample location on the boring logs. The borings were backfilled to the surface with cement-bentonite grout. Asphalt cold patch was used to repair the existing paved surface.

At the completion of the field exploration program, the samplers, hollow stem augers, and rotary wash drill stem were steam cleaned prior to leaving the site. The rinse water was contained in 55-gallon drums stored on site. Rotary wash drilling mud and soil cuttings generated during drilling was also transferred to the drums. At the completion of the drilling operations, the drums were stored on-site at the south end of the site.

Eleven exploratory test pits were excavated at the locations shown on Figure 1 on April 3, 1997, using a small backhoe operated by Ghillotti Brothers. The pits were excavated to a maximum depth of 7 feet, and were terminated at shallower depths when obstructions were encountered. A representative of Geomatrix observed the excavations, and recorded conditions revealed in logs of test pits. Final logs were prepared based on field classifications and descriptions of conditions encountered, and are presented on Figures A-4 through A-14. The excavations were backfilled with the excavated materials, placing the material in lifts and using the backhoe bucket to compact each lift. Each exploratory pit was wheel-rolled using the backhoe once the pit was backfilled to the level of the surrounding ground surface. A conduit for electrical lines servicing the existing parking lot lighting at the site was encountered at a depth of 15 inches in Test Pit T-8. The conduit was damaged by the backhoe at this location, and the encased electrical wiring was pulled. Repair of the active lines was provided by St. Francis Electric Company.

LABORATORY TESTING PROGRAM

Laboratory testing was performed on selected soil samples recovered from the borings. Tests performed included moisture content, unit weight, grain size distribution, and unconfined compression. Measurement of resistance values (R-values) also was performed on one bulk soil sample obtained from Test Pit T-5.

Laboratory testing was performed by Geomatrix in our laboratory in Oakland, California, and by Cooper Testing Laboratory (Cooper) in Mountainview, California. Measurement of moisture content, unit weight, grain size distribution and unconfined compressive strength was performed by Geomatrix , and the R-value measurement was performed by Cooper.

Samples of near surface soil obtained in Boring H-1 at a depth of 3.5 feet, and Test Pit T-5 at a depth of 0.75 to 5 feet were tested by ConCeCo Engineering of Concord, California, to evaluate the corrosivity of the site soils. Tests included electrical resistivity, redox potential, and pH. The sulfide, chloride, sulfate, and ammonia content of the soils also was measured. Results of the tests are presented in Appendix D.

Moisture Content and Unit Weight

Measurement of the moisture content, unit weight, and dry density was performed on 21 representative samples recovered from the borings. These tests were conducted in accordance with ASTM Test Methods D-2216 and D-2850. Results of the moisture content and dry density measurements are presented at the corresponding sample locations on the logs of borings included as Figures A-2 and A-3.

Grain-size Distribution
Two particle size analyses were performed on representative samples to determine their grain size distribution in accordance with ASTM Test Method D-422. The results of the sieve analyses are presented on Figure A-15.

Unconfined Compression Tests
The unconfined compressive strength of cohesive soils on the site was measured on 15 relatively undisturbed samples. These tests were performed in accordance with ASTM Test Method D-2166. The results of these tests, along with moisture content and dry density, are presented on the boring logs at the corresponding sample locations in the logs of borings included in Figures A-2 and A-3.

R-Value Measurement
The resistance value (R-value) measurement of existing subgrade soils was performed on a bulk sample collected from 0.75 to 5 feet in Test Pit T-5 in accordance with the State of California Department of Transportation Test Method 301. A resistance value of 35 was measured on the soil sample tested.

PROJECT: WOODFIN SUITE HOTEL
Emeryville, California

Boring Log and Test Pit Explanation

DEPTH (feet)	Sample No.	Sample	Blows/Foot	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	Other
				Standard penetration split spoon drive sampler, 2-inch outside diameter, 1 3/8-inch inside diameter (without liners)			
				Modified California drive sampler, 2 1/2-inch outside diameter, 2.0-inch inside diameter (with liners)			
5				Modified California drive sampler, 3-inch outside diameter, 2 1/2-inch inside diameter (with liners)			
				Bulk sample collected from test pit excavation			
			23	Blow count for last 12 inches of sample, or as noted			
10							
				Distinct contact			
15				Gradational or uncertain contact			
				First groundwater encountered during drilling or excavating			
				Measured groundwater after completion			
20							
				Unconfined Compressive Strength (psf)			UC=1300
				Grain size distribution test			Sieve
25				Photoionization Detector Reading (ppm)			PID=0
				Resistance Value (California Test-301)			R-Value=35
				Testing for Corrosivity Evaluation			Corr
30							
35							
40							

Notes

1. The stratification lines shown on the boring logs represent the approximate boundaries between material types. The actual transitions between materials may be gradual.
2. These logs of the test borings and related information depict subsurface conditions only at the specific locations and at the particular time the boring was made. Soil conditions at other locations may differ from conditions occurring at these locations. Also, the passage of time may result in a change in the soil and groundwater conditions at these locations.

GT Explanation (03/97)

Log of Boring No. H-1

PROJECT: WOODFIN SUITE HOTEL Emeryville, California	**Log of Boring No. H-1**
BORING LOCATION: See Figure 1, Site and Boring Location Plan	ELEVATION AND DATUM: Approximately 8.5 feet
DRILLING CONTRACTOR: Pitcher Drilling	DATE STARTED: 4/11/97 / DATE FINISHED: 4/11/97
DRILLING EQUIPMENT: Failing 1500	TOTAL DEPTH: 91.5 feet / MEASURING POINT: Top of pavement
DRILLING METHOD: Rotary wash	DEPTH WHERE FREE WATER FIRST ENCOUNTERED: No water encountered
SAMPLING METHOD: See Boring Log Explanation, Figure A-1	DEPTH TO WATER AFTER COMPLETION (Date/Time): ---
HAMMER WEIGHT: 140 pounds / DROP: 30 inches	LOGGED BY: A. Blanc

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Blows/ Foot	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content (%)	LABORATORY TESTS: Dry Density (pcf)	LABORATORY TESTS: Other
			3 inches asphalt concrete over 9 inches aggregate base			
			6 inches concrete			
	1	9	SILTY SAND with GRAVEL (SM) Loose, very dark grayish brown, moist, fine to coarse sand, fine gravel, with debris and trash [FILL]			PID=0
5	2	3	Becoming gravel with silt (GP-GM)	39	84	PID=0.6 UC=260
			SILTY CLAY with SAND (CL) Soft, very dark gray, moist, fine sand			
			SILTY SAND (SM) With shell fragments			
10	3	35	CLAY (CH) Soft to medium stiff, dark greenish gray, moist			
			GRAVELLY CLAY with SAND (CL) Hard, dark greenish gray, fine to coarse gravel, angular to subrounded			
15	4	27	Interbedded GRAVELLY CLAY (CL), SANDY CLAY with GRAVEL (CL) and CLAYEY SAND with GRAVEL (SC) Very stiff, medium dense to dense, mottled olive gray and dark yellowish brown			
20	5	22	CLAY (CL) Very stiff, yellowish brown, moist, trace sand	25	103	UC=4130
25	6	18	SANDY CLAY (CL) Very stiff, greenish gray, moist, fine sand Lens of gravel	23	104	UC=1880
30	7	19	Interbedded SILTY SAND (SM) and SANDY SILT (ML) Medium dense, mottled greenish gray and dark yellowish brown, moist Lens of fine gravel	27	100	UC=3010
35						

GT-1 (03/97)

PROJECT: WOODFIN SUITE HOTEL
Emeryville, California

Log of Boring No. H-1 (cont.)

DEPTH (feet)	Sample No.	Sample	Blows/ Foot	Description	Moisture Content (%)	Dry Density (pcf)	Other
				Interbedded SILTY SAND (SM) and SANDY SILT (ML) (continued)			
				CLAY (CH) Hard, dark greenish gray, moist			
40	8		33		22	108	PID=0.4 UC=6880
				Increased sand			
				SILTY CLAY (CL) Soft to medium stiff, grayish brown, moist			
45							
				CLAY (CH) Soft to medium stiff, dark greenish gray, moist			
50	9		12		43	77	UC=2010
				SANDY SILT (ML) Medium dense, dark blue gray, moist, fine sand			
				SILTY CLAY (CH) Medium stiff, olive gray, moist			
55							
				SANDY CLAY (CL) Stiff, mottled olive gray and dark yellowish brown, moist			
				Becoming very stiff			
60	10		21	CLAY (CH) Very stiff, mottled olive gray and greenish gray, moist	21	109	UC=6260
65							
				SANDY CLAY (CL) Hard, mottled olive gray and dark yellowish brown, moist, fine to coarse sand, few fine gravel			
70	11		45	Sand becoming less coarse	17	117	UC=6330
				CLAYEY SAND with GRAVEL (SC) Medium dense, light olive gray to olive gray, moist, fine to coarse sand, fine gravel			
75							

GT-2 (02/97)

PROJECT: WOODFIN SUITE HOTEL
Emeryville, California

Log of Boring No. H-1 (cont.)

DEPTH (feet)	Sample No.	Sample	Blows/ Foot	Description	Moisture Content (%)	Dry Density (pcf)	Other
				CLAYEY SAND with GRAVEL (SC) (continued)			
				SILTY CLAY (CL) Very stiff, olive, moist			
80	12		29		22	106	UC=5900
				SILTY CLAY (CL) Very stiff, olive, moist			
85				Becoming sandy			
				CLAYEY SAND with GRAVEL (SC) Dense, dark yellowish brown, moist to wet			
90	13		42				Sieve PID=0
				Bottom of boring at 91.5 feet.			
95							
100							
105							
110							
115							

GT-2 (02/97)

PROJECT: WOODFIN SUITE HOTEL
Emeryville, California

Log of Boring No. H-2

BORING LOCATION: See Figure 1, Site and Boring Location Plan	ELEVATION AND DATUM: Approximately 9.1 feet
DRILLING CONTRACTOR: Pitcher Drilling	DATE STARTED: 4/14/97 / DATE FINISHED: 4/14/97
DRILLING EQUIPMENT: Failing 1500	TOTAL DEPTH: 86.5 feet / MEASURING POINT: Top of pavement
DRILLING METHOD: Rotary wash	DEPTH WHERE FREE WATER FIRST ENCOUNTERED: No water encountered
SAMPLING METHOD: See Boring Log Explanation, Figure A-1	DEPTH TO WATER AFTER COMPLETION (Date/Time): ---
HAMMER WEIGHT: 140 pounds / DROP: 30 inches	LOGGED BY: A. Blanc

DEPTH (feet)	Sample No.	Blows/Foot	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	Other
			3 inches asphalt concrete over 6 inches aggregate base			
			CLAYEY SAND with GRAVEL (SC) Medium dense, dark yellowish brown, moist [FILL]			
	1	8	— Becoming dark greenish gray			PID=0.6
			— Becoming very dark grayish brown to very dark brown			
5	2	10	SAND (SP-SM) and GRAVEL (GP-GM) with SILT Loose, very dark grayish brown, moist [FILL]	26		
	3	18	SANDY CLAY (CL) Soft, black, moist to wet			
10			CLAY (CH) Stiff, dark greenish gray, moist			
	4	39	CLAY with SAND (CH) Stiff, mottled olive gray and dark yellowish brown, moist, fine sand, few fine gravel	28	98	PID=0.3 UC=1030
			— Becomes less plastic			
15	5	14	SAND with CLAY and GRAVEL (SW-SC) Dense, light olive brown, wet, fine to coarse, fine to coarse gravel	20	111	UC=2900
			CLAY with SAND (CL) Stiff, mottled light olive gray and dark yellowish brown, moist			
20	6	17	CLAYEY SAND (SC) Medium dense, olive brown, moist, fine sand	21	110	UC=1400
			— Becomes coarser			
			SILTY CLAY (CL) Stiff to very stiff, light yellowish brown to light olive brown, moist			
25			— Trace sand			
	7	16	SILTY CLAY (CH) Very stiff, olive, moist	19	107	UC=3280
			SILTY CLAY (CL) Stiff, mottled olive and dark yellowish brown, moist			
30	8	18	SANDY CLAY with GRAVEL (CL) Very stiff, mottled olive and dark yellowish brown, moist, fine to medium sand, fine gravel	27	100	UC=2250
			SAND (SW) and GRAVEL (GP) Medium dense, brown, moist to wet, fine to coarse sand, fine gravel, subangular to subrounded			
35						

GT-1 (03/97)

PROJECT: WOODFIN SUITE HOTEL
Emeryville, California

Log of Boring No. H-2 (cont.)

DEPTH (feet)	SAMPLES: Sample No.	Sample	Blows/ Foot		LABORATORY TESTS: Moisture Content (%)	Dry Density (pcf)	Other
				SILTY CLAY (CL) Stiff, dark grayish brown, moist			
				SANDY CLAY (CH) Very stiff, dark greenish gray, moist			
40				Hard			
	9		38	SILTY CLAY (CH) Hard, mottled olive and dark yellowish brown, moist	17	115	UC=4530
				Stiff, gray, moist			
45				SAND (SP) and GRAVEL (GP)			
	10		15	SANDY SILT (ML) Medium dense, mottled olive gray and dark yellowish brown, moist	26		
50				SILTY CLAY (CH) Stiff to very stiff, mottled olive gray and dark yellowish brown, moist			
55	11		17	GRAVEL (GP) and SAND (SP) Medium dense, gray to brown, wet, subrounded, fine to coarse gravel, fine to coarse sand	19		PID=0
				CLAYEY SAND with GRAVEL (SC) Medium dense, olive gray and strong brown, wet, fine to coarse sand, fine gravel, fine			
60				SANDY CLAY (CL) Very stiff, olive gray, moist, fine to medium sand			
				Interbedded Layers of GRAVEL (GP) and CLAY(CL) Medium dense, olive gray and brown			
				SILTY CLAY (CL) Stiff, olive gray, moist			
65	12		50	GRAVEL with CLAY (GP-GC) Very dense, olive gray and brown, wet, fine to coarse, trace sand	18	117	Sieve
				CLAYEY GRAVEL (GC) Dense, light olive brown, moist			
70				CLAYEY SAND with GRAVEL (SC) Dense, dark yellowish brown, moist, fine to coarse sand, fine gravel			
				CLAYEY SAND with GRAVEL (SC) Medium dense, mottled olive gray and dark yellowish brown, moist			
75	13		43	CLAY (CH) Very stiff, olive gray, moist	18		

GT-2 (02/97)

PROJECT: WOODFIN SUITE HOTEL
Emeryville, California

Log of Boring No. H-2 (cont.)

DEPTH (feet)	Sample No.	Sample	Blows/ Foot	Description	Moisture Content (%)	Dry Density (pcf)	Other
				CLAY (CH) (continued)			
				SAND (SP) and GRAVEL (GP)			
80							
				CLAYEY SAND with GRAVEL (SC) Dense, light olive brown, moist, fine to medium, fine gravel			
85	14		35		17	117	UC=5080
				SANDY CLAY (CL) Hard, olive gray, moist, fine to medium sand			
				Bottom of boring at 86.5 feet.			
90							
95							
100							
105							
110							
115							

GT-2 (02/97)

PROJECT: WOODFIN SUITE HOTEL Emeryville, California	# Log of Test Pit No. T-1	
BORING LOCATION: See Figure 1, Site and Boring Location Plan	ELEVATION AND DATUM: Approximately 8.5 feet	
DRILLING CONTRACTOR: Ghilotti Brothers	DATE STARTED: 4/3/97	DATE FINISHED: 4/3/97
DRILLING EQUIPMENT: Backhoe John Deere 710D	TOTAL DEPTH: 7 feet	MEASURING POINT: Top of pavement
DRILLING METHOD: Backhoe pit	DEPTH WHERE FREE WATER FIRST ENCOUNTERED: No water encountered	
SAMPLING METHOD: See Boring Log Explanation, Figure A-1	DEPTH TO WATER AFTER COMPLETION (Date/Time): —	
HAMMER WEIGHT: --- DROP: ---	LOGGED BY: A. Blanc	

DEPTH (feet)	SAMPLES			MATERIAL DESCRIPTION	LABORATORY TESTS		
	Sample No.	Sample	Blows/Foot		Moisture Content (%)	Dry Density (pcf)	Other
				3 inches asphalt concrete over 5 inches aggregate base			
1				5 inches asphalt concrete over 2 inches of yellowish brown clayey sand			
2				SILTY SAND with GRAVEL (SM) Dark greenish gray, fine to coarse sand, fine to coarse gravel and rock fragments [FILL] Brick and wood debris			
3				SANDY CLAY (CL) Black, moist, with debris (plywood, building paper) [FILL]			
4				CLAY (CL) Olive gray, moist			
5				SAND with SILT (SP-SM) Gray, wet, with shell fragments			
6							
7				Bottom of test pit at 7 feet.			

GT-1 (03/97)

PROJECT: WOODFIN SUITE HOTEL Emeryville, California	# Log of Test Pit No. T-2	
BORING LOCATION: See Figure 1, Site and Boring Location Plan	ELEVATION AND DATUM: Approximately 9 feet	
DRILLING CONTRACTOR: Ghilotti Brothers	DATE STARTED: 4/3/97	DATE FINISHED: 4/3/97
DRILLING EQUIPMENT: Backhoe John Deere 710D	TOTAL DEPTH: 6 inches	MEASURING POINT: Top of pavement
DRILLING METHOD: Backhoe pit	DEPTH WHERE FREE WATER FIRST ENCOUNTERED: No water encountered	
SAMPLING METHOD: See Boring Log Explanation, Figure A-1	DEPTH TO WATER AFTER COMPLETION (Date/Time): ---	
HAMMER WEIGHT: --- DROP: ---	LOGGED BY: A. Blanc	

DEPTH (feet)	SAMPLES: Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content (%)	Dry Density (pcf)	Other
				3 inches asphalt concrete over 3 inches aggregate base			
				Bottom of test pit at 6 inches. Concrete slab encountered at base of pit.			
1							
2							
3							
4							
5							
6							
7							

GT-1 (03/97)

Project No. 4105	**Geomatrix Consultants**	Figure A-5

4105.008

PROJECT: WOODFIN SUITE HOTEL Emeryville, California		**Log of Test Pit No. T-3**	
BORING LOCATION: See Figure 1, Site and Boring Location Plan		ELEVATION AND DATUM: Approximately 9.3 feet	
DRILLING CONTRACTOR: Ghilotti Brothers		DATE STARTED: 4/3/97	DATE FINISHED: 4/3/97
DRILLING EQUIPMENT: Backhoe John Deere 710D		TOTAL DEPTH: 7 feet	MEASURING POINT: Top of pavement
DRILLING METHOD: Backhoe pit		DEPTH WHERE FREE WATER FIRST ENCOUNTERED: No water encountered	
SAMPLING METHOD: See Boring Log Explanation, Figure A-1		DEPTH TO WATER AFTER COMPLETION (Date/Time): ---	
HAMMER WEIGHT: ---	DROP: ---	LOGGED BY: A. Blanc	

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/ Foot	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content (%)	LABORATORY TESTS: Dry Density (pcf)	LABORATORY TESTS: Other
				3 inches asphalt concrete over 3 inches aggregate base			
				SAND with GRAVEL Brown, moist, fine to coarse sand, fine to coarse gravel [FILL]			
1				SAND with GRAVEL (SW) Dark greenish gray to dark olive gray, dry, fine to coarse sand, fine to coarse gravel, moderate hydrocarbon odor [FILL]			
2							
3				Layer of debris and wood, shingles			
				SILT (ML) and SILTY SAND (SM) Black, fine sand, oily appearance, strong hydrocarbon odor			
4							
5							
6				CLAY (CH) Olive gray, moist			
7				Bottom of test pit at 7 feet.			

GT-1 (03/97)

PROJECT: WOODFIN SUITE HOTEL Emeryville, California	**Log of Test Pit No. T-4**	
BORING LOCATION: See Figure 1, Site and Boring Location Plan	ELEVATION AND DATUM: Approximately 9.2 feet	
DRILLING CONTRACTOR: Ghilotti Brothers	DATE STARTED: 4/3/97	DATE FINISHED: 4/3/97
DRILLING EQUIPMENT: Backhoe John Deere 710D	TOTAL DEPTH: 6.5 feet	MEASURING POINT: Top of pavement
DRILLING METHOD: Backhoe pit	DEPTH WHERE FREE WATER FIRST ENCOUNTERED: No water encountered	
SAMPLING METHOD: See Boring Log Explanation, Figure A-1	DEPTH TO WATER AFTER COMPLETION (Date/Time): ---	
HAMMER WEIGHT: --- DROP: ---	LOGGED BY: A. Blanc	

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/ Foot	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content (%)	LABORATORY TESTS: Dry Density (pcf)	LABORATORY TESTS: Other
				3 inches asphalt concrete over 6 inches aggregate base over 3 inches asphalt concrete			
1				CLAYEY SAND with GRAVEL (SC) Yellowish brown, moist, [FILL]			
2				CLAYEY SAND with GRAVEL (SC) Dark greenish gray, moist to wet [FILL]			
3				SILTY SAND (SM) Very dark brown to black, with wood debris and fragments of brick [FILL]			
				SANDY CLAY (CL) Black			
4				CLAY (CH) Olive gray, moist			
5							
6							
				Bottom of test pit at 6.5 feet.			
7							

GT-1 (03/97)

PROJECT: WOODFIN SUITE HOTEL Emeryville, California		**Log of Test Pit No. T-5**	
BORING LOCATION: See Figure 1, Site and Boring Location Plan		ELEVATION AND DATUM: Approximately 8.7 feet	
DRILLING CONTRACTOR: Ghilotti Brothers		DATE STARTED: 4/3/97	DATE FINISHED: 4/3/97
DRILLING EQUIPMENT: Backhoe John Deere 710D		TOTAL DEPTH: 6 feet	MEASURING POINT: Top of pavement
DRILLING METHOD: Backhoe pit		DEPTH WHERE FREE WATER FIRST ENCOUNTERED: Approximately 2.2 feet	
SAMPLING METHOD: See Boring Log Explanation, Figure A-1		DEPTH TO WATER AFTER COMPLETION (Date/Time): ---	
HAMMER WEIGHT: ---	DROP: ---	LOGGED BY: A. Blanc	

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/Foot	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content (%)	LABORATORY TESTS: Dry Density (pcf)	LABORATORY TESTS: Other
				3 inches asphalt concrete over 6 inches aggregate base			
1				SAND with GRAVEL (SW) Yellowish to reddish brown, fine to coarse sand, fine to coarse gravel [FILL]			
				6-inch-diameter vitrified clay pipe encountered at west end of pit, adjacent to 1-foot-wide strip of concrete, 6 inches thick. 6-inch diameter tar-coated steel pipe encountered at east end of pit			
2				ATD ▽			R-Value=35 (Soil from 0.75-2 feet and 2.2-5 feet)
				2.5 inches asphalt concrete			
				SILTY CLAY with SAND (CL) Black, moist			
3							
4							
5				CLAY (CH) Olive gray, moist			
6				Bottom of test pit at 6 feet.			
7							

GT-1 (03/97)

PROJECT: WOODFIN SUITE HOTEL Emeryville, California	# Log of Test Pit No. T-6	
BORING LOCATION: See Figure 1, Site and Boring Location Plan	ELEVATION AND DATUM: Approximately 8.5 feet	
DRILLING CONTRACTOR: Ghilotti Brothers	DATE STARTED: 4/3/97	DATE FINISHED: 4/3/97
DRILLING EQUIPMENT: Backhoe John Deere 710D	TOTAL DEPTH: 16 inches	MEASURING POINT: Top of pavement
DRILLING METHOD: Backhoe pit	DEPTH WHERE FREE WATER FIRST ENCOUNTERED: No water encountered	
SAMPLING METHOD: See Boring Log Explanation, Figure A-1	DEPTH TO WATER AFTER COMPLETION (Date/Time): ---	
HAMMER WEIGHT: --- DROP: ---	LOGGED BY: A. Blanc	

DEPTH (feet)	SAMPLES			MATERIAL DESCRIPTION	LABORATORY TESTS		
	Sample No.	Sample	Blows/ Foot		Moisture Content (%)	Dry Density (pcf)	Other
				2 inches asphalt concrete over 6 inches aggregate base			
1				SANDY CLAY (CL) Black [FILL]			
				Bottom of test pit at 16 inches. Concrete slab encountered at base of test pit.			
2							
3							
4							
5							
6							
7							

GT-1 (03/97)

PROJECT: WOODFIN SUITE HOTEL Emeryville, California		**Log of Test Pit No. T-7**	
BORING LOCATION: See Figure 1, Site and Boring Location Plan		ELEVATION AND DATUM: Approximately 8.2 feet	
DRILLING CONTRACTOR: Ghilotti Brothers		DATE STARTED: 4/3/97	DATE FINISHED: 4/3/97
DRILLING EQUIPMENT: Backhoe John Deere 710D		TOTAL DEPTH: 18 inches	MEASURING POINT: Top of pavement
DRILLING METHOD: Backhoe pit		DEPTH WHERE FREE WATER FIRST ENCOUNTERED: No water encountered	
SAMPLING METHOD: See Boring Log Explanation, Figure A-1		DEPTH TO WATER AFTER COMPLETION (Date/Time):	
HAMMER WEIGHT: ---	DROP: ---	LOGGED BY: A. Blanc	

DEPTH (feet)	SAMPLES			MATERIAL DESCRIPTION	LABORATORY TESTS		
	Sample No.	Sample	Blows/ Foot		Moisture Content (%)	Dry Density (pcf)	Other
				3 inches asphalt concrete over approximately 9 inches aggregate base, brown, gravel to 2 inches in size			
1							
				2 inches asphalt concrete			
				Bottom of test pit at 18 inches. Concrete wtih reinforcing steel bars encountered at base of test pit.			
2							
3							
4							
5							
6							
7							

GT-1 (03/97)

PROJECT: WOODFIN SUITE HOTEL Emeryville, California		# Log of Test Pit No. T-8	
BORING LOCATION: See Figure 1, Site and Boring Location Plan		ELEVATION AND DATUM: Approximately 9.3 feet	
DRILLING CONTRACTOR: Ghilotti Brothers		DATE STARTED: 4/3/97	DATE FINISHED: 4/3/97
DRILLING EQUIPMENT: Backhoe John Deere 710D		TOTAL DEPTH: 15 inches	MEASURING POINT: Top of pavement
DRILLING METHOD: Backhoe pit		DEPTH WHERE FREE WATER FIRST ENCOUNTERED: No water encountered	
SAMPLING METHOD: See Boring Log Explanation, Figure A-1		DEPTH TO WATER AFTER COMPLETION (Date/Time): ---	
HAMMER WEIGHT: ---	DROP: ---	LOGGED BY: A. Blanc	

DEPTH (feet)	SAMPLES			MATERIAL DESCRIPTION	LABORATORY TESTS		
	Sample No.	Sample	Blows/ Foot		Moisture Content (%)	Dry Density (pcf)	Other
				3 inches asphalt concrete over 6 inches aggregate base			
1				SILTY SAND (SM) Yellowish brown, moist [FILL]			
				SANDY CLAY (CL) Black, moist			
2				Bottom of test pit at 15 inches. Excavation terminated when utility lines/conduit encountered at base of test pit.			
3							
4							
5							
6							
7							

GT-1 (03/97)

Log of Test Pit No. T-9

PROJECT: WOODFIN SUITE HOTEL Emeryville, California	**Log of Test Pit No. T-9**
BORING LOCATION: See Figure 1, Site and Boring Location Plan	ELEVATION AND DATUM: Approximately 8.7 feet
DRILLING CONTRACTOR: Ghilotti Brothers	DATE STARTED: 4/3/97 — DATE FINISHED: 4/3/97
DRILLING EQUIPMENT: Backhoe John Deere 710D	TOTAL DEPTH: 15 inches — MEASURING POINT: Top of pavement
DRILLING METHOD: Backhoe pit	DEPTH WHERE FREE WATER FIRST ENCOUNTERED: No water encountered
SAMPLING METHOD: See Boring Log Explanation, Figure A-1	DEPTH TO WATER AFTER COMPLETION (Date/Time): ---
HAMMER WEIGHT: --- — DROP: ---	LOGGED BY: A. Blanc

DEPTH (feet)	SAMPLES: Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content (%)	Dry Density (pcf)	Other
				3 inches asphalt concrete over 6 inches aggregate base			
1				SANDY CLAY (CL) Black, moist [FILL]			
				Bottom of test pit at 15 inches. Concrete encountered at base of pit.			
2							
3							
4							
5							
6							
7							

GT-1 (03/97)

PROJECT: WOODFIN SUITE HOTEL Emeryville, California	**Log of Test Pit No. T-10**	
BORING LOCATION: See Figure 1, Site and Boring Location Plan	ELEVATION AND DATUM: Approximately 8.5 feet	
DRILLING CONTRACTOR: Ghilotti Brothers	DATE STARTED: 4/3/97	DATE FINISHED: 4/3/97
DRILLING EQUIPMENT: Backhoe John Deere 710D	TOTAL DEPTH: 6.5 feet	MEASURING POINT: Top of pavement
DRILLING METHOD: Backhoe pit	DEPTH WHERE FREE WATER FIRST ENCOUNTERED: No water encountered	
SAMPLING METHOD: See Boring Log Explanation, Figure A-1	DEPTH TO WATER AFTER COMPLETION (Date/Time): ---	
HAMMER WEIGHT: --- DROP: ---	LOGGED BY: A. Blanc	

DEPTH (feet)	SAMPLES: Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content (%)	Dry Density (pcf)	Other
				3 inches asphalt concrete over 6 inches aggregate base			
1				SANDY CLAY (CL) Black, with pieces of wood [FILL]			
				8 inches asphalt concrete with pieces of wood, over 18 inches aggregate base with round to subrounded gravel up to 2 inches in size			
2				? 8-inch-diameter steel pipe encountered at west end of pit, on south side of pit			
3							
				GRAVEL(GP) with SAND (GP) Brown, moist [FILL]			
4				SAND (SP) and SILT (ML) Gray, wet			
				CLAY (CH) Dark greenish gray, moist			
5							
6							
				Bottom of test pit at 6.5 feet.			
7							

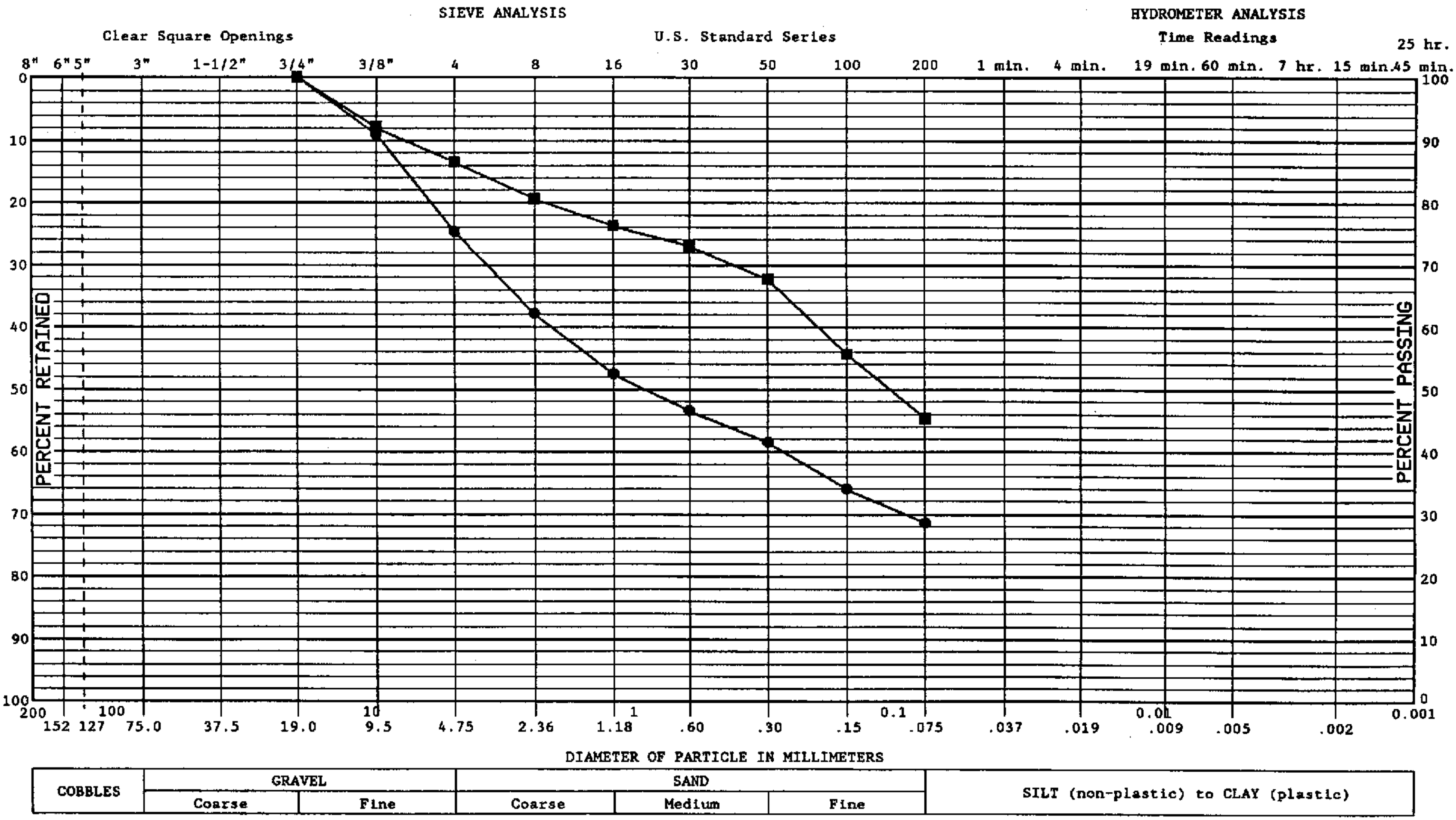
GT-1 (03/97)

PROJECT: WOODFIN SUITE HOTEL Emeryville, California	# Log of Test Pit No. T-11
BORING LOCATION: See Figure 1, Site and Boring Location Plan	ELEVATION AND DATUM: Approximately 8.1 feet
DRILLING CONTRACTOR: Ghilotti Brothers	DATE STARTED: 4/3/97 — DATE FINISHED: 4/3/97
DRILLING EQUIPMENT: Backhoe John Deere 710D	TOTAL DEPTH: 6.5 feet — MEASURING POINT: Top of pavement
DRILLING METHOD: Backhoe pit	DEPTH WHERE FREE WATER FIRST ENCOUNTERED: No water encountered
SAMPLING METHOD: See Boring Log Explanation, Figure A-1	DEPTH TO WATER AFTER COMPLETION (Date/Time): ---
HAMMER WEIGHT: --- — DROP: ---	LOGGED BY: A. Blanc

DEPTH (feet)	SAMPLES: Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content (%)	Dry Density (pcf)	Other
0–0.75				3 inches asphalt concrete over 6 inches aggregate base			
0.75–1.5				Layers of SILTY SAND with GRAVEL (SM) [FILL] 4 inches Asphalt Concrete at east end of pit			
1.5–3				GRAVEL (GP) Rounded to subrounded gravel, with wood debris [FILL]			
3–4				SAND (GP), GRAVEL (GP), and DEBRIS Oily appearance, wood, burned shingles [FILL]			
4–5.5				CLAY (CH) Olive gray, moist ATD ▽			
5.5–6.5				SILTY SAND (SM) Gray, wet, with shell fragments			
6.5				Bottom of test pit at 6.5 feet.			

GT-1 (03/97)

BORING NO.	SAMPLE NO.	DEPTH, FT.	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
H-1		90.0	●			
H-2		65.5	■			



Project No. 4105
Geomatrix Consultants

GRAIN SIZE DISTRIBUTION CURVES
WOODFIN SUITES HOTEL
EMERYVILLE, CALIFORNIA

Figure A-15

GEOMATRIX

APPENDIX B

ConCeCo ENGINEERING, INC.
CORROSION INVESTIGATION

ConCeCo Engineering, Inc.

5167 Clayton Road, Suite F
Concord, CA 94521

May 7, 1997 ConCeCo No. 2S97045

Geomatrix Consultants, Inc.
100 Pine Street, 10th Floor
San Francisco, CA 94111

Attention: Ms. Laura Varner

Subject: **Corrosion Investigation - Woodfin Suites Hotel, Emeryville, CA.**
Materials of Construction Including Copper, Steel,
Ductile and Cast Iron, Concrete and Steel Rebar

Dear Ms. Varner,

In accordance with your request, we present the following general analysis with respect to the above referenced materials of construction.

SCOPE

A. Determine the corrosivity of two soil samples furnished by **Geomatrix** and evaluate the potential for corrosion on materials of construction.

B. Conduct chemical and electrical tests on the soil samples in accordance with generally accepted methods and standards.

C. Analyze test data and provide a short letter-type report which includes test results and recommendations necessary for corrosion control.

PROCEDURES

Laboratory Tests:

Soil tests which were conducted which are useful in the evaluation of soil corrosivity in four basic categories:

1. ELECTRICAL RESISTIVITY

   The resistivity of soil is a good general indicator of soil corrosivity since metal corrosion is an electrochemical reaction, i.e., the greater the current in a given corrosion cell the higher the corrosion rate.

2. ANAEROBIC CONDITIONS (pH, Redox, sulfides)

   Anaerobic conditions indicate the presence of depolarizing agents which sustain greater current levels and result in corrosion rates which can be two to ten times higher than that indicated by soil resistivity alone.

3. ACIDITY (pH)

   Acidity can be a key indicator of the tendency for metal ions to go into solution in the soil electrolyte.

4. CONCRETE AND CEMENT MORTAR COATED STEEL (sulfates, chlorides, acidity)

   Acidity (pH <5.5) is also an indication of possible chemical attack on concrete. Steel in concrete has also been found to sustain severe corrosion where chlorides are able to migrate to the steel surface and sulfates are found to promote the deterioration of the concrete and chloride attack on the rebar.

Standard test methods were selected which were best suited for repeatable results, accuracy, interpretation, and generally accepted practice. The standard methods for the tests involved are listed at the bottom of Table I. A complete description of the test procedures and standard references can be furnished if required.

DISCUSSION

The NACE International classifications of corrosivity for the soils reported in Table I would be as shown below:

NACE SOIL CORROSIVITY CLASSIFICATION[1]

Soil Resistivity (ohm-cm)	Soil Classification
Below 500	Very Corrosive
500 to 1,000	Corrosive
1,000 to 2,000	Moderately Corrosive
2,000 to 10,000	Mildly Corrosive
Above 10,000	Progressively Less Corrosive

[1]NACE, Corrosion Basics - An Introduction, NACE, Houston, TX., pp. 191 (1984)

Steel:

The NACE soil corrosivity classification is based on average soil resistivity values for soils in general. Based upon the low resistivity alone (from 1,440 ohm-cm to 3,781 ohm-cm), the soil would be classified as moderately corrosive to mildly corrosive. Because of the low redox, neutral pH and high concentration of sulfides, indicating a strong depolarizing environment, we would classify the soils tested as being "corrosive" to "very corrosive" to steel.

Ductile and Cast Iron:

The evaluation of ductile iron and cast iron can be compared to the steel classification presented above and the table from Appendix A, ANSI A21, attached. Since the corrosion rates of iron and steel are very close, we generally compare the two classification systems. The two classification systems compare favorably and lead to the conclusion that soil represented the sample tested are "corrosive" to "very corrosive" to ductile and gray cast iron.

Concrete Rebar:

The Uniform Building Code (UBC) sets standards for exposure limits (see Table 26-A-7); and, if we convert the chloride and sulfate content from ppm to percent, we can see that the soil samples tested exceeded these limits for Type II cement as follows.

Sample No.	Water Soluble Chloride in Soil % by Weight (from Table I)	Water Soluble Sulfate in Soil % by Weight (from Table I)
1. H-1 (1-4)	0.116 (1,160 ppm)	0.200 (2,000 ppm)
2. 4105 T-5	.00846	.01058

Concrete:

The Uniform Building Code (UBC) sets standards for exposure limits and the sulfate content of the sample tested is compared to the soil class by the UBC and the USBR (U.S. Bureau of Reclamation). The sulfate content in the soil sample H-1 was 0.20% (2,000 ppm) and was in the "Severe" class for soils and would be severely aggressive to concrete exposed to water of this type. The water soluble chloride found in the sample is high and would cause corrosion on any rebar not fully coated with good quality concrete. Criteria in "Caltrans Test #532 (1993)" recommends at least 3-inches of concrete cover over rebar using a 6 sack cement/yd. mix with 15% maximum total water content.

The concrete designer is referred to the Caltrans reference as well as the UBC, USBR, and PCA (Portland Cement Association) to select an appropriate concrete mix design and minimum rebar concrete cover.

Copper:

Copper is generally attacked by ammonia and extreme acid in soils. Ammonia alone or a combination of ammonia, pH below 5.0 and soils with resistivity below 500 ohm-cm can indicate problems with copper. Copper will not tolerate even small amounts of ammonia. Recent external corrosion failures have generally been related to design deficiencies where copper pipes are placed in adjacent dissimilar environments or are subject to stray electrical current. A significant increase in these types of failures shows the need for design review by a qualified corrosion engineer. Although the pH of the soil samples was less acidic than 5.0, the soils were found to have resistivity above 500 ohm-cm which would support classification of the soils as mildly or less corrosive to copper. Ammonia tests were conducted, due to the organic and peat content in the sample were found to contain from 29 to 120 mg/kg total ammonia, these results would give rise to a classification of "very corrosive" to copper.

CONCLUSIONS

1. The soils tested were classified in accordance with the National Association of Corrosion Engineers (NACE) for steel, the American National Standards Institute (ANSI) A21.5 for cast or ductile iron, the Portland Cement Association (PCA) for concrete and on the basis of our experience, research and investigations for concrete encased steel or mortar coated steel. Materials anticipated for use in this project are classified as follows:

Material	Corrosion Class
Steel, Bare/Galvanized/Coated	Very Corrosive
Iron, Cast/Ductile or Mortar Coated Steel	Very Corrosive
Steel, Concrete Coated(good quality)	Very Corrosive
Copper	Very Corrosive

2. The pH value of 7.02 indicate a soil acidity which is not aggressive to portland cement. However, soils containing high sulfates such as those found in this soil sample are considered severely aggressive to portland cement paste in concrete. Soils containing more than .03% chloride generally may cause corrosion of steel rebar in concrete. This soil sample exceeded the criteria (0.116%) and are expected to cause early rebar corrosion with standard design for concrete.

RECOMMENDATIONS

1. All buried bare, galvanized, concrete/mortar, or dielectric coated steel, ductile or cast iron and copper pipe should be cathodically protected. The cathodic protection system should be designed by a qualified corrosion engineer.

2. Steel reinforcement for concrete should be protected by providing more than 3-inch minimum concrete cover, to avoid corrosion of rebar from chloride contamination for soils representative of soil sample H-1. The concrete mix design should include selecting appropriate cement to improve resistance to sulfate attack, and by reducing the concrete porosity.

3. Concrete which is exposed soils with sulfate contents found in sample H-1 should be provided with type V cement. Care should be taken to eliminate any cracking of concrete which would allow soil or ground water contact with rebar.

4. Cast iron or ductile iron pipe and fittings should be polyethylene encased in accordance with ANSI A21.5, as a cost effective method to reduce current requirements for cathodic protection.

5. Insulating elements should be provided between different materials and different environs (concrete-soil, soil-air, low pH native soil-high pH sand, etc.) both above and below ground. Most importantly is an insulating union or flange between above and below grade piping located just **above grade**.

6. Soil conditions are not the only factors which may cause corrosion loss; design and construction practice may also be primary causes for failure. A review of plans and specifications for underground structures should be conducted by a qualified corrosion engineer prior to construction.

The user of this report should keep in mind that these conclusions were derived from just two soil samples. These conclusions and recommendations are preliminary; and, depending upon the size of the project and how representative the soil samples tested, a more extensive corrosivity study may be required.

We appreciate the opportunity to provide this service to you and your company. If further information is required in this matter or if we can be of further service, please advise.

Sincerely,
ConCeCo Engineering, Inc.

Robert E. Colson

Robert E. Colson, P.E.

REC:clj

Enclosures: Table I
UBC Tables 26-A-6, and 26-A-7
Excerpts from USBR, PCA and ANSI A21.1/AWWA C105
Table A.1

CONCRETE MANUAL

A WATER RESOURCES TECHNICAL PUBLICATION

A manual for the control of concrete construction

EIGHTH EDITION, RIVISED REPRINT 1981

REPRINTED 1988

U.S. DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

ENGINEERING BULLETIN

Design and Control of Concrete Mixtures

THIRTEENTH EDITION

by Steven H. Kosmatka and William C. Panarese

PORTLAND CEMENT PCA ASSOCIATION
5420 Old Orchard Road, Skokie, Illinois 60077-1083

An organization of cement manufacturers to improve and extend the uses of portland cement and concrete through market development, engineering, research, education, and public affairs work.

TABLE A.1

Soil-Test Evaluation*

Soil Characteristics	Points
Resistivity-*ohm-cm* (based on single-probe at pipe depth or water-saturated soil box):	
<700	10
700-1000	8
1000-1200	5
1200-1500	2
1500-2000	1
>2000	0
pH:	
0-2	5
2-4	3
4-6.5	0
6.5-7.5	0†
7.5-8.5	0
>8.5	3
Redox potential:	
>+100 mV	0
+50 to +100 mV	3.5
0 to +50 mV	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

* Ten points - corrosive to gray or ductile cast iron pipe: protection is indicated.

† If sulfides are present and less or negative redox-potential results are obtained, three points shall be given for this range.

ConCeCo Engineering, Inc.

Client: Geomatrix Consultants — ConCeCo Job No. 2S97045

Project: Geomatrix Project No. 4105
Woodfin Suites Hotel, Emeryville

Samples Received: April 16, 1997
Samples Tested: April 17, 1997

SOIL TEST SUMMARY

Sample No.	Resistivity (ohm-cm)		Redox (mV)	pH	Sulfides	Sulfate mg/l	Chlorides mg/l	Ammonia mg/kg	Soil Description
	As-received	Saturated							
H-1 (1-4)	1,440	1,440	-185	7.02	Pos.	2,000	1,160	120	Odorous, Saturated Mixed Black & Brown, w/Leaves, Stones, Wood, and other organic, matter-clayey peat
4105 T-5		3,781	119	6.64	Pos.	105.8	84.6	29	Brown clayey silt composite - moist

Notes:

1. Appendix A of ANSI/AWWA C105/A215, TABLE A, provides soil test methods and evaluation for conditions corrosive to gray or ductile-cast iron pipe and fittings.

2. The above test (excluding redox and sulfides) were performed in accordance with the following Caltrans Test Methods:

   a. California Test 643 (1993): METHODS FOR ESTIMATING THE SERVICE LIFE OF STEEL CULVERTS.

   b. California Test 532 (1993): METHOD FOR ESTIMATING THE TIME TO CORROSION OF REINFORCED CONCRETE SUBSTRUCTURES.

   c. California Test 422 (1978): METHOD OF TESTING SOILS AND WATERS FOR CHLORIDE CONTENT.

   d. California Test 417 (1986): METHOD OF TESTING SOILS AND WATERS FOR SULFATE CONTENT.

3. EPA Method No. 350.3: TOTAL AMMONIA

Table 2-1. Sources of Raw Materials Used in Manufacture of Portland Cement

Lime, CaO	Iron, Fe_2O_3	Silica, SiO_2	Alumina, Al_2O_3	Gypsum, $CaSO_4 \cdot 2H_2O$	Magnesia, MgO
Alkali waste	Blast-furnace flue dust	Calcium silicate	Aluminum-ore refuse*	Anhydrite	Cement rock
Aragonite*	Clay*	Cement rock	Bauxite	Calcium sulfate	Limestone
Calcite*	Iron ore*	Clay*	Cement rock	Gypsum	Slag
Cement-kiln dust	Mill scale*	Fly ash	Clay*		
Cement rock	Ore washings	Fuller's earth	Copper slag		
Chalk	Pyrite cinders	Limestone	Fly ash*		
Clay	Shale	Loess	Fuller's earth		
Fuller's earth		Marl*	Granodiorite		
Limestone*		Ore washings	Limestone		
Marble		Quartzite	Loess		
Marl*		Rice-hull ash	Ore washings		
Seashells		Sand*	Shale*		
Shale*		Sandstone	Slag		
Slag		Shale*	Staurolite		
		Slag			
		Traprock			

Note: As a generalization, probably 50% of all industrial byproducts have potential as raw materials for the manufacture of portland cement.

*Most common sources.

tions of all cement plants are basically the same, no flow diagram can adequately illustrate all plants. There is no typical portland cement manufacturing plant; every plant has significant differences in layout, equipment, or general appearance.*

Selected raw materials (Table 2-1) are crushed, milled, and proportioned in such a way that the resulting mixture has the desired chemical composition. The raw materials are generally a mixture of calcareous (calcium oxide) material, such as limestone, chalk or shells, and an argillaceous (silica and alumina) material such as clay, shale, or blast-furnace slag. Either a dry or a wet process is used. In the dry process, grinding and blending are done with dry materials. In the wet process, the grinding and blending operations are done with the materials in slurry form. In other respects, the dry and wet processes are very much alike. Fig. 2-3 illustrates important technological developments that can improve significantly the productivity and energy efficiency of dry-process plants.

After blending, the ground raw material is fed into the upper end of a kiln. The raw mix passes through the kiln at a rate controlled by the slope and rotational speed of the kiln. Burning fuel (powdered coal, oil, or gas) is forced into the lower end of the kiln where temperatures of 2600°F to 3000°F change the raw material chemically into cement clinker, grayish-black pellets about the size of ½-in.-diameter marbles.

The clinker is cooled and then pulverized. During this operation a small amount of gypsum is added to regulate the setting time of the cement. The clinker is ground so fine that nearly all of it passes through a No. 200 mesh (75 micron) sieve with 40,000 openings per square inch. This extremely fine gray powder is portland cement.

TYPES OF PORTLAND CEMENT

Different types of portland cement are manufactured to meet various normal physical and chemical requirements for specific purposes. The American Society for Testing and Materials (ASTM) Designation C 150, Standard Specification for Portland Cement, provides for eight types of portland cement as follows:

Type I	normal
Type IA	normal, air-entraining
Type II	moderate sulfate resistance
Type IIA	moderate sulfate resistance, air-entraining
Type III	high early strength
Type IIIA	high early strength, air-entraining
Type IV	low heat of hydration
Type V	high sulfate resistance

Type I

Type I portland cement is a general-purpose cement suitable for all uses where the special properties of other types are not required. It is used in concrete that is not subject to aggressive exposures, such as sulfate attack from soil or water, or to an objectionable temperature rise due to heat generated by hydration. Its uses in concrete include pavements, floors, reinforced concrete buildings, bridges, railway structures, tanks and reservoirs, pipe, masonry units, and other precast concrete products.

Type II

Type II portland cement is used where precaution against moderate sulfate attack is important, as in drainage structures where sulfate concentrations in groundwaters are higher than normal but not unusually severe (see Table 2-2). Type II cement will usually generate less heat at a slower rate than Type I. The requirement of moderate heat of hydration can be

*Mechanical equipment is described in Reference 2-16.

Table 2-2. Types of Cer t Required for Concrete Exposed to ate Attack

Sulfate exposure	Water-soluble sulfate (SO_4) in soil, percent by weight	Sulfate (SO_4) in water, ppm	Cement type
Negligible	0.00-0.10	0-150	—
Moderate*	0.10-0.20	150-1500	II, IP(MS), IS(MS)
Severe	0.20-2.00	1500-10,000	V
Very severe	Over 2.00	Over 10,000	V plus pozzolan**

*Seawater.
**Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.
Source: Reference 2-20 and ACI 318, Table 4.5.3.

specified at the option of the purchaser. If heat-of-hydration maximums are specified, this cement can be used in structures of considerable mass, such as large piers, and heavy abutments and retaining walls. Its use will reduce temperature rise, which is especially important when concrete is placed in warm weather.

Type III

Type III portland cement provides high strengths at an early period, usually a week or less. It is chemically and physically similar to Type I cement, except that its particles have been ground finer. It is used when forms need to be removed as soon as possible or when the structure must be put into service quickly. In cold weather its use permits a reduction in the controlled curing period. Although richer mixes of Type I cement can be used to gain high early strength, Type III may provide it more satisfactorily and more economically.

Type IV

Type IV portland cement is used where the rate and amount of heat generated from hydration must be minimized. It develops strength at a slower rate than other cement types. Type IV cement is intended for use in massive concrete structures, such as large gravity dams, where the temperature rise resulting from heat generated during hardening must be minimized.

Type V

Type V portland cement is used only in concrete exposed to severe sulfate action—principally where soils or groundwaters have a high sulfate content. It gains strength more slowly than Type I cement. Table 2-2 describes sulfate concentrations requiring the use of Type V cement. The high sulfate resistance of Type V cement is attributed to a low tricalcium aluminate (C_3A) content as illustrated in Fig. 2-4. Sulfate resistance also increases with air entrainment and increasing cement contents (low water-cement ratios). Type V

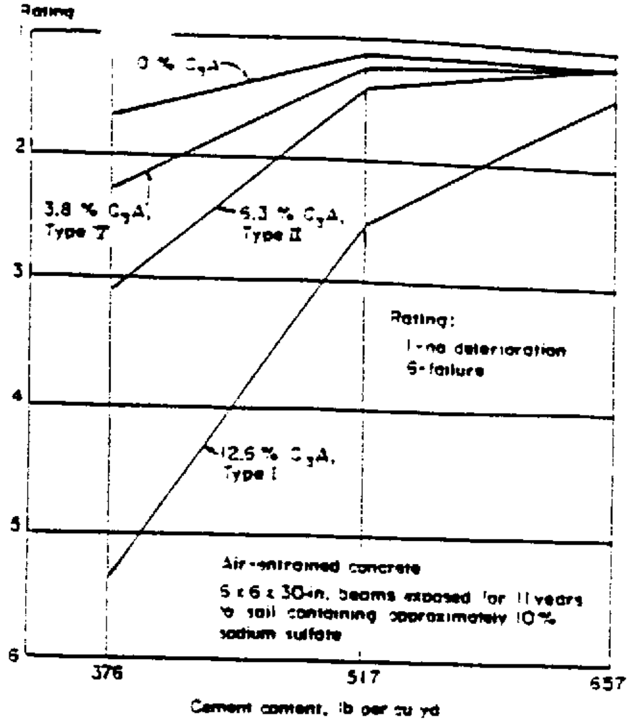


Fig. 2-4. Performance of concretes made with cemen with different C_3A contents in sulfate soil. See Fig. 2-5 the rating description. Reference 2-24.

Fig. 2-5. Range of durability represented, from left to righ by visual ratings of about 1, 4, and 6 for the sulfate-resis ance tests in Fig. 2-4. Reference 2-24.

cement, like other portland cements, is not resistant acids and other highly corrosive substances.

Air-Entraining Portland Cements

Specifications for three types of air-entraining portlan cement (Types IA, IIA, and IIIA) are given in ASTM C150. They correspond in composition to ASTM Type dicated, an appropriate surface covering or treatment should be employed.

When cement and water combine, one of the compounds formed is hydrated lime, which is readily dissolved by water (often made more aggressive by the presence of dissolved carbon dioxide) passing through cracks, along improperly treated construction planes, or through interconnected voids. The removal of this or other solid material by leaching may seriously impair the quality of concrete. The white deposit, or efflorescence, commonly seen on concrete surfaces is the result of leaching and subsequent carbonation and evaporation.

(2) Certain agents combine with cement to form compounds which have a low solubility but which disrupt the concrete because their volume is greater than the volume of the cement paste from which they were formed. Disintegration may be attributed to a combination of chemical and physical forces. In dense concretes this type of attack would be largely superficial. Porous concrete would be affected throughout the mass. Most prominent among aggressive substances which affect Bureau concrete structures are the sulfates of sodium, magnesium, and calcium. These salts which are known as white alkali are frequently encountered in the alkali soils and ground waters of the western half of the United States.

The stronger the concentration of these salts the more active the corrosion. Sulfate solutions increase in strength in dry seasons when dilution is at a minimum. The sulfates react chemically with the hydrated lime and hydrated calcium aluminate in cement paste to form calcium sulfate and calcium sulfoaluminate, respectively, and

Figure 5.—Disintegration of concrete caused by sulfate attack. PX–D–32050.

Table 2.—Attack on concrete by soils and waters containing various sulfate concentrations

Relative degree of sulfate attack	Percent water-soluble sulfate (as SO_4) in soil samples	mg/l sulfate (as SO_4) in water samples
Negligible	0.00 to 0.10	0 to 150
Positive [1]	0.10 to 0.20	150 to 1,500
Severe [2]	0.20 to 2.00	1,500 to 10,000
Very severe [3]	2.00 or more	10,000 or more

[1] Use type II cement.
[2] Use type V cement, or approved combination of portland cement and pozzolan which has been shown by test to provide comparable sulfate resistance when used in concrete.
[3] Use type V cement plus approved pozzolan which has been determined by tests to improve sulfate resistance when used in concrete with type V cement.

these reactions are accompanied by considerable expansion and disruption of the paste. Figure 5 illustrates the effect of sulfate attack on concrete in a canal lining and a turnout wall. Concrete containing cement with a low content of the vulnerable calcium aluminate is highly resistant to attack by sulfate-laden soils and waters. (See sec. 15(b).) The relative degrees of attack on concrete by sulfates from soils and ground waters are given in table 2.

(3) Where concrete is subjected to alternate wetting and drying, certain salts, such as sodium carbonate, may cause surface disintegration by crystallizing in the pores of the concrete. Such action appears to be purely physical.

(4) In environments such as flash distillation chambers of desalination plants where concrete is exposed to condensing cool-to-hot water vapors or the resulting flowing or dripping of distilled water, the concrete is rapidly attacked by this mineral-free liquid. The liquid rapidly dissolves available lime and other soluble compounds of the cement matrix. Subsequent rapid deterioration and eventual decomposition result. The only palliative known at this time is complete insulation of the concrete from the mineral-free water by coatings or lining materials which are not affected by the water.

(5) Concrete in desalination plants is adversely affected by the feed water, sea water, or brine from wells. At these plants, high-quality concrete has been found unsuitable for use in brine exposures at temperatures of 290° F but suitable at 200° to 250° F provided adequate sacrificial concrete is made available for surface deterioration. Below about 200° F no provision for sacrificial concrete is generally required. Deterioration such as occurs at the higher temperature is a chemical alteration of the peripheral concrete paste which results in extensive microfracturing with resultant reduction of compressive strength, effective cross-sectional area of the member, and

TABLE NO. 26-A-6—REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

SULFATE EXPOSURE	WATER SOLUBLE SULFATE (SO_4) IN SOIL, PERCENT BY WEIGHT	SULFATE (SO_4) IN WATER, PPM	CEMENT TYPE	NORMAL-WEIGHT AGGREGATE Maximum Water-Cement Ratio, by Weight[1]	LIGHTWEIGHT-AGGREGATE CONCRETE Minimum Compressive Strength, f'_c psi[1]
Negligible	0.00-0.10	0-150	—	—	—
Moderate[2]	0.10-0.20	150-1500	II, IP(MS), IS(MS)	0.50	3750
Severe	0.20-2.00	1500-10,000	V	0.45	4250
Very Severe	Over 2.00	Over 10,000	V plus pozzolan[3]	0.45	4250

[1] A lower water-cement ratio or higher strength may be required for watertightness or for protection against corrosion of embedded items or freezing and thawing (Table No. 26-A-5).

[2] Seawater.

[3] Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

TABLE NO. 26-A-7—MAXIMUM CHLORIDE ION CONTENT FOR CORROSION PROTECTION

TYPE OF MEMBER	MAXIMUM WATER SOLUBLE CHLORIDE ION (CL) IN CONCRETE, PERCENT BY WEIGHT OF CEMENT
Prestressed concrete	0.06
Reinforced concrete exposed to chloride in service	0.15
Reinforced concrete that will be dry or protected from moisture in service	1.00
Other reinforced concrete construction	0.30

GEOMATRIX

APPENDIX C

EXPLORATORY BORINGS FROM 1988 GEOTECHNICAL STUDy

Log of Boring No. 1

PROJECT: HAWTHORN SUITES EMERYVILLE, CALIFORNIA	Log of Boring No. 1

BORING LOCATION: See Site and Boring Location Plan, Figure 1.

DATE STARTED: 3/1/88 | DATE FINISHED: 3/1/88

DRILLING METHOD: Rotary Wash

HAMMER WEIGHT: 140 lbs | DROP: 30 inches

SAMPLER: 2-inch Modified California Sampler

NOTES: Water level not measured.

DEPTH (feet)	SAMPLES Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION (Surface Elevation: 9.5 ft +/-)	LABORATORY TESTS Moisture Content (%)	Dry Density (pcf)	Unconf. Comp. Str. (pcf)
				Asphalt pavement (3 inches)			
				CLAYEY SAND (SC) Loose, brown, mixed with gravel and silt			
	1		14	CLAYEY SAND (SC) Medium dense, gray, silty and gravelly	13	125	1200
5				Brown, occasional sand lenses			
				SILTY CLAY (CL) Medium stiff, gray-tan to brown, occasional gravel			
	2		7		25	99	1960
10							
				Becoming tan, with occasional fine gravel			
15							
	3		34	CLAYEY SAND (SC) Dense, yellow-brown, with gravel			
20							
25							
				SILTY CLAY (CL) Stiff, gray-tan to yellow-brown, with rust mottling			
30							

PROJECT: HAWTHORN SUITES EMERYVILLE, CALIFORNIA	Log of Boring No. 1

DEPTH (feet)	Sample No.	Blows/ Foot	MATERIAL DESCRIPTION (Surface Elevation: 9.5 ft +/-)	Moisture Content (%)	Dry Density (pcf)	Unconf. Comp. Str. (pcf)
	4	22	SILTY CLAY (CL) Stiff, gray-tan to yellow-brown, with rust mottling	24	101	4530
35						
			Gravelly			
			Clayey			
40			Becoming stiff to very stiff and olive green			
	5	35		21	106	3920
			Becoming sandy and gravelly			
45						
			Calcite particles encountered in washings			
50						
	6	19	SILTY CLAY (CL-CH) Stiff, blue-gray occasional calcite fragments in top of layer	48	73	2280
55			Becoming sandy and gravelly encountered calcite and rust mottling			
60						
	7	31	SILTY CLAY (CL) Stiff, gray-brown, with rust mottling and gravel	21	106	7720
64						

PROJECT: HAWTHORN SUITES EMERYVILLE, CALIFORNIA	Log of Boring No. 1

DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION (Surface Elevation: 9.5 ft +/-)	Moisture Content (%)	Dry Density (pcf)	Unconf. Comp. Str. (pcf)
65				SILTY CLAY (CL) Stiff, gray-brown, with rust mottling and gravel			
				SANDY CLAY (CL) Very stiff, yellow-brown, with fine gravel and rock fragments Occasional olive green clay lenses and rust mottling			
70	8		74		19	111	3470
75				GRAVELLY CLAY (CL) to CLAYEY GRAVEL (GC) Very stiff (very dense), yellow-brown to olive-tan Occasional calcite and serpentine fragments			
80	9		50/4"		14	118	4260
85							
90	10		92				
				Bottom of boring at 91.5 ft			

Project No: 1322B | **Geomatrix Consultants** | Figure B-3

PROJECT: HAWTHORN SUITES EMERYVILLE, CALIFORNIA

Log of Boring No. 2

BORING LOCATION: See Site and Boring Location Plan, Figure 1.

DATE STARTED: 2/29/88 | DATE FINISHED: 3/1/88

DRILLING METHOD: Rotary Wash

HAMMER WEIGHT: 140 lbs | DROP: 30 inches

SAMPLER: 2-inch Modified California Sampler

NOTES: Water level not measured.

DEPTH (feet)	SAMPLES: Sample No.	Sample	Blows/Foot	MATERIAL DESCRIPTION (Surface Elevation: 9.5 ft +/-)	LABORATORY TESTS: Moisture Content (%)	Dry Density (pcf)	Unconf. Comp. Str. (pcf)
				Asphalt pavement (2.75inches)			
				Base rock (4 inches)			
				Asphalt concrete (2 inches)			
	1		18	SILTY CLAY (CL) Medium stiff, yellow-brown	16	114	3260
				SILTY to SANDY CLAY (CL-ML) Stiff, gray, with gravel and some organic material			
5				Becoming dark brown , with wood debris Clayey silt (ML)			
	2		2	SILTY CLAY (CL-CH) Soft, dark gray, with some gravel			
10							
15				SILTY CLAY (CL) Stiff, yellow-brown, abundant gravel			
	3		23		23	103	4660
				Less gravelly			
20							
	4		69	Clayey sand or sandy clay (SM-SC) Very dense, yellow-brown	22	108	
				SANDY CLAY (CL) Stiff, tan			
25							
30							

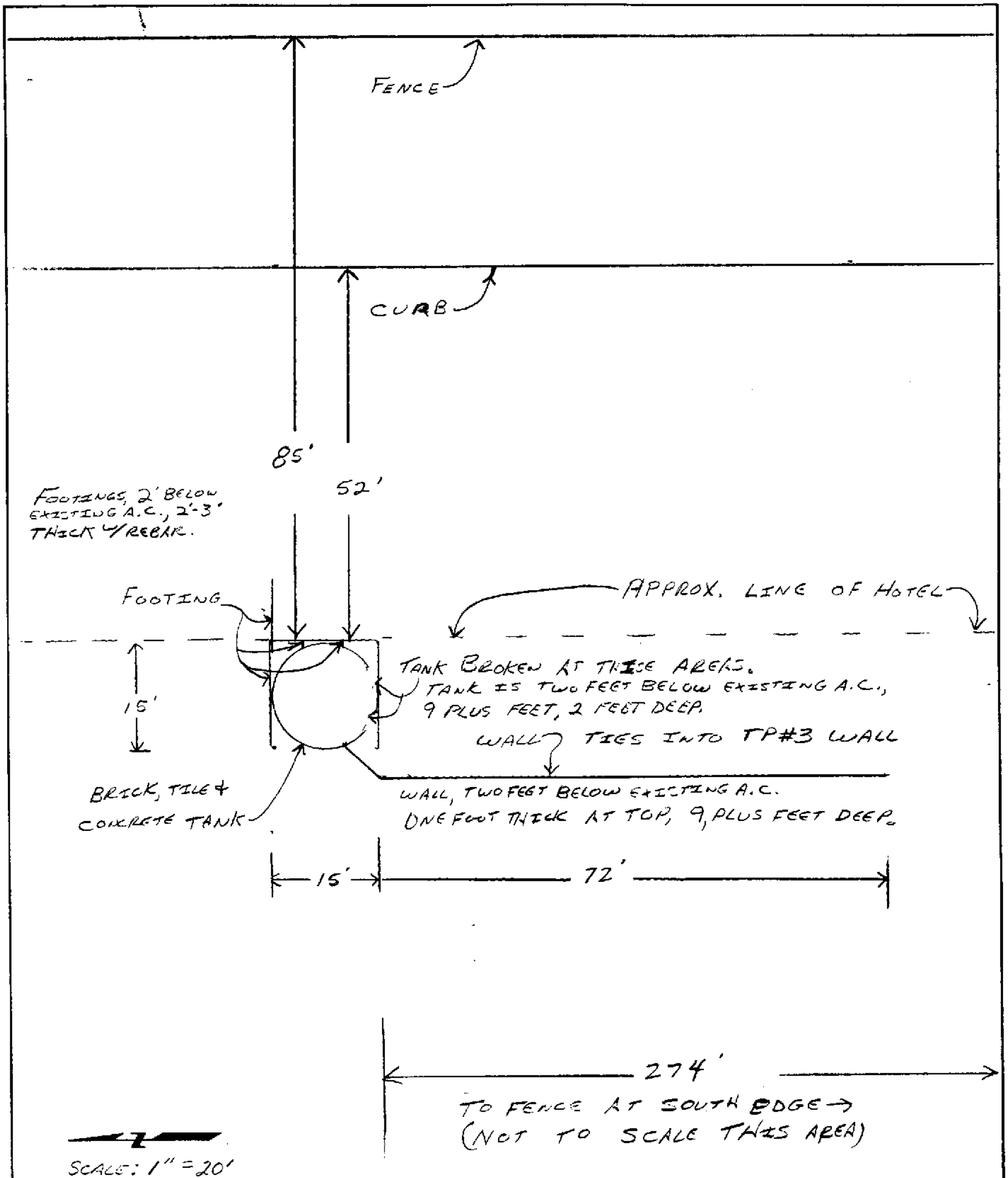
PROJECT: HAWTHORN SUITES EMERYVILLE, CALIFORNIA	Log of Boring No. 2

DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION (Surface Elevation: 9.5 ft +/-)	Moisture Content (%)	Dry Density (pcf)	Unconf. Comp. Str. (pcf)
	5		21	SILTY CLAY (CL-CH) Stiff, gray-brown, with dark brown mottling	24	100	3230
35							
				Sand layer Brown, medium grained			
				SILTY CLAY (CL-CH) Very stiff, blue-gray mixed with gravel and sand			
40	6		39				
				SANDY CLAY (CL) Very stiff, gray-brown, with fine-grained sand.			
45	7		30	Rust mottling encountered in sample	22	105	4700
50	8		92	CLAYEY GRAVEL (GC) Very dense, yellow-brown, with some silt	12	126	2660
55							
60	9		30	CLAYEY SAND (SC) Dense to very dense, yellow brown, clayey fine sand	25	103	1770
64							

PROJECT: HAWTHORN SUITES
EMERYVILLE, CALIFORNIA

Log of Boring No. 2

DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: 9.5 ft +/-	Moisture Content (%)	Dry Density (pcf)	Unconf. Comp. Str. (pcf)
65				CLAYEY GRAVEL (GC) Very dense, yellow-brown to dark brown			
70	10		101		15	118	4810
75							
				Harder drilling, larger gravel size			
80	11		56	Becoming very dense			
				Becoming clayey			
85							
90	12		77				
				Bottom of boring at 91.5 ft			

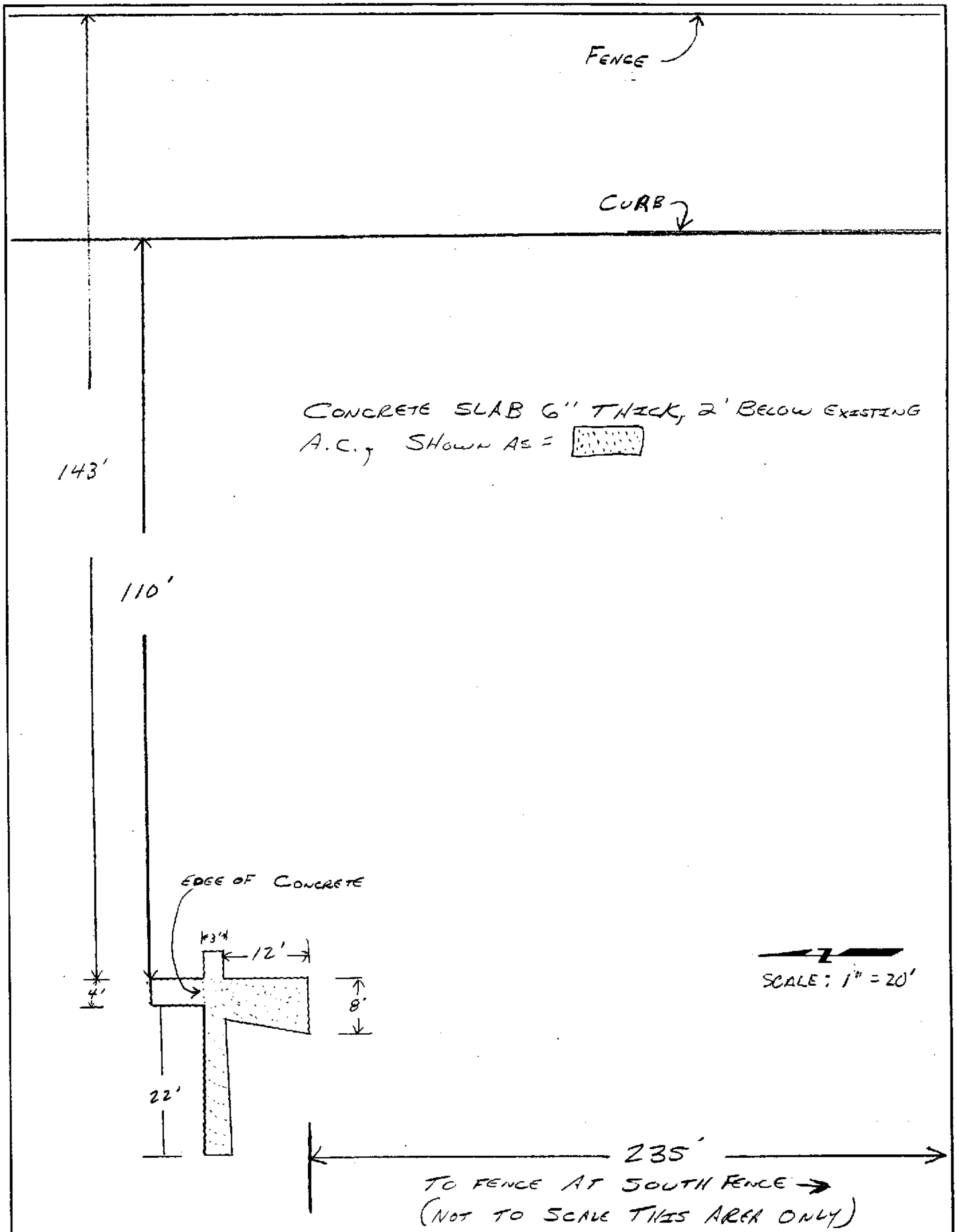


GEOMATRIX

TEST PIT #1
Hawthorne Suites Hotel
Emeryville, California

Figure **A-1**

Project No. **1322B**

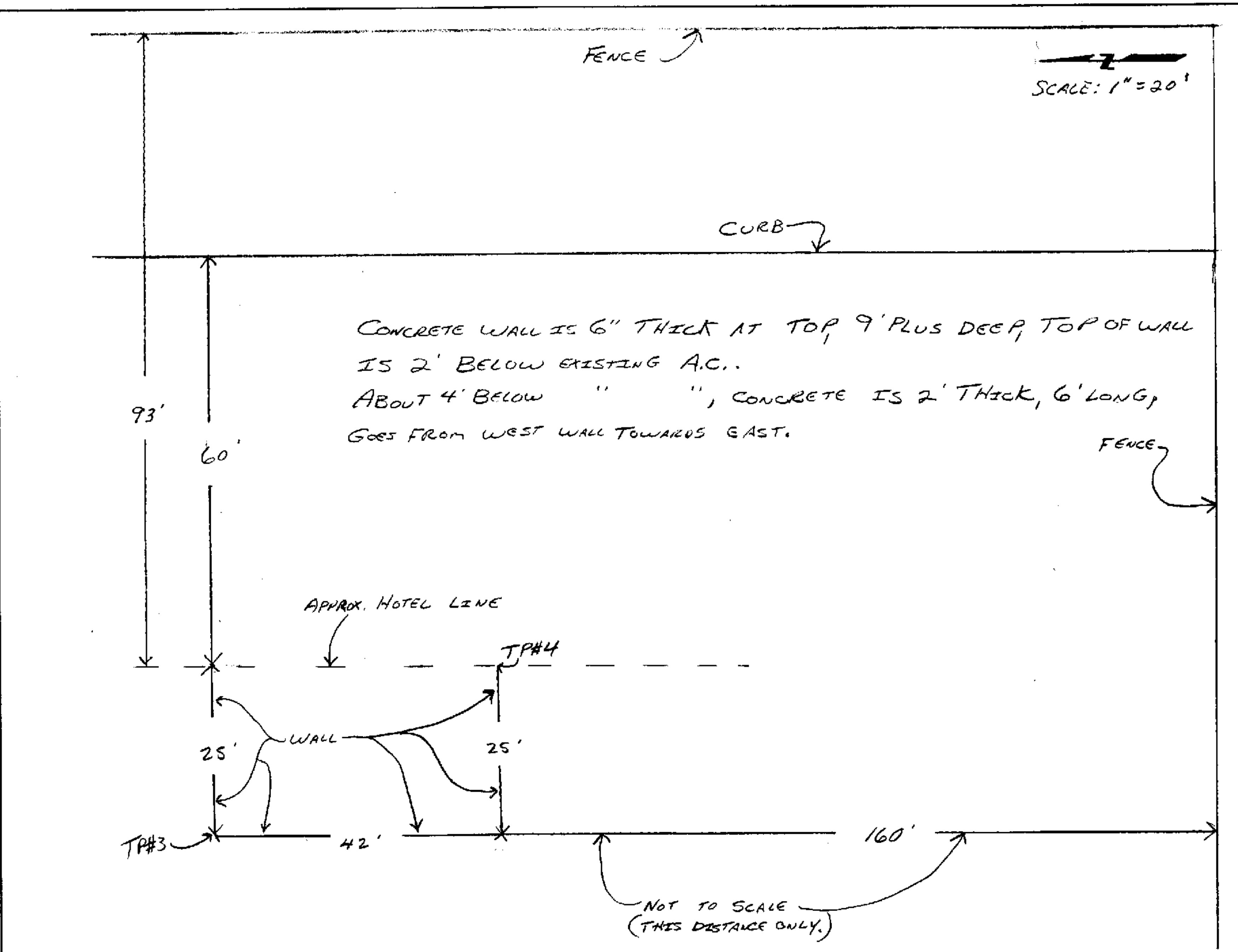


GEOMATRIX

TEST PIT #2
Hawthorne Suites Hotel
Emeryville, California

Figure
A-2

Project No.
1322B

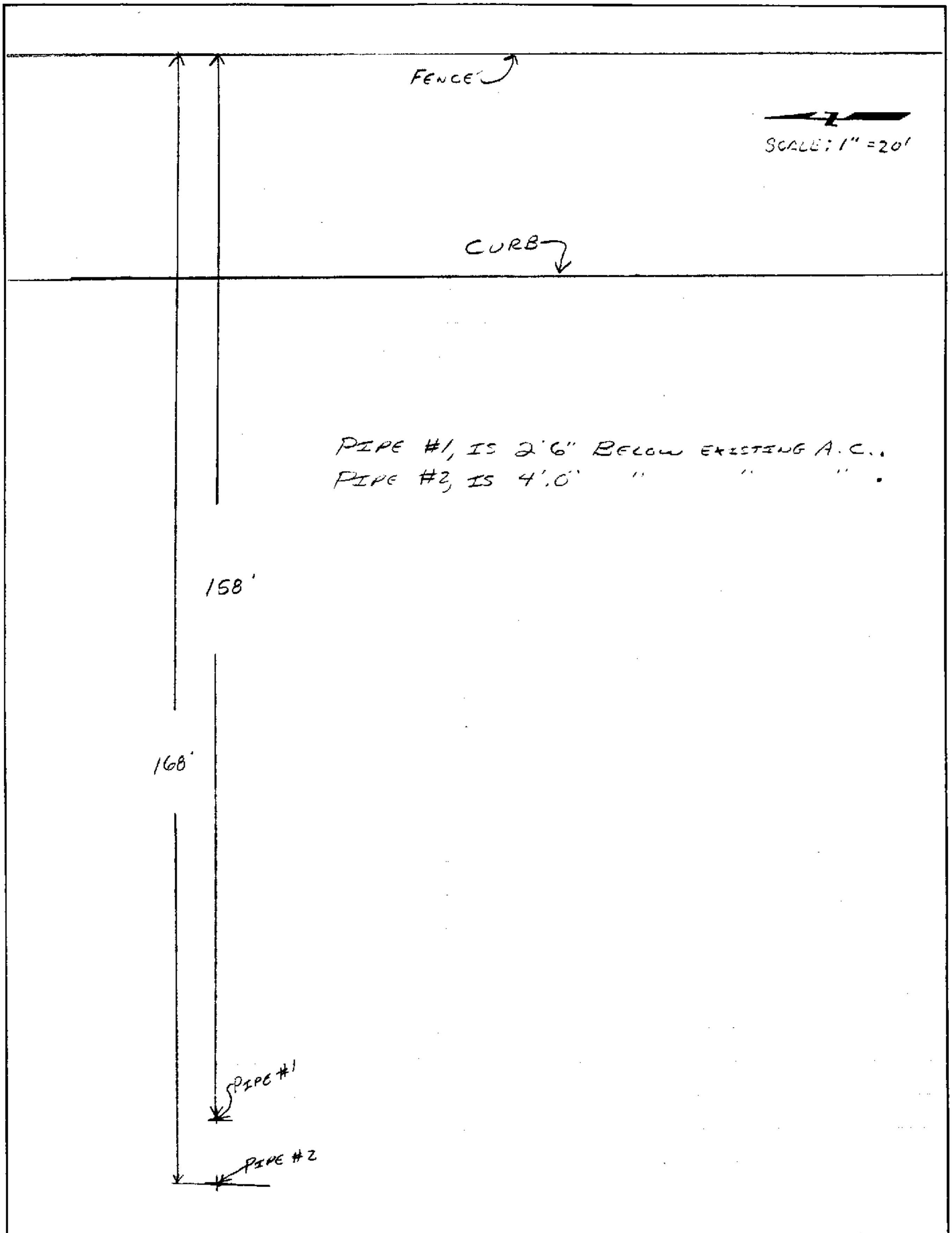


GEOMATRIX

TEST PITS #3 AND #4
Hawthorne Suites Hotel
Emeryville, California

Figure
A-3

Project No.
1322B



GEOMATRIX

TEST PIT #6
Hawthorne Suites Hotel
Emeryville, California

Figure	**A-4**
Project No.	**1322B**