APPENDIX E

GEOTECHNICAL REPORTS

APPENDIX E–1

GEOTECHNICAL REPORT
BAGG, APRIL 2005

REPORT

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
FASSLER AVENUE
PACIFICA, CALIFORNIA

For Home Pride Construction

April 21, 2005

BAY AREA GEOTECHNICAL GROUP

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April 21, 2005
BAGG Job No. PACIQ-01-00

Home Pride Construction Inc.
1201 Danmann Avenue
Pacifica, California

Attention: Mr. Rick Lee

Geotechnical Engineering Investigation
Proposed Residential Development,
APN 022-083-20 and 30 (11 Acres)
Fassler Avenue
Pacifica, California

Dear Mr. Lee,

Transmitted herewith is our geotechnical investigation report for the proposed residential development at APN 022-083-20 and 30, Fassler Avenue in Pacifica, California. The report includes the results of our subsurface exploration program and laboratory testing and presents our recommendations for site grading, retaining wall construction, building foundations, utility trench backfilling, and on-site pavements.

We thank you for the opportunity to perform these services. Should you have any questions or comments please contact our office.

Very truly yours,

BAY AREA GEOTECHNICAL GROUP

Ajay Singh
Senior Engineer

REGISTERED PROFESSIONAL ENGINEER AJAY SINGH No. C056647 Exp. 06/07 CIVIL STATE OF CALIFORNIA

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Distribution: 6 copies addressee

REPORT

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
APN 022-083-20 AND 30 (11 ACRES)
FASSLER AVENUE, PACIFICA, CALIFORNIA
For Home Pride Construction

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REPORT

**GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
APN 022-083-20 AND 30 (11 ACRES)
PACIFICA, CALIFORNIA
For Home Pride Construction**

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation performed for the proposed residential development at APN 022-083-20 and 30 (11 acres) in Pacifica, California. The attached Plate 1, Vicinity Map, shows the general location of the site. Plate 2, Site Plan, shows the site topography and the approximate locations of our exploratory borings drilled and test pits excavated for this investigation. These services were performed in accordance with the scope of services outlined in our Proposal No. 05-121, dated February 15, 2005.

2.0 PROJECT DESCRIPTION

The proposed residential development will include construction of 34 new homes in the western portion of the subject 11-acre hilly site. Seventeen of these homes (Units 13 through 29) will be constructed on the top of an underground parking garage, the remaining 17 homes will be constructed in the areas around the underground parking garage. Each of the proposed homes will have a 700 square foot building footprint. The homes proposed to be constructed above the underground parking garage will have a 400 square foot yard, whereas the homes around the parking garage will have a 1200 square foot yard area. A community room and picnic area will be constructed near the entrance and a small paved play area is planned for the area adjacent to the northern edge of the parking garage.

Preliminary plans indicate the lower floor elevation within the proposed parking structure will be at about 298 feet in the west and 295 feet in the eastern half. The parking level in the eastern half of the garage will be raised to elevation 303 feet, with the lower area in the eastern half of the garage used for service areas, ventilation ducts, and water tanks. The upper deck (or roof) of the garage structure, where 17 living units will be constructed, will also be split level, at elevation 308 feet in the west and 316 feet in the eastern half.

The floor elevations and grading details for the detached homes around the parking garage are not known at this time; however, it is anticipated that all existing fill will either be removed and/or reconstructed as engineered fill.

The entrance to the homes constructed on the parking garage deck slab will be via a main access road connecting the parking garage to Fassler Avenue. The access to the homes on the western side of the parking garage will be by a road connecting to the main access road, whereas access to the residential units to the east of the parking garage will be by another road parallel to the western road but at an uphill location. Additional development at the site will include construction of community gardens, a play area, and picnic area.

The proposed site development will involve relatively deep cuts to accommodate the parking garage and up to 13 foot high retaining walls. The actual cut and fill volumes were not available at the time this report was issued. The new residential structures around the parking garage will most likely be supported on a drilled, cast-in-place reinforced concrete pier and grade beam type foundation system. The parking garage structure will likely be constructed on a concrete structural mat slab.

3.0 PURPOSE

The purpose of this investigation was to obtain pertinent information regarding the geologic features and the soil conditions at the subject site in order to develop geotechnical recommendations for the design and construction of the proposed residential development. Based on our understanding of the project, this report presents our conclusions, opinions, and recommendations regarding:

- Site history, regional geology, and site geology including any landslides, undocumented fills, etc.,
- Specific soil conditions discovered by our borings (such as loose, expansive, creeping, or collapsible surface soils) that may require special mitigation or impose restrictions on the project, including the type, consistency, and suitability of any on-site soils,
- Evaluation of stability of existing slopes and proposed cut and fill slopes,
- Recommendations for site grading and pavement design of new roads and driveways.
- Retaining wall options and design recommendations,
- Recommendations for the most appropriate foundation type necessary to support the vertical and lateral loads associated with the proposed structures.
- Recommendations for utility trench backfilling,
- Estimates of post-construction settlement under the anticipated loads,
- Provisions for surface and subsurface drainage.

4.0 SCOPE OF SERVICES

The scope of our services consisted of the following specific tasks:

- Visit the site and mark the boring locations at least 72 hours in advance of the planned explorations and notify Underground Service Alert (USA) to mark the known utilities entering to the building and/or within the site.
- Review published geologic maps and reports pertinent to the site area regarding the seismic and geologic history of the site.
- Drill, log, and sample 5 exploratory borings and excavate up to 10 test pits to evaluate subsurface soil and groundwater conditions. The soil borings were drilled to depths of 24 to 29½ feet, as shown on Plates 12 through 16, with a track mounted rig equipped with hollow stem augers. The borings were drilled under the supervision of one of our engineers who directed the exploration and obtain samples of

the subsurface materials for visual classification and laboratory testing. The BAGG engineer also recorded the penetration resistance values and the type of sampler used for collection. He also measured the depth to groundwater in the boreholes, as encountered. At the completion of drilling activities, the boreholes were backfilled with neat cement.

- In addition to the soil borings, 9 test pits were excavated at the site at the locations shown on Plate 2, Site Plan. The subsurface soil conditions encountered at the test pit locations were logged by a certified engineering geologist and are shown on Plates 16 through 19. The test pits were loosely backfilled with the soil previously excavated from the pits.

- Perform a laboratory testing program on the collected soil samples to evaluate some of the index properties and engineering characteristics of the subsurface materials. Tests included direct shear strength, Plasticity Index, and moisture-density measurements.

- Perform engineering analyses based on the results of the tasks listed above and oriented towards the above-described purpose of the investigation.

- Prepare six copies of a report describing the geologic and seismic setting of the site, procedures and results of the subsurface exploration program, results of the laboratory testing program, subsurface soil and groundwater conditions, and geotechnical engineering recommendations for the proposed development.

5.0 SITE DESCRIPTION AND HISTORY

The subject site is located on the eastern side of Fassler Avenue near Roberts Road, in Pacifica, California. The subject 11-acre site has a hilly terrain with site elevations ranging from roughly 440 feet above mean sea level (MSL) in the southeastern portion to about 240 feet in the northwest corner. The western portion of the site will be developed for the proposed residential buildings. About 50 foot high ridge exists in approximately the middle portion of the property, roughly parallel to the Fassler Avenue.

The parcel and adjoining land is vacant and contains thickets of brush and shrubs of chaparral varieties. Access is at the southern edge of the site from Fassler Avenue. Remnants of an old road were noticed along the northern boundary of the site. It is our understanding that a quarry once existed at the subject site. Comparison of a series of aerial photographs dating back to 1943 indicates that the site area was graded and filled as part of the lower staging or loading area for a local quarrying operation that was active about 50 years ago.

The property occupies part of a broad ridge that extends downward in a northwesterly direction toward the coast from residential development within the higher reaches of Fassler Avenue, which follows the ridge crest in this area. A short distance southeast of the site, Fassler Avenue is bounded by steep, high cuts that were originally excavated by the quarry operation. The lower portion of the quarry land in the site area was graded and terraced into its present topography as a result of excavations and fills from those operations. Fills form steep banks around the outer edges of the terrace. Various low mounds of fills and boulders exist across the center of the site, which has been used in the recent past for dumping of logs, stumps, wood and other debris.

Fassler Avenue originally climbed from Highway 1 and terminated in the lower portion of the quarry land at the site area. After the quarry was abandoned, the road was extended through the central area of the property to access the upland areas to the east. That old, paved road is now abandoned, but is still visible on site. The present alignment of Fassler Avenue now forms the south boundary of the site project and extends uphill through an old quarry cut toward residential developments to the east.

The following stereo-paired aerial photographs were reviewed at the U.S. Geological Survey in Menlo Park, California:

Photo Nos	**Date**	**Scale**
DDB-2B-197 and 198	10/11/43	1:20,000
DDB-1R-8 and 9	5/27/56	1:20,000
GS-VBZJ-1-6 and7	4/16/68	1:30,000
7-24 and 25	6/7/74	1:20,000

6.0 FIELD EXPLORATION AND LABORATORY TESTING

To address the geotechnical aspects of the subject project, we conducted a subsurface exploration program at the site which consisted of drilling 5 soil borings and excavating 9 test pits. The soil borings were generally placed within and around the proposed underground parking garage. The test pits were excavated in the areas around the parking garage. The total depths of the soil borings and the test pits are shown on the attached logs of soil borings and test pits. Bulk, Standard Penetration Test, and undisturbed samples of the subsurface materials were obtained at 3- to 5-foot-intervals from the borings as necessary for visual classification and laboratory testing. A laboratory testing program was then designed and conducted on the samples collected from the borings to evaluate some of the index and strength properties of the subsurface materials encountered in the soil borings.

The materials encountered in the borings, and the results of laboratory tests are shown on the boring logs included in this report as Plates 12 through16. The test pit logs are shown on Plates 17 through 19. The legends for the symbols used on the boring logs and other soil/rock terminology on the boring logs are shown on Plates 5 through 9.

Selected undisturbed samples were tested in direct shear to evaluate the strength characteristics of the foundation soils. Tests were performed at natural moisture contents and under various surcharge pressures. The moisture content and dry density of undisturbed samples were measured to aid in correlating their engineering properties. The results of our laboratory strength tests and moisture-density data are summarized on the boring logs.

7.0 GEOLOGIC SETTING

The proposed residential development is situated atop a northwest-trending ridge that extends into the ocean at a rocky point located about 2/3 of a mile west of the project, between Rockaway Beach and San Pedro Valley within the City of Pacifica in northwestern San Mateo County, California. The site is at an average elevation of about 300 at the top edge of the steep-sided northern wall of an unnamed local valley that drains the project parcel area and upland regions to the northeast into the mouth of Clara Valley and the Pacific Ocean at Rockaway Beach.

This locality is within a system of steep-sided ridges that form the rugged northern terminus of the Santa Cruz Mountains. Geologic maps of the region (Pampeyan, 1994 and Brabb and Pampeyan, 1983) indicate the ridge of the site vicinity to be composed almost entirely of Franciscan Assemblage sheared rocks of the of Jurassic to Cretaceous age (150 million years). Outcrops and inclusions of sandstone, greenstone and limestone exist throughout the area. Residual soils, colluvium and slope wash materials that are derived from weathering and mass-wasting of the underlying bedrock, mantle much of the hillside areas below the site. A unit of manmade debris and poorly consolidated fill, primarily spoils from past quarrying operations in the vicinity, is within the site area. Some local landslide deposits exist within colluvium-filled ravines below the site. Plate 3, Site Geology Map, shows the locations of various geologic features mapped at the site and the locations of the geologic cross-sections are the geologic cross-sections are included herein as Plate 4.

Hillsides flanking the deeply incised valleys in the area are moderately dissected by erosion and display numerous well-developed drainages and more subtle linear swales that extend down the slopes. Landslides and debris flows originating in colluvium-filled swales and drainages are a recognized hazard in the Pacifica area. An exceptionally heavy storm in January, 1982 triggered hundreds of such slope failures in the steep terrain of Pacifica.

The structural grain of this region is influenced by the northwest-trending San Andreas fault, one of the major, active strike-slip faults of the world. The fault slices northwesterly through San Mateo County and crosses the northern edge of the City of Pacifica before going offshore in the vicinity of Mussel Rock. The northwest-trending Pilarcitos Fault, considered a pre-historic trace of the San Andreas, enters the ocean at San Pedro Valley on the southern edge of Pacifica. Several short fault segments have been mapped in Franciscan Assemblage rocks in the mountainous region between the San Andreas and Pilarcitos faults, but none have been mapped, or are known to exist, within the site area.

8.0 SEISMICITY

The site locality, on the northern San Francisco Peninsula, is within one of the more seismically active areas of the world. Numerous earthquakes, generated by any of a number of active faults in the region have been recorded during the short historical record of the area. Recent studies indicate that there is a 62 percent probability that one or more earthquakes of magnitude 6.7, or greater, will occur before the year 2032 on a fault segment in the San Francisco Bay region (Working Group, 2003).

The regional stress field of the San Andreas fault, a northwesterly-trending, major earthquake-producing feature located about 2.7 miles northeast of the site, extends inland from the Pacific Coast for a distance of about 60 miles and includes several active faults and branches thereof. The San Andreas fault generated the great San Francisco Earthquake of 1906, which caused considerable damage throughout the region. In 1957, a Magnitude 5.3 earthquake occurred on the fault near Pacifica, causing local landsliding and some structural damage.

The northern end of the San Gregorio fault zone, considered active in southern San Mateo County, lies in the ocean about 2.4 miles southwest of the site. The Pilarcitos fault, thought to be an ancestral trace of the San Andreas fault and possibly seismically active based on seismic evidence near its southern end (Brabb and Olson, 1986) is approximately one mile to the south. Several other faults in the region, including the Hayward and Calaveras faults, which have generated large earthquakes in historic time, are somewhat more distant, but have the potential for generating earthquakes that could impact the subject site.

Due to its historic activity and proximity to the site, the San Andreas fault is considered the primary seismic hazard from an earthquake-generating standpoint. Maps of shaking intensity that have been modeled for a magnitude 7.2 earthquake on the Peninsula - Golden Gate segment of the San Andreas fault have been made available by the Association of Bay Area Governments (ABAG, 2004). They indicate that the site area would likely experience "very strong to violent" shaking (Modified Mercalli Intensity VII to IX) during such an event.

The conclusions of the Working Group on California Earthquake Probabilities (2003) assign a 21 percent probability for one or more a major earthquakes (magnitude 6.7 or greater) on the peninsula segment of the San Andreas fault by 2032. The probability for a similar scenario on the San Gregorio fault is 10 percent. The major fault features of the San Francisco Bay area, in relation to the site, are illustrated on the attached Regional Fault Map, Plate 5.

9.0 SITE CONDITIONS

9.1 Subsurface Conditions

Plate 2, Site Plan shows the approximate locations of the soil borings and the test pits. The materials encountered during our subsurface exploration were generally consistent with the materials mapped on the referenced Geology Map.

Our site area reconnaissance and subsurface explorations have determined that benches graded for old roads, trails and cuts in the areas proposed for development in the northern area of the property have several feet of manmade fills that were constructed or cast over the outer, downslope side of the benches. The fills generally appear to be unengineered, poorly sorted, gravelly silts and clays derived from old quarry debris. Locally, some fills appear to be remnants of old quarry stockpiles. Others may represent berms that were placed at the edges of graded areas for runoff control. Exploration in the southern half of the development encountered some areas of relatively deep fills underlain by residual soils and bedrock, and other areas of near-surface bedrock in locations of apparent cuts or natural ground.

Bedrock is rarely exposed at the surface. However, our subsurface exploration indicates that the site is entirely underlain by bedrock of the Franciscan Assemblage. The rock appears to be a mix of sandstone, siltstone or shale and greenstone. In most cases the bedrock was deeply weathered and could be excavated or drilled relatively easily in at least the upper 10 or 15 feet. Other than man-made fills, surficial deposits on the site consist of residual soils developed in place by disintegration of the underlying rock, or colluvial materials deposited by surface runoff and erosion toward the base of slopes and in downslope swales around the north and west-facing perimeter of the property.

The soil borings indicate the presence of top soil over intensely weathered, soft, closely bedded, sandstone (bedrock). The depth of the bedrock varied across the site. At the location of Boring B-1, the depth to bedrock was estimated to be about 20 feet. The depth to bedrock ranged from approximately 4 feet at B-2 to 12 feet at B-5. The varying depth to bedrock observed in the borings may be related to the old quarry operation. It is possible that some of the old quarry cuts were backfilled with top soil when the quarry operation was halted. The details of the subsurface materials encountered at the site are shown on the boring and the test pit logs included in this report as Plates 11 through 19. The geologic conditions encountered at the site are shown on Plates 3 and 7.

9.2 Groundwater

Free groundwater was not encountered in the borings drilled for this investigation, except at the location of Boring B-1 where it was encountered at about 20 feet below the existing ground surface in Boring B-1. It should be noted that groundwater levels will fluctuate as a result of seasonal changes and perched water may develop in the rainy season, particularly in the bedrock fractures.

10.0 DISCUSSION AND RECOMMENDATIONS

10.1 General

Based on the subsurface exploration conducted at the subject site and the results obtained from our laboratory testing program, it is our opinion that the proposed residential development is feasible from a geotechnical engineering standpoint, provided the recommendations presented in this report are incorporated into the project design and implemented during construction. When the final development plans become available, they should be submitted to BAGG for a review prior to construction. The purpose of the plan review is to evaluate whether the project foundation and grading plans meet the intent of our recommendations, as well as evaluate that our recommendations properly address the project in its final form.

Although an exceptional rainstorm in January, 1982 caused mobilization of almost 500 landslides within the City of Pacifica (Howard-Donley and Associates, 1982), no recent slope failures are in evidence on the subject site. Our examination of morphological features utilizing stereo-paired aerial photographs taken in 1943, before the natural topography was obscured by manmade alterations of the area, did not indicate any gross slope instabilities in the site area. There are, however some apparent areas of local landsliding at the head of a broad, colluvium-filled ravine that extends downslope from the northern edge of the site property. These could be potential sources of future debris-flow activity below the upper edges of the northern side of the site. Under adverse drainage conditions, the heads of these features could eventually encroach upward, toward the outer edges of the proposed development.

The final site grades were not available at the time this report was issued. BAGG will evaluate the stability of the proposed site slopes once the site grading plan becomes available.

Based on the results of our field exploration and laboratory tests, it is our opinion that the proposed foundation loads of the parking structure may be supported on structural mat foundations. Prior to placing the concrete mat, it will be necessary to remove all the loose fill soil from within the slab area, and place it back as properly compacted fill. The residential structures around the parking structure may be supported on drilled cast-in-place, reinforced concrete piers with interconnecting grade beams.

The site development will include construction of relatively high retaining walls. The retaining walls associated with the parking garage may be constructed as cantilever-type walls. The retaining wall foundations may be combined with the structural mat slab. The retaining wall design recommendations are included in the following sections of the report.

The site could experience very strong ground shaking from future earthquakes during the anticipated lifetime of the project. The intensity of the ground shaking will depend on the magnitude of the earthquake, distance to the epicenter, and the response characteristics of the native soils and bedrock materials. While it is not possible to totally preclude damage to structures during major earthquakes, strict adherence to good engineering design and construction practices will help reduce the risk of damage to the proposed development. The Uniform Building Code defines the minimum standards of good engineering practice.

The subject site is located in a hilly area. Therefore, it is important that the project Civil Engineer familiarizes himself with the topography of the site and the vicinity area to design a surface water control system to minimize future erosion. The cut and fill slopes should be no steeper than 2 to 1 (horizontal to vertical) without the approval of the project Geotechnical Engineer.

10.2 Site Grading

Site grading will involve major cuts and fills to create room for the underground parking lot. The on-site slopes are fairly steep, therefore, we do not recommend placement of fills on the existing steep slopes at the site without following the recommendations included in the sections below or without properly retaining it with walls designed to withstand the lateral loads.

As used in this report, the term "compact" and its derivatives mean that all on-site soils and bedrock materials should be compacted to at least 90 percent of maximum dry density, as determined by ASTM Test Method D1557-01, while at a moisture content that is slightly over optimum. In the slab and pavement areas, the top 6-inches of the subgrade, including aggregate base, should be compacted to at least 95 percent of the maximum dry density.

The following grading procedures should be followed in the building areas.

- Remove brush, trees, roots, and debris from the areas proposed for development. The trees and roots should be removed to a minimum depth of 6 inches below the existing grade.
- Existing utilities where known should be located on the grading plan so that the project engineer could evaluate the necessity of removing abandoned pipes during site grading
- Holes/depressions created by the removal of brush, roots, existing utilities, or old foundations should be backfilled with engineered fill placed in thin lifts not exceeding 8-inches in loose thickness compacted to a minimum of 90 percent relative compaction at above optimum moisture content.

- The test pits excavated for this study were backfilled with loosely placed soil cuttings previously removed from the pits. Prior to construction, the loosely placed material in the test pits should be removed and placed back as compacted fill.

- Acceptable structural fill at the site should be essentially non-expansive (Plasticity Index of less than 15), free of debris, organic and deleterious material, have a maximum rock size of 4-inches in diameter, and have fines content of between 15 and 65 percent. On-site material free of organics and debris is likely to meet the requirements of acceptable structural fill. A sample of the import fill, if planned, should be delivered to the Geotechnical Engineer for testing and approval prior to importing to the site.

- The fills placed on sloping ground must be keyed into firm, native materials. Keyway depths and locations will be determined by the Geotechnical Engineer. A typical keyway schematic is Shown on Plate 21. The keyway should be approximately 15 feet wide (1½ times the width of the equipment). The bottom of the keyway should be inclined towards the slope. Subsurface drainage should be installed in the keyway. The typical construction details of a subdrain are shown on Plate 21. Subdrains should be connected at their low points to storm drain system or other approved drainage facilities. Subdrain outlets should be protected from erosion and siltation.

10.3 1997 Uniform Building Code Site Characterization

The Structural Engineering Design Provisions in Chapter 16 of the 1997 Uniform Building Code (UBC) introduced substantial changes to earthquake design for new buildings (International Conference of Building Officials, 1997). The new code considers local (near source) seismic affects. It uses "near-source factors" to account for the fact that recorded near-fault ground motions and load requirements on structures have frequently exceeded those specified in earlier editions of the UBC. Based on our geologic research, including published maps of known active fault zones prepared for the 1997 UBC and the distance to the seismic sources, the seismic design parameters per Chapter 16 of the 1997 UBC are listed below. The seismic design parameters per 2001 California Building Code are similar to those listed below.

UBC 1997 defines Type A faults as Faults which are capable of producing large magnitude events and that have a high rate of seismic activity (slip rate greater than 5 mm/year) (Table 16-U - Seismic Source Type). Type C faults are defined as faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity. Type B faults are defined vaguely as all faults other than Type A and Type C faults.

Soil Profile S_C is defined as "very dense soil and soft rock" with an average shear wave velocity of 1200 to 2500 feet per second, average Standard Penetration Test (N) values greater than 50, and average undrained shear strength greater than 2000 psf for the top 100 feet of the profile (UBC 1997 - Table 16-J).

The location of the subject site relative to the nearby Type A and B faults is shown on Plate 4, Regional Fault Map. The seismic design parameters per UBC 1997 are listed below. Please note, that these values represent the minimum design standards.

TABLE 1

SEISMIC DESIGN PARAMETERS PER UBC 1997

UBC, 1997	Site Parameter
Figure 16-2, Seismic Zone Map of the U.S.	Zone 4
Table 16-I, Seismic Zone Factor Z	0.4
Table 16-Q, Seismic Coefficient C_a	0.53
Table 16-R, Seismic Coefficient C_v	0.98
Table 16-J, Soil Profile Type	S_C, Soft Rock
Closest Type A Fault	San Gregorio
Distance to Closest Type A Fault	3.8 kilometer
Table 16-S, Near-Source Factor, N_a	1.3
Table 16-T, Near-Source Factor, N_v	1.8

10.4 Pier and Grade Beam Foundations

The residential structures proposed to be constructed around the parking garage may be adequately supported on a drilled, cast-in-place, reinforced concrete friction pier and grade beam foundation system. The drilled piers should have a minimum diameter of 18 inches. The end bearing capacity of the piers should be ignored in the design of the piers. The foundation piers should extend past the surface soils and a minimum of 8 feet into competent bedrock, as determined by the Geotechnical Engineer in the field. The pier depth, diameter, and reinforcement should be determined by the project structural engineer; however, we recommend each pier should be reinforced with a minimum of four #5 bars. The piers may be sized using an allowable skin friction value of 600 pounds per square foot (psf) within the bedrock for downward as well as the uplift forces. Where the drilled piers are located horizontally within 5 feet of the existing slope face, the allowable skin friction value should be ignored in the upper 3.0 feet, and the piers should be designed to withstand creep load of 50 pcf acting on the embedded portions of the grade beams and piers. The skin friction values provided above may be increased by one-third for short-term seismic and transient loads. The bottom of the piers should be cleaned of all loose soil cuttings before placement of reinforcing steel.

A passive resistance of 300 pcf, up to a maximum of 3,000 pcf, over two times the diameter of the pier could be used to resist lateral loads. Since in many cases, the ground surface against the piers will be sloping, the passive resistance from the top 3.0 feet of the pier should be ignored, unless the ground surface extending 5 feet beyond the pier is flat.

Loads between piers should be supported on grade beams that are designed to span between the piers with the assumption that they obtain no vertical support from soils beneath them. It is recommended that the exterior grade beams be established a minimum of 12 inches below final grade. Pier and grade beams should be reinforced appropriately and the reinforcement should be properly tied together to enable the entire system to act as a unit. Design of the grade-beams, including the size, spacing, and shape of the reinforcement should be determined by the project Structural Engineer responsible for the foundation design.

The bottom of pier and grade beam excavations should be firm, clean, and relatively free of any loose or disturbed soils and bedrock materials. There is a small possibility that groundwater may be encountered in some of the pier excavations. Where encountered, the groundwater should be pumped out immediately before pouring concrete, or the concrete should be tremied into the hole. The end of the tremie pipe should be kept near the bottom of the hole at the beginning and kept a minimum of 2 feet below the top of concrete as the concrete is being placed in the hole. The concrete placement operation should only be terminated when the mixture of water and sloughage is completely removed. The area around the pier hole should be cleaned and the concrete spilled around the pier hole should be removed.

10.5 Concrete Mat Foundation

The proposed parking garage loads may be supported on a concrete mat foundation system constructed on a prepared building pad consisting of undisturbed native materials and/or properly compacted and tested fill soils. The mat foundation system could be designed for an allowable bearing pressure value of 2,000 psf. This value may be increased by one-third for short term loads such as wind and seismic loads. The mat slab should be sufficiently thick to uniformly spread all concentrated loads. The mat slab should be designed for a modulus of subgrade reaction value of 200 psi per inch. This value is for one-foot square area and the size of the mat has not been taken into consideration.

Lateral resistance may be provided by friction between the concrete mat slab and the underlying soil and passive pressure between the foundation and the adjacent soil. The base friction may be calculated using a coefficient of friction value of 0.35. The passive pressure may be calculated using an equivalent fluid weight of 350 pcf, up to a maximum of 3,500 psf.

10.6 Settlements

We have estimated that the total post construction, static settlements of the proposed structures supported on properly constructed pier and grade beam foundations will be negligible. The total settlement of the parking garage structure supported on concrete mat foundation will be less than 1-inch with differential settlement of less than ½-inch.

10.7 Retaining Walls

Retaining walls should be designed to resist lateral earth pressures from natural materials and backfills. Restrained walls for the parking garage, supporting native materials or compacted fill soils should be designed to resist at rest lateral pressures taken as an equivalent fluid pressure of 60 pounds per cubic foot (pcf) for level backfill. The unrestrained retaining walls across the site should be designed to resist an active lateral pressure taken as an equivalent fluid pressure of 45 pounds per cubic foot for level backfill. These pressures should be increased by 3 pcf for every 5 degrees increase in backfill slope.

The above lateral pressures do not include hydrostatic pressures resulting from groundwater, seepage water, or infiltration of natural rainfall and/or irrigation water behind the walls. Therefore all walls over 3 feet in height should be provided with a drainage blanket behind the wall. The drainage blanket should consist of a pre-manufactured drainage panel or one foot thick blanket of Caltrans Class 2 Permeable rock or free-draining gravel or drain rock encapsulated by a suitable filter fabric. A 12-inch cap of relatively impermeable soil should be placed at the top of the drainage blanket to minimize infiltration of surface water. The cap material should be compacted to 90 percent relative compaction. A 4-inch diameter perforated PVC pipe should be installed at the base of the drainage blanket to facilitate removal of water collected behind the wall. A schematic showing fill placement behind retaining walls is presented herein as plate 22.

The retaining walls for the parking garage may be supported on the mat foundation. Retaining walls in other portions of the site should be supported on drilled, cast-in-place, reinforced concrete piers designed as recommended above for foundations, except that the depth of the retaining wall piers should be determined using the UBC flagpole formula. However, as a minimum the piers should extend 8 feet into bedrock type material.

The retaining walls may be backfilled with on-site material free of debris, organics, and deleterious materials. The backfill material should be compacted to a minimum of 90 percent relative compaction while at above optimum moisture content.

BAGG should be retained to review the design calculation and the final retaining wall design plans to evaluate compliance with the intent of the recommendations included in this letter report. BAGG should be retained to provide observations and soil compaction testing services during pier drilling, and placement of backfill materials.

The surcharge loads behind the retaining wall may be treated as uniform loads (rectangular distribution) over the entire depth of the wall with pressure equal to one-half the vertical surcharge load for restrained conditions and one-third the surcharge load for unrestrained condition.

Permanent tiebacks may be used to resist lateral forces associated with the retaining walls. The tiebacks should extend a minimum of 5 feet beyond a plane inclined at an angle of 30 degrees from the back side of the wall. The tiebacks may be designed using an allowable skin friction value of 1,000 psf. The tiebacks should be installed in clean holes with a minimum diameter of 8-inches. The tiebacks should be grouted in two stages. The first portion of the grout should extend from the bottom of the hole to the assumed failure plane. At this point, the tiebacks should be tested to at least 150 percent of the design loads. The design load should be held for a minimum of 10 minutes. The tiebacks should be located off at 110 percent of the design load capacity. If the design load capacity is not obtained, additional tiebacks should be installed at the locations approved by the Geotechnical Engineer. After testing the tieback to the design load, grout should be pumped into the remaining length of the hole.

The holes drilled for tiebacks should be dry and free of loose cuttings. If during drilling, wet or caving ground is encountered, adequate casing should be provided by the contractor. This casing should be removed in an approved satisfactory manner while the grout is being placed in the hole. We recommend that high strength grout be used in conjunction with clean, corrosion protected reinforcing bars.

10.7.1 Block-type Retaining Walls

Gravity retaining walls such as keystone walls, or Alan Block walls may be used in certain portions of the site to retain the lateral loads. The gravity retaining walls provide resistance to lateral loads through a combination of friction between the geogrid and the surrounding soil , and the frictional resistance between the blocks themselves. In general, the block type retaining walls are economical in situations where no additional cuts are required to accommodate the geogrid material behind the walls. The block type retaining walls are generally not very practical in the situations where a steep slope exists near the toe of the wall. In these cases, because of the lack of passive resistance, the foundation portion of the walls has to be extended deeper thus loosing the economical advantage. Since the site slopes are relatively steep, it is our opinion that block type retaining walls will only be suitable for only very small portions of the site. BAGG can evaluate the suitability of a block-type retaining wall at any particular location and perform design analysis for the retaining wall, if requested.

10.8 Slabs-on-Grade and Exterior Flatwork

Concrete slabs and flat work to be constructed at or near the ground surface should be supported on a 12-inch thick layer of reworked on-site soils and/or engineered fill that has been prepared and compacted as recommended under "Site Grading." The subgrade soils should be maintained at a moisture content of slightly over optimum, and should be approved by the Geotechnical Engineer immediately before the slab is poured.

It is recommended that the exterior slabs may be placed directly over compacted, and smooth-rolled soil subgrade. In moisture sensitive areas, the slabs should be underlain by 4 inches of compacted clean, crushed rock covered with a plastic vapor barrier with a minimum thickness of 10 mils. To aid in curing the concrete and to protect the membrane during construction, the vapor barrier should be covered with a 2-inch-thick layer of curing sand that should be wetted prior to pouring the slab.

10.9 Drainage

Because of the slope around the site, drainage measures to control and collect surface run-off should be considered an integral part of the proposed development. The ground surface adjacent to all sides of the proposed buildings should be sloped to drain away from the foundations. Unpaved and landscaped areas should slope at least 5 percent for a distance of at least 5 feet away from the face of the building. Surface drainage swales at the site should slope at least 1 percent toward a suitable discharge point. Runoff should not be allowed to flow over graded or natural slopes, and any area where surface run-off becomes concentrated should be provided with a catch basin. The run-off from building roofs should be collected in closed, non-perforated pipes and discharged to the local storm drain system, or discharged in a manner that will not allow ponding adjacent to foundations or erosion on native or graded slopes.

Surface and subsurface drainage facilities and catchment areas should be checked frequently and cleaned or maintained throughout the project life, as necessary.

10.10 On-Site Pavements

Design of new pavement sections are provided below. We have also included deep-lift asphalt sections for the new roadway and parking area pavements. The design is based on an R-value of 10 for the subgrade, an R-value of 78 for Class 2 aggregate base. We recommend that additional R-value tests should be conducted to confirm the R-value of the soil subgrade prior to construction of the roadway. The pavement design recommendations tabulated below are based on Traffic Indices of 4.5, 6.0, and 7.0. A Traffic Index of 6.0 is usually appropriate where the pavement will be subject to frequent use by vans or light delivery trucks with only occasional heavy truck traffic, and 7.0 for frequent use of the roadway by heavy trucks. A Traffic Index of 4.5 can be used for areas accommodating light automobile parking only.

TABLE 2
FLEXIBLE PAVEMENT SECTIONS
(Subgrade R-value=10)*

Pavement Component	TI = 4 .5		TI=6.0		TI=7.0	
Asphaltic Concrete (AC)	7	2.5	10	3.5	12	4
Imported Class II Aggregate Base (R_{Min}=78)	-	9.5	-	12.5	-	15.5
Total Thickness in Inches	7	12	9	16	12	19.5

*Will require confirmational R-Value testing during construction.

The alternative pavement sections presented above were calculated using the design method described in the Caltrans Highway Design Manual (Topic 608, July 1, 1995) with the safety factors included. The method characterizes the subgrade soil conditions with laboratory R-value tests, and characterizes the traffic loading conditions with a Traffic Index.

10.11 Utility Trenches

Vertical trenches deeper than 5 feet will require temporary shoring to protect workers in the trench. Where shoring is not used, the sides should be sloped or benched, with a maximum slope of 1:1 (horizontal : vertical). The trench spoils should not be placed closer than 3 feet (or one-half of the trench depth) from the trench sidewalls. All work associated with trenching must conform to the State of California, Division of Industrial Safety requirements. In our opinion, the soils at the upper 10 feet of the site should be classified as "Type B Soil."

Trench backfill materials and compaction should conform to the following:

- In general, soils used for trench backfill shall be free of debris, roots and other organic matter, debris, and rocks or lumps exceeding 3 inches in greatest dimension.
- Compaction shall be performed to a minimum of 90% relative compaction in accordance with ASTM D1557-01, at slightly above optimum moisture content. In pavement areas, the upper 12 inches of the backfill (below the pavement subgrade) shall be compacted to 95% of the maximum dry density.
- Jetting shall not be allowed.

10.12 Plan Review

It is recommended that BAGG be retained to review the final grading and foundation plans. This review is to assess general suitability of the earthwork and foundation recommendations contained in this report and to verify the appropriate implementation of our recommendations into the project plans and specifications.

10.13 Observation and Testing

It is recommended that BAGG be retained to provide observation and testing services during the grading, excavation, backfilling, and foundation construction phases of work. This is to verify that the work in the field is performed as recommended and in accordance with the approved plans and specifications, and more importantly, to verify that subsurface conditions encountered during construction are similar to those anticipated during the design phase.

11.0 CLOSURE

This report has been prepared in accordance with generally-accepted engineering practices for the strict use of Home Pride Construction and other professionals associated with the specific project described in this report. The conclusions and recommendations contained in this report are based on subsurface conditions revealed by widely spaced borings and test pits. It is not uncommon for unanticipated conditions to be encountered during site grading and/or foundation installation, and it is not possible for all such variations to be found by a field exploration program appropriate for this type of project. The recommendations contained in this report are therefore contingent upon the review of the final grading, drainage, and foundation plans by this office, and upon geotechnical observation and testing by BAGG of all pertinent aspects of construction including clearing, site grading, foundation excavations, placement of fills or backfills, and preparation of subgrades.

Surface conditions and standards of practice change with time. Therefore, we should be consulted to update this report, if the construction does not commence within 18 months from the date this report is submitted. Additionally, the recommendations of this report are only valid for the proposed development as described herein. If the proposed project is modified, our recommendations should be reviewed and approved or modified by this office in writing.

The following plates are attached and complete this report:

Attached Plates:

Plate 1	Vicinity Map
Plate 2	Site Plan
Plate 3	Site Geology Map
Plate 4	Geologic Cross-Sections
Plate 5	Regional Fault Map
Plate 6	Unified Soil Classification System
Plate 7	Soil Terminology
Plate 8	Rock Terminology
Plate 9	Boring Log Notes
Plate 10	Key to Symbols
Plate 11 through 15	Logs of Borings B-1 through B-5
Plate 16 through 19	Logs of Test Pits P-1 through P-9
Plate 20	Plasticity Chart
Plate 21	Schematic Showing Fill Placement on Sloping Ground
Plate 22	Schematic Showing Fill Placement Behind Retaining Walls

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

12.0 REFERENCES

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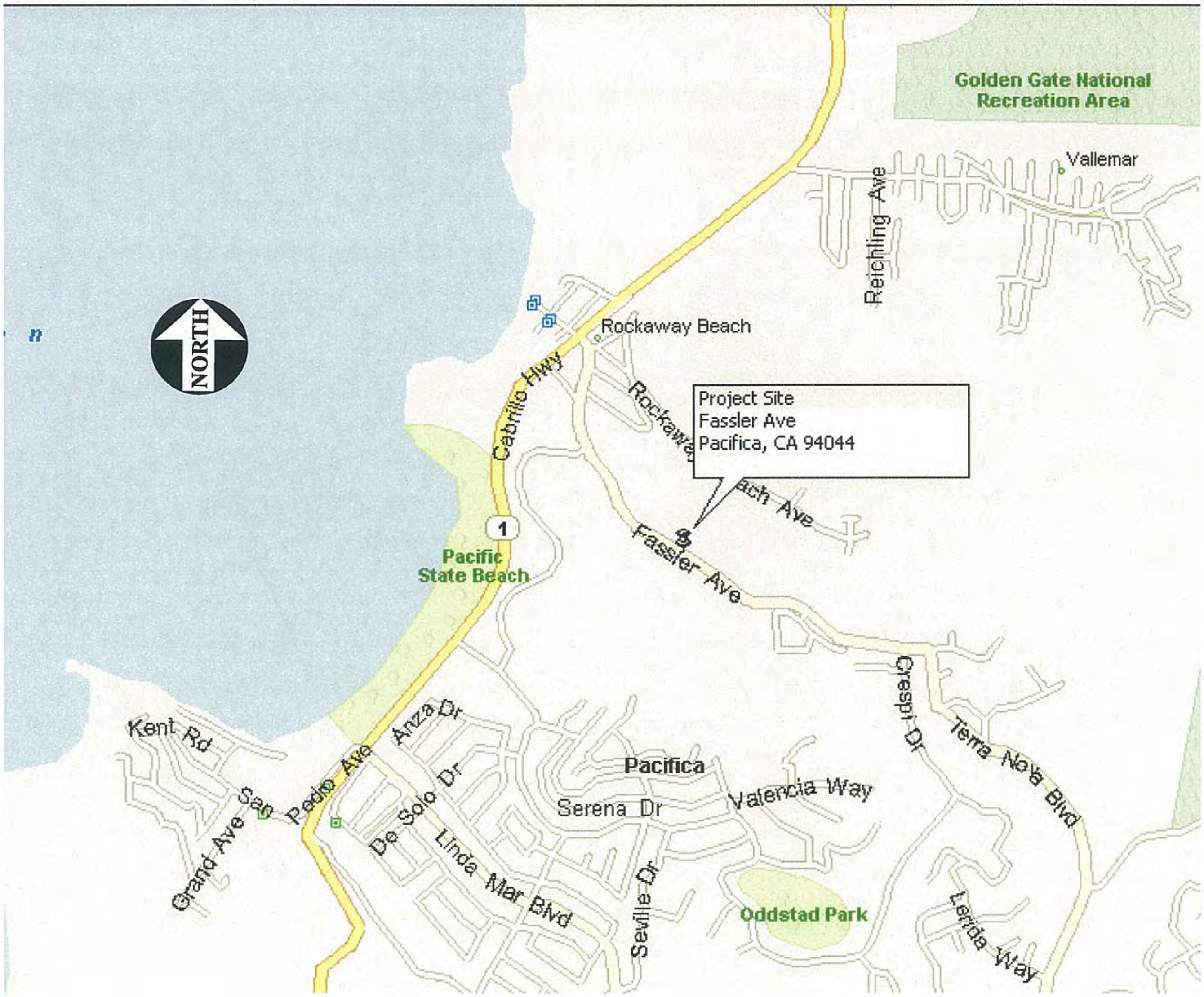


PROPOSED RESIDENTIAL DEVELOPMENT FASSLER AVENUE PACIFICA, CALIFORNIA	VICINITY MAP		
	JOB NO. PACIQ-01-00	DATE April 2005	PLATE 1

BAGG

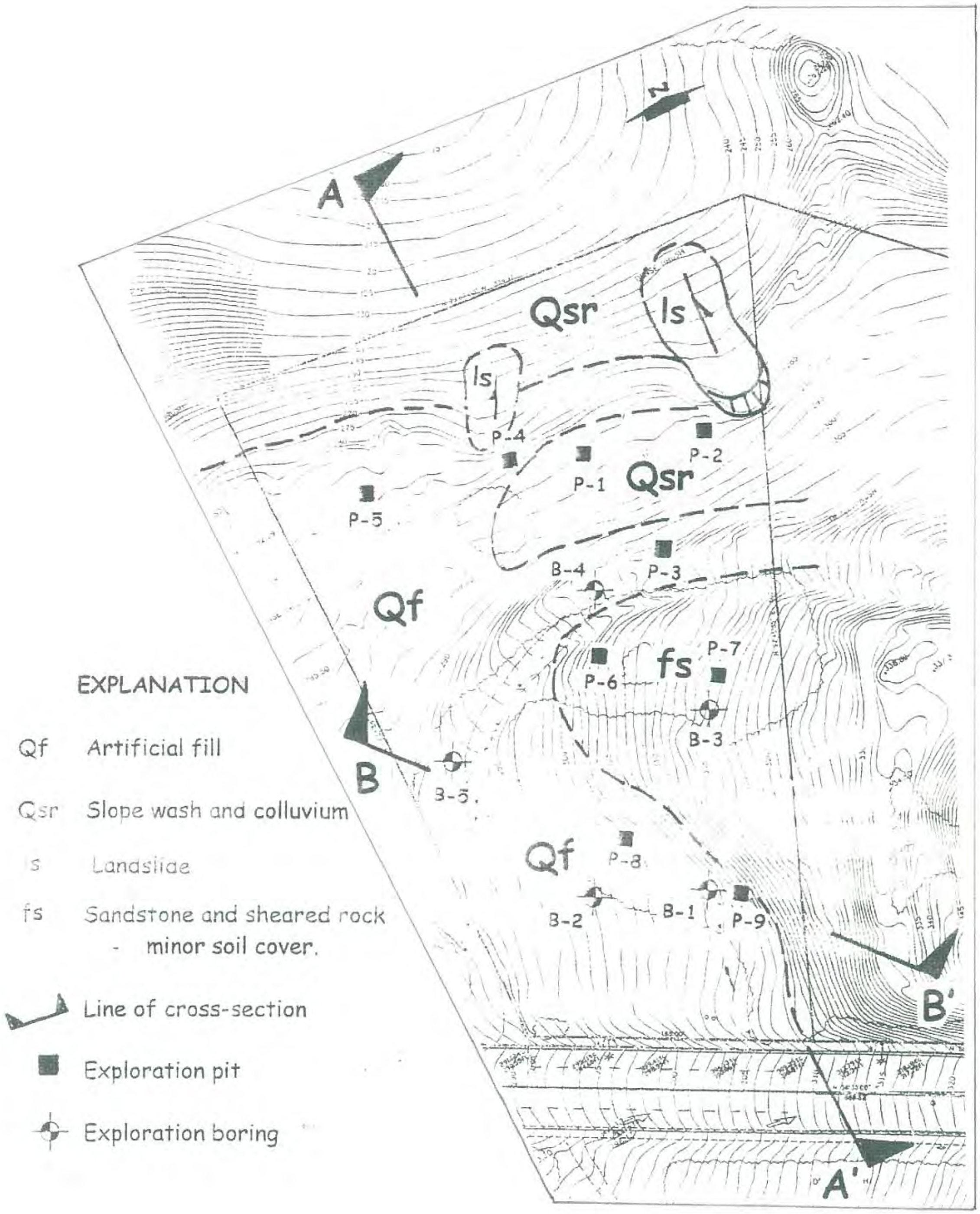
SOURCE: HOME PRIDE CONSTRUCTION

PROPOSED RESIDENTIAL DEVELOPMENT
FASSLER AVENUE
PACIFICA, CALIFORNIA

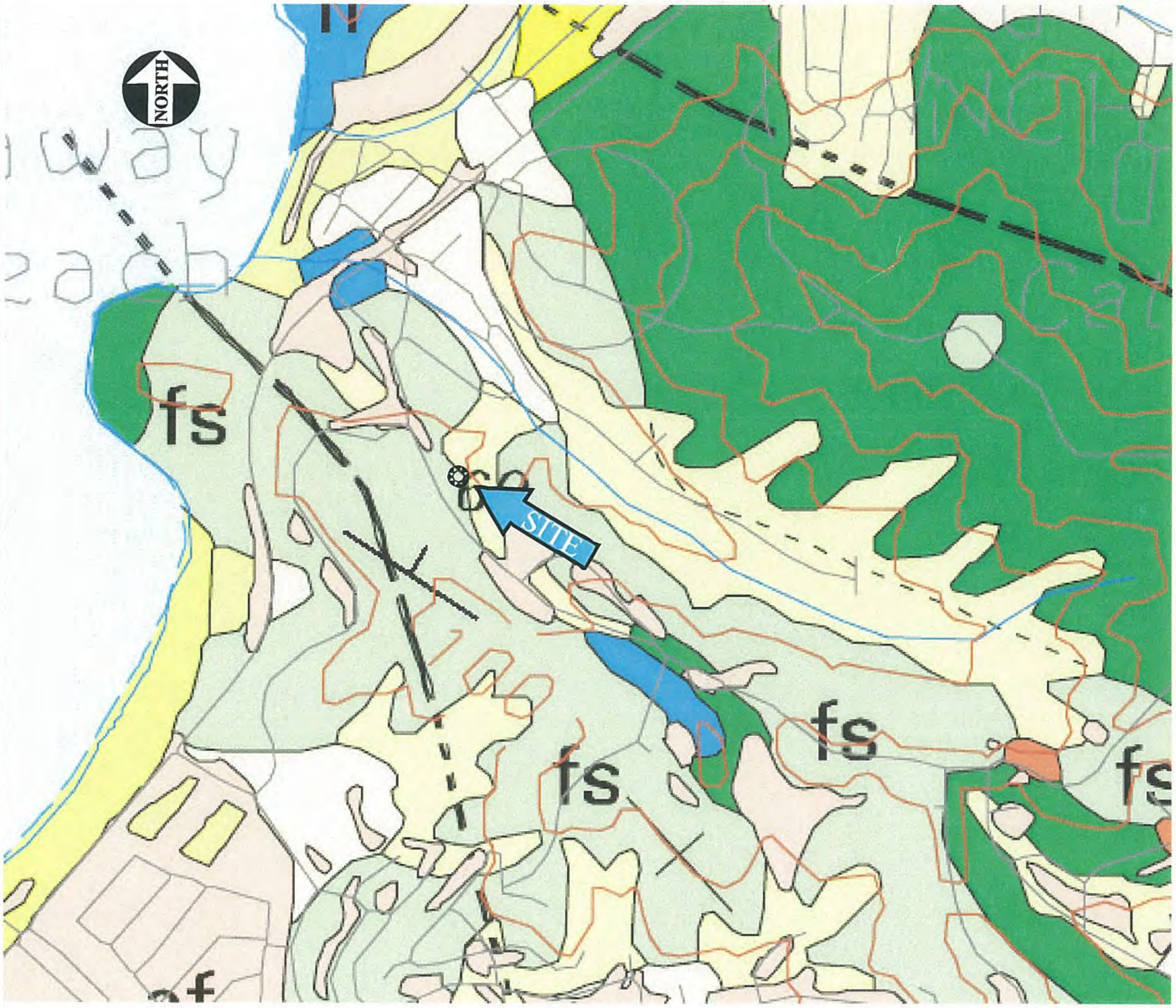
BAGG

SITEPLAN

JOB NO.	DATE	PLATE 2
PACIQ-01-00	4/21/2005	



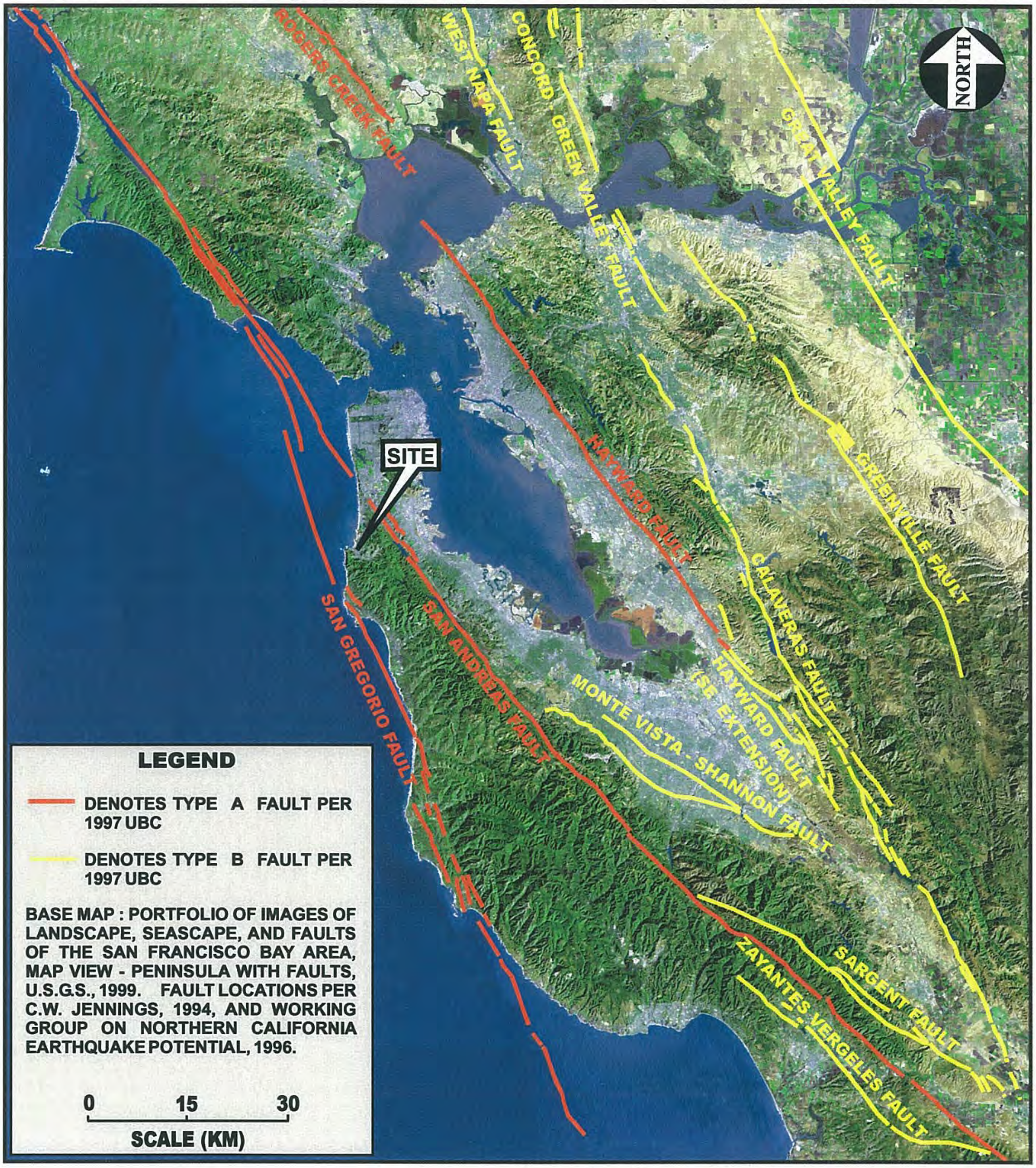
PROPOSED RESIDENTIAL DEVELOPMENT FASSLER AVENUE PACIFICA, CALIFORNIA	SITE GEOLOGY MAP			
	JOB NO. PACIQ-01-00	SCALE 1" = 100'	DATE April 2005	PLATE 3



Fs Sandstone - Greenish-gray to buff, fine- to coarse-grained sandstone (graywacke), with interbedded siltstone and shale. Siltstone and shale interbeds constitute less than 20 percent of unit, but in places form sequences as much as several tens of meters thick. In many places shearing has obscured bedding relations; rock in which shale has been sheared to gouge constitutes about 10 percent of unit. Gouge is concentrated in zones that are commonly less than 30m wide but in places may be as much as 150 m wide. Total thickness of unit is unknown but is probably at least many hundreds of meters.

Reference: U.S. Department of Interior PamphletDerived from Digital Map of 98-197 Geology of Onshore Part of San Mateo County, California: A Digital Database, by E.E. Brabb, R.W. Graymer and D.L. Jones, U.S. Geologic Survey, 1998.

PROPOSED RESIDENTIAL DEVELOPMENT FASSLER AVEUE PACIFICA, CALIFORNIA	REGIONAL GEOLOGY MAP		
	JOB NO. PACIQ-01-00	DATE April 2005	PLATE 4



PROPOSED RESIDENTIAL DEVELOPMENT **FASSLER AVENUE** **PACIFICA, CALIFORNIA**	**REGIONAL FAULT MAP**		
	JOB NO. PACIQ-01-00	DATE April 2005	PLATE 5

BAGG

COARSE-GRAINED SOILS
LESS THAN 50% FINES*

GROUP SYMBOLS	ILLUSTRATIVE GROUP NAMES	MAJOR DIVISIONS
GW	Well graded gravel Well graded gravel with sand	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size
GP	Poorly graded gravel Poorly graded gravel with sand	
GM	Silty gravel Silty gravel with sand	
GC	Clayey gravel Clayey gravel with sand	
SW	Well graded sand Well graded sand with gravel	SANDS More than half of coarse fraction is smaller than No. 4 sieve size
SP	Poorly graded sand Poorly graded sand with gravel	
SM	Silty sand Silty sand with gravel	
SC	Clayey sand Clayey sand with gravel	

NOTE: Coarse-grained soils receive dual symbols if:
(1) their fines are CL-ML (e.g. SC-SM or GC-GM) or
(2) they contain 5-12% fines (e.g. SW-SM, GP-GC, etc.)

FINE-GRAINED SOILS
MORE THAN 50% FINES*

GROUP SYMBOLS	ILLUSTRATIVE GROUP NAMES	MAJOR DIVISIONS
CL	Lean clay Sandy lean clay with gravel	SILTS AND CLAYS liquid limit less than 50
ML	Silt Sandy silt with gravel	
OL	Organic clay Sandy organic clay with gravel	
CH	Fat clay Sandy fat clay with gravel	SILTS AND CLAYS liquid limit more than 50
MH	Elastic silt Sandy elastic silt with gravel	
OH	Organic clay Sandy organic clay with gravel	
PT	Peat Highly organic silt	HIGHLY ORGANIC SOIL

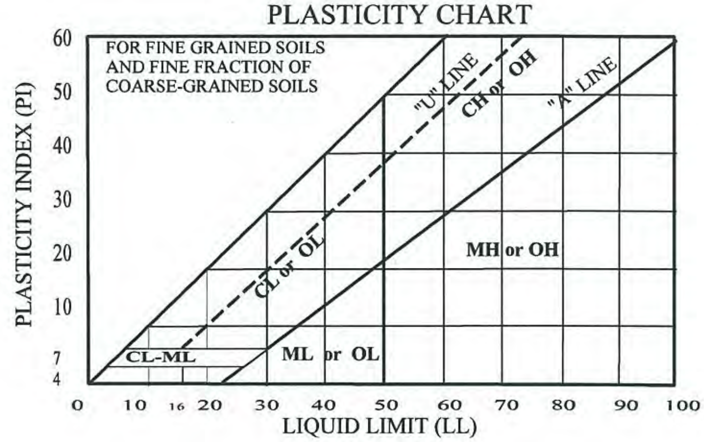
NOTE: Fine-grained soils receive dual symbols if their limits plot in the hatched zone on the Plasticity Chart (CL-ML).

SOIL SIZES

COMPONENT	SIZE RANGE
BOULDERS	ABOVE 12 in.
COBBLES	3 in. to 12 in.
GRAVEL	No. 4 to 3 in.
Coarse	¾ in to 3 in.
Fine	No. 4 to ¾ in.
SAND	No. 200 to No.4
Coarse	No. 10 to No. 4
Medium	No. 40 to No. 10
Fine	No. 200 to No. 40
*FINES:	BELOW No. 200

NOTE: Classification is based on the portion of a sample that passes the 3-inch sieve.

PLASTICITY CHART



Reference: ASTM D 2487-98, Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System).

GENERAL NOTES: The tables list 30 out of a possible 110 Group Names, all of which are assigned to unique proportions of constituent soils. Flow charts in ASTM D 2487 aid assignment of the Group Names. Some general rules for fine grained soils are: less than 15% sand or gravel is not mentioned; 15% to 25% sand or gravel is termed "with sand" or "with gravel;" and 30% to 49% sand or gravel is termed "sandy" or "gravelly." Some general rules for coarse-grained soils are: uniformly-graded or gap-graded soils are "Poorly" graded (SP or GP); 15% or more sand or gravel is termed "with sand" or "with gravel;" 15% to 25% clay and silt is termed clayey and silty and any cobbles or boulders are termed "with cobbles" or "with boulders."

UNIFIED SOIL CLASSIFICATION SYSTEM

SOIL TYPES (Ref 1)

Boulders: particles of rock that will not pass a 12-inch screen.
Cobbles: particles of rock that will pass a 12-inch screen, but not a 3-inch sieve.
Gravel: particles of rock that will pass a 3-inch sieve, but not a #4 sieve.
Sand: particles of rock that will pass a #4 sieve, but not a #200 sieve.
Silt: soil that will pass a #200 sieve, that is non-plastic or very slightly plastic, and that exhibits little or no strength when dry.
Clay: soil that will pass a #200 sieve, that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength when dry.

MOISTURE AND DENSITY

Moisture Condition: an observational term; dry, moist, wet, or saturated.
Moisture Content: the weight of water in a sample divided by the weight of dry soil in the soil sample, expressed as a percentage.
Dry Density: the pounds of dry soil in a cubic foot of soil.

DESCRIPTORS OF CONSISTENCY (Ref 3)

Liquid Limit: the water content at which a soil that will pass a #40 sieve is on the boundary between exhibiting liquid and plastic characteristics. The consistency feels like soft butter.
Plastic Limit: the water content at which a soil that will pass a #40 sieve is on the boundary between exhibiting plastic and semi-solid characteristics. The consistency feels like stiff putty.
Plasticity Index: the difference between the liquid limit and the plastic limit, i.e. the range in water contents over which the soil is in a plastic state.

MEASURES OF CONSISTENCY OF COHESIVE SOILS (CLAYS) (Ref's 2 & 3)

Very Soft	N=0-1*	C=0-250 psf	Squeezes between fingers
Soft	N=2-4	C=250-500 psf	Easily molded by finger pressure
Medium Stiff	N=5-8	C=500-1000 psf	Molded by strong finger pressure
Stiff	N=9-15	C=1000-2000 psf	Dented by strong finger pressure
Very stiff	N=16-30	C=2000-4000 psf	Dented slightly by finger pressure
Hard	N>30	C>4000 psf	Dented slightly by a pencil point

*N=blows per foot in the Standard Penetration Test. In cohesive soils, with the 3-inch-diameter ring sampler, 140-pound weight, divide the blow count by 1.2 to get N (Ref 4).

MEASURES OF RELATIVE DENSITY OF GRANULAR SOILS (GRAVELS, SANDS, AND SILTS) (Ref's 2 & 3)

Very Loose	N=0-4**	RD=0-30	Easily push a ½-inch reinforcing rod by hand
Loose	N=5-10	RD=30-50	Push a ½-inch reinforcing rod by hand
Medium Dense	N=11-30	RD=50-70	Easily drive a ½-inch reinforcing rod
Dense	N=31-50	RD=70-90	Drive a ½-inch reinforcing rod 1 foot
Very Dense	N>50	RD=90-100	Drive a ½-inch reinforcing rod a few inches

**N=Blows per foot in the Standard Penetration Test. In granular soils, with the 3-inch-diameter ring sampler, 140-pound weight, divide the blow count by 2 to get N (Ref 4).

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Ref 1: ASTM Designation: D 2487, **Standard Classification of Soils for Engineering Purposes** (Unified Soil Classification System).

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SOIL TERMINOLOGY

WEATHERING DESCRIPTORS

Fresh — No discoloration, not oxidized, no separation, hammer rings when crystalline rocks are struck.

Slight — Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull, no visible separation, hammer rings when crystalline rocks are struck, body of rock not weakened.

Moderate — Discoloration extends from fractures, usually throughout ;Fe-Mg materials are "rusty", feldspar crystals are "cloudy", all fractures are discolored or oxidized, partial separation of boundaries visible, texture generally preserved, hammer dose not ring when rock is struck, body of rock is slightly weakened.

Intense — Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation, all fracture surfaces are discolored or oxidized, surfaces friable, partial separation, texture altered by chemical disintegration, dull sound when struck with hammer, rock is significantly weakened.

Decomposed — Discolored or oxidized throughout, but resistant mineral such as quartz may be unaltered, all feldspars and Fe-Mg minerals are completely altered to clay, complete separation of grain boundaries, resembles a soil, partial or complete remnant of rock structure may be preserved, can be granulated by hand, resistant minerals such as quartz may be present as "stringers" or "dykes".

BEDDING FOLIATION AND FRACTURE SPACING DESCRIPTORS

Millimeters	Feet	Bedding	Fracture Spacing
>10	<0.03	Laminated	Very Close
10-30	0.03-0.1	Very Thin	Very Close
30-100	0.1-0.3	Thin	Close
100-300	0.3-1	Moderate	Moderate
300-1000	1-3	Thick	Wide
1000-3000	3-10	Very Thick	Very Wide
>3000	>10	Massive	Extremely Wide

ROCK HARDNESS/STRENGTH DESCRIPTORS*

Extremely Hard — Core, fragment, or exposure cannot be scratched with knife or sharp pick; can only be chipped with repeated heavy hammer blows.

Very Hard — Cannot be scratched with knife or sharp pick. Core or fragment breaks with repeated heavy hammer blows.

Hard — Can be scratched with knife or sharp pick with difficulty (heavy pressure). Heavy hammer blow required to break specimen.

Moderately Hard — Can be scratched with knife or sharp pick with light or moderate pressure. Core or fragment breaks with moderate hammer blow.

Moderately Soft — Can be grooved $^{1}/_{16}$ inch (2mm) deep by knife or sharp pick with moderate or heavy pressure. Core fragment breaks with light hammer blow or heavy manual pressure.

Soft — Can be grooved or gouged easily by knife or sharp pick with light pressure, can be scratched with fingernail. Breaks wit light to moderate manual pressure.

Very Soft — Can be readily indented, grooved, or gouged with fingernail, or carved with a knife. Breaks with light manual pressure.

*Note: Although "sharp pick" is included in those definitions, descriptions of ability to be scratched, grooved, or gouged by a knife is the preferred criteria.

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"Engineering Geology Field Manual, Second Edition, Volume 1, by U.S. Department of Interior, Bureau of Reclamation, 1998

ROCK TERMINOLOGY

GENERAL NOTES FOR BORING LOGS:

The boring logs are intended for use only in conjunction with the text, and for only the purposes the text outlines for our services. The Plate "Soil Terminology" defines common terms used on the boring logs.

The plate "Unified Soil Classification System," illustrates the method used to classify the soils. The soils were visually classified in the field; the classifications were modified by visual examination of samples in the laboratory, supported, where indicated on the logs, by tests of liquid limit, plasticity index, and/or gradation. In addition to the interpretations for sample classification, there are interpretations of where stratum changes occur between samples, where gradational changes substantively occur, and where minor changes within a stratum are significant enough to log.

There may be variations in subsurface conditions between borings. Soil characteristics change with variations in moisture content, with exchange of ions, with loosening and densifying, and for other reasons. Groundwater levels change with seasons, with pumping, from leaks, and for other reasons. Thus boring logs depict interpretations of subsurface conditions only at the locations indicated, and only on the date(s) noted.

SPECIAL FIELD NOTES FOR THIS REPORT:

1. The five borings were drilled on March 22, 2005, with a track-mounted drilling rig utilizing 4½-inch-diameter (outside) solid stem flight augers. The boring s were sealed with cement and capped with soil immediately after the last soil sample was collected.

2. The boring locations were approximately located by pacing from known points on the site. The elevations were approximated using the site plans and parcel map provided by the client, with the elevation datum as shown on Plate 2, Site Plan.

3. The soils' Group Names [e.g. SANDY LEAN CLAY] and Group Symbols [e.g. (CL)] were determined or estimated per ASTM D 2487-00, Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System, see Plate 5). Other soil and rock engineering terms used on the boring logs are defined on Plate 6, Soil Terminology, and Plate 7, Rock Terminology.

4. The "Blow Count" Column on the boring logs indicates the number of blows required to drive the sampler below the bottom of the boring, and the blow counts given are for each 6 inches of sampler penetration.

5. Groundwater was encountered during drilling, at the depths and locations as shown on the boring logs.

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KEY TO SYMBOLS

Symbol Description

Strata symbols

Lean clay

Sandy lean clay

Clayey gravel

Lean clay with gravel

Sandstone

Clayey sand

High plasticity organic clay

Misc. Symbols

Boring continues

Water first encountered during drilling

Water level at completion of boring

Soil Samplers

Modified California Sampler: 2.375" ID by 3" OD, split-barrel sampler driven w/ 140-pound hammer falling 30" (ASTM D 3550-01)

Symbol Description

Standard Penetration Test: 1 3/8" ID by 2" OD, split-spoon sampler driven with 140-pound hammer falling 30" (ASTM D 1586-99)

Line Types

Denotes a sudden, or well identified strata change

Denotes a gradual, or poorly identified strata change

Laboratory Tests

Symbol	Description
DS	Denotes direct shear test performed at field moisture content (ASTM D2166-00).
DSm	Denotes the second half of a multi-phase direct shear test performed on a DS sample (ASTM D2166-00).
LL	Denotes Liquid Limit per ASTM D4318-00
PI	Denotes Plasticity Index per ASTM D4318-00

BORING LOG

Boring No. B-1
Page 1 of 2

JOB NAME: Proposed Residential Development
CLIENT: LegacyQuest, Inc.
LOCATION: APN 022-083-20 & -30, Fassler Ave., Pacifica
DRILLER: Britton Exploration
DRILL METHOD: Track Mounted CME-75 w/ 4½" flight augers

JOB NO.: PACIQ-01-00
DATE DRILLED: 03/22/05
ELEVATION: 319± feet
LOGGED BY: MT
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		CL	LEAN CLAY WITH SAND, brown, moist, very stiff, medium plasticity	
						3	7, 11, 13		increasing sand content	
DS DSm	1000 2000	Nat. Nat.	930 1700	14.3	118	6	7, 11, 11		SANDY LEAN CLAY, brown, moist to wet, very stiff, medium plasticity increasing gravel content	LL = 43 PI = 22
				19.3	107	9	5, 9, 8	GC	CLAYEY GRAVEL WITH SAND, brown, moist, medium dense dark brown, with coarser gravels	
				6.2	125	12	10, 17, 20			
						15			increasing fines content	
DS	1500	Nat.	560	30.9	91	18	7, 4, 8	CL	GRAVELLY LEAN CLAY, brown, moist, stiff, medium plasticity gravels coarse grained, subangular to angular	

BORING LOG

Boring No. B-1

JOB NAME: Proposed Residential Development *JOB NO.:* PACIQ-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
									increasing gravel content	
						21		ROCK	SANDSTONE, red-brown and brown, moist, intensely weathered, soft, very close bedded, friable at seams	Bedrock
						24	6 10 17			
						27	20 30 25			
						30			Boring terminated at 29.5 feet. Tremie grouted with neat cement.	
						33				
						36				
						39				

BORING LOG

Boring No. B-2
Page 1 of 2

JOB NAME: Proposed Residential Development
CLIENT: LegacyQuest, Inc.
LOCATION: APN 022-083-20 & -30, Fassler Ave., Pacifica
DRILLER: Britton Exploration
DRILL METHOD: Track Mounted CME-75 w/ 4½" flight augers

JOB NO.: PACIQ-01-00
DATE DRILLED: 03/22/05
ELEVATION: 301± feet
LOGGED BY: MT
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		CL	LEAN CLAY, brown, moist, medium stiff, medium plasticity	
								GC	CLAYEY GRAVEL, orange brown, wet, medium dense	
				14.7	114	3	7 / 15 / 23		increasing sand and fines content	
								ROCK	SANDSTONE, orange brown, moist, decomposed, very soft	
DS DSm	1000 2000	Nat. Nat.	1100 2000	13.9	119	6	25 / 50			
									very soft to soft, portions clayey	
				11.7	121	9	25 / 50			
						12				
				12.3	122		20 / 30 / 50		with claystone/siltstone portions	
						15				
						18	50-5"			

BORING LOG

Boring No. B-2
Page 2 of 2

JOB NAME: Proposed Residential Development *JOB NO.:* PACIQ-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						21				
							30 50-4"			
						24			Boring terminated at 23.8 feet. Backfilled with neat cement and capped with soil.	No groundwater encountered.
						27				
						30				
						33				
						36				
						39				

BORING LOG

Boring No. B-3
Page 1 of 2

JOB NAME: Proposed Residential Development
CLIENT: LegacyQuest, Inc.
LOCATION: APN 022-083-20 & -30, Fassler Ave., Pacifica
DRILLER: Britton Exploration
DRILL METHOD: Track Mounted CME-75 w/ 4½" flight augers

JOB NO.: PACIQ-01-00
DATE DRILLED: 03/22/05
ELEVATION: 314± feet
LOGGED BY: MT
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		CL	SANDY LEAN CLAY, orange brown, moist, very stiff to hard, medium plasticity	
DS DSm	1000 2000	Nat. Nat.	1300 2500	15.3	111	3	21 25 25			
								ROCK	SANDSTONE, orange brown, moist, decomposed, soft, very thin bedded	Bedrock
				12.3	116	6	17 50-3"			
DS DSm	1500 2500	Nat. Nat.	2000 3000	8.8	123	9	30 50			
						12				
				14.5	116	15	26 35 50			
									with decomposed, sandy portions	
				16.1	114	18	15 50			

BORING LOG

Boring No. B-3

JOB NAME: Proposed Residential Development *JOB NO.:* PACIQ-01-00

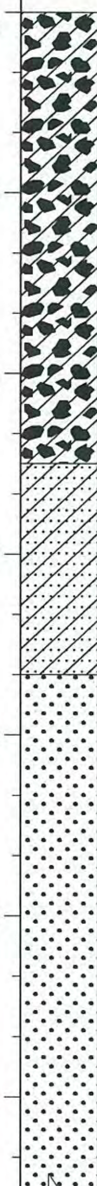
Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						21				
							25			
							50			
						24			Boring terminated at 24.0 feet. Backfilled with neat cement and capped with soil.	No groundwater encountered.
						27				
						30				
						33				
						36				
						39				

BORING LOG

Boring No. B-4
Page 1 of 2

JOB NAME: Proposed Residential Development
CLIENT: LegacyQuest, Inc.
LOCATION: APN 022-083-20 & -30, Fassler Ave., Pacifica
DRILLER: Britton Exploration
DRILL METHOD: Track Mounted CME-75 w/ 4½" flight augers

JOB NO.: PACIQ-01-00
DATE DRILLED: 03/22/05
ELEVATION: 304± feet
LOGGED BY: MT
CHECKED BY:



Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		GC	CLAYEY GRAVEL WITH SAND, orange brown, moist, loose	
						3	5, 7, 6			
									brown	
DS DSm	1000 2000	Nat. Nat.	1050 2250	15.0	117	6	4, 6, 5		increasing sand content	
DS	1500	Nat.	1300	17.0	115	9	3, 4, 3	SC	CLAYEY SAND WITH GRAVEL, brown, moist, loose	
						12		ROCK	SANDSTONE, orange brown, moist, decomposed, very soft to soft, very thin beded	
DS DSm	1500 2500	Nat. Nat.	1500 2500	16.8	115		10, 25, 25		with claystone/siltstone portions	
						15				
						18	15, 25, 35		sandstone friable - consistency like clayey sand	

BORING LOG

Boring No. B-4
Page 2 of 2

JOB NAME: Proposed Residential Development   *JOB NO.:* PACIQ-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						21				
DS DSm	2000 3000	Nat. Nat.	3000 4000	9.0	135	24	10 25 30			
									Boring terminated at 24.5 feet. Backfilled with neat cement and capped with soil.	No groundwater encountered.
						27				
						30				
						33				
						36				
						39				

BORING LOG

Boring No. B-5

JOB NAME: Proposed Residential Development
CLIENT: LegacyQuest, Inc.
LOCATION: APN 022-083-20 & -30, Fassler Ave., Pacifica
DRILLER: Britton Exploration
DRILL METHOD: Track Mounted CME-75 w/ 4½" flight augers

JOB NO.: PACIQ-01-00
DATE DRILLED: 03/22/05
ELEVATION: 292± feet
LOGGED BY: MT
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		SC	CLAYEY SAND WITH GRAVEL, orange brown, moist, loose	
DS DSm	1000 2000	Nat. Nat.	1000 1800	17.2	113	3	8 6 7			
								OH	ORGANIC FAT CLAY, dark gray brown/black, moist, soft to medium stiff, high plasticity	
								CL	LEAN CLAY WITH FINE SAND, brown, moist, stiff, medium plasticity	
DS DSm	1000 2000	Nat. Nat.	1300 2100	20.2	106	6	6 14 15		increasing sand content	
								SC	CLAYEY SAND, trace gravels, orange brown, moist, very dense	
				8.8	125	9	14 30 45			
						12		ROCK	SANDSTONE, orange brown, moist, decomposed to intensely weathered, very thin bedded	Bedrock
				10.1	125		14 50			
						15				
									dry, intensely weathered	
						18	7 45 40			

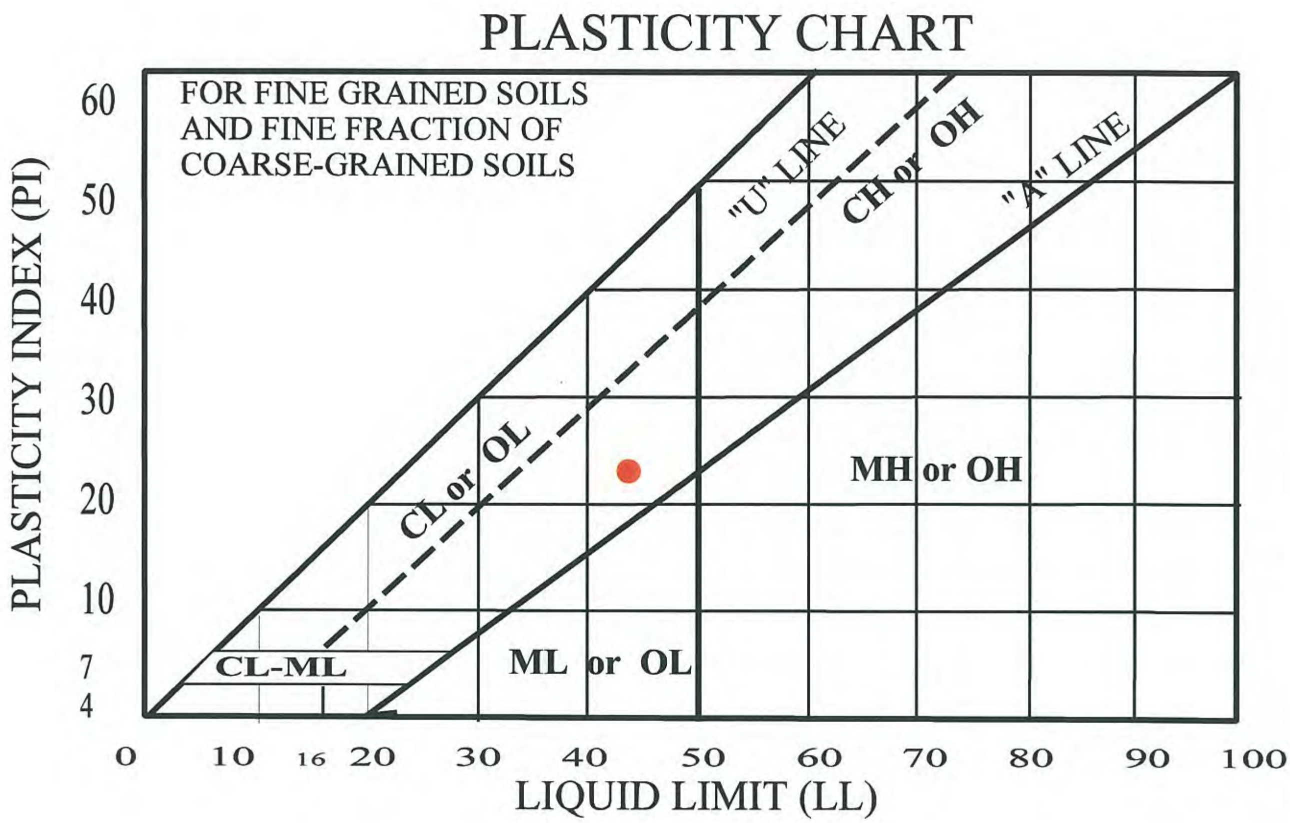
BORING LOG

Boring No. B-5
Page 2 of 2

JOB NAME: Proposed Residential Development

JOB NO.: PACIQ-01-00

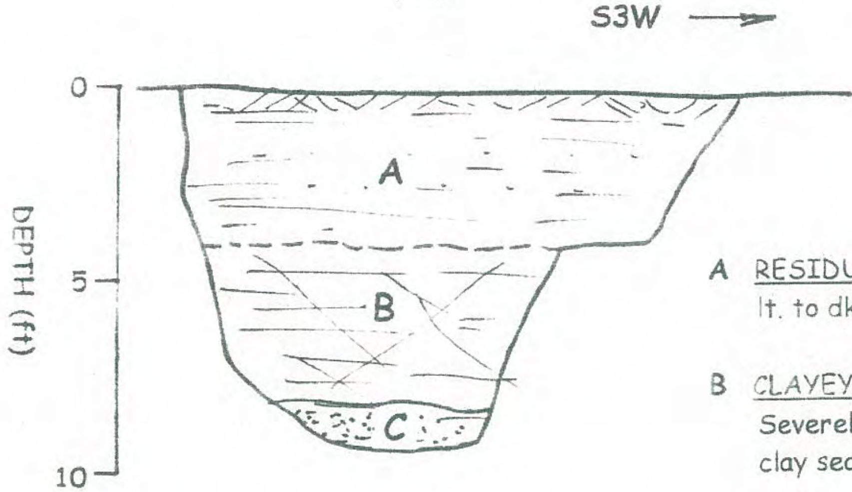
Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						21			friable, consistency like cemented sand	
							30 50			
						24			Boring terminated at 24.0 feet. Backfilled with neat cement and capped with soil.	No groundwater encountered.
						27				
						30				
						33				
						36				
						39				



SYMBOL	SAMPLE SOURCE	DEPTH (FEET)	NATURAL WATER CONTENT W[%]	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	SOIL DESCRIPTION
●	B-1	5½	14.3	43	21	22	Brown SANDY LEAN CLAY (CL)

PROPOSED RESIDENTIAL DEVELOPMENT FASSLER AVENUE PACIFICA, CALIFORNIA	**PLASTICITY DATA**		
	Job No. PACIQ-01-00	Date April 2005	Plate 17

P-1

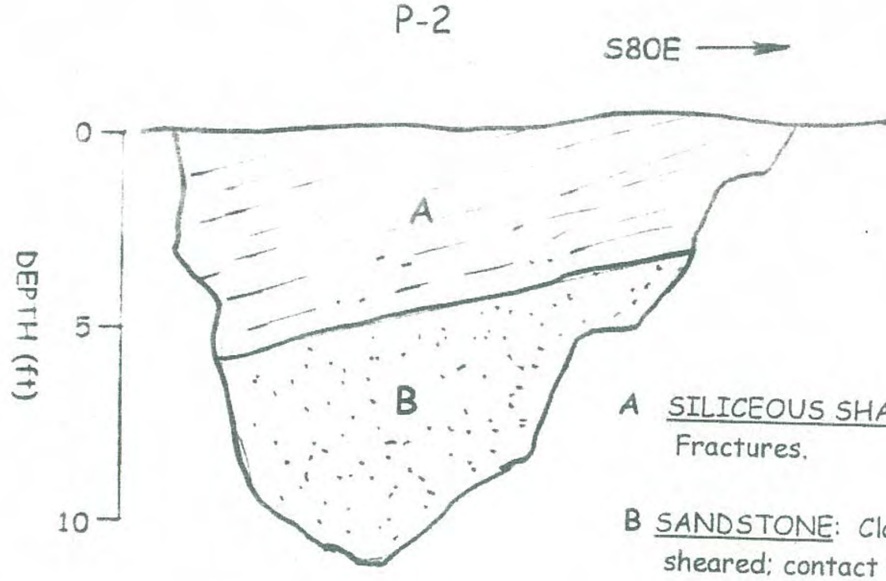


A RESIDUAL SOIL: Gravelly Clay; lt. to dk. yel. orange; moist.

B CLAYEY SILTSTONE: Very Severely weathered; fractured; clay seams; digs easily to 8'

C SANDSTONE w/calcite seams: Refusal at 9'.

P-2



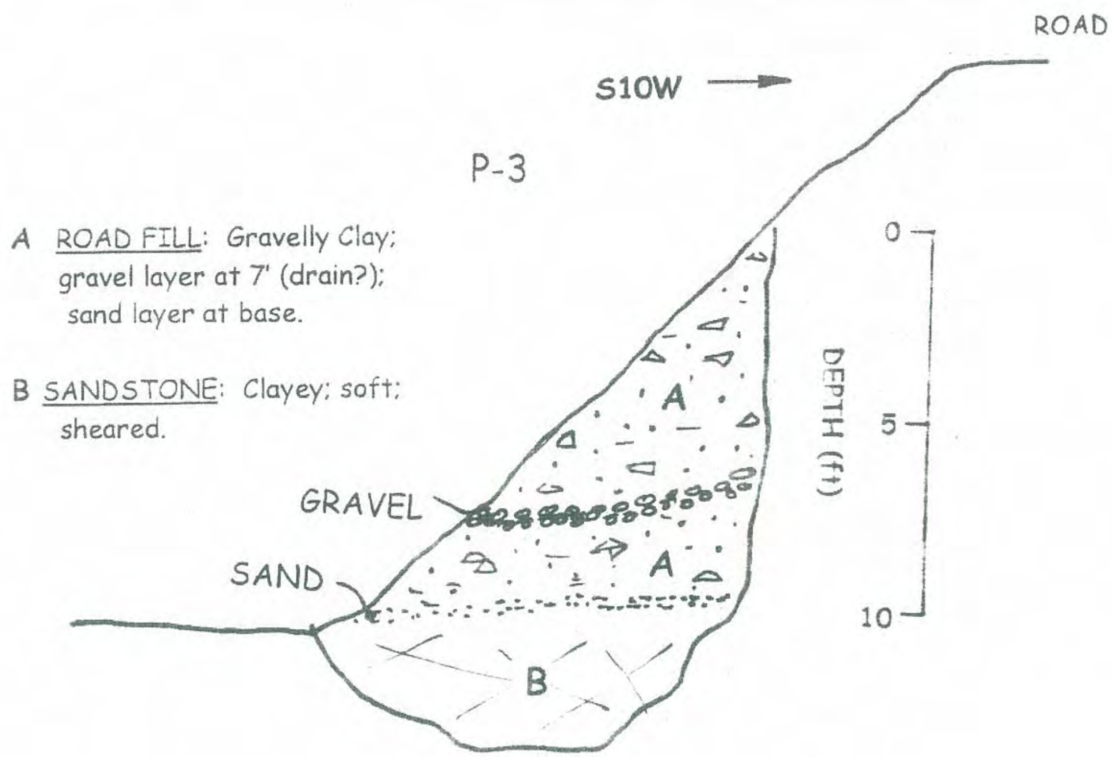
A SILICEOUS SHALE: Calcite in Fractures.

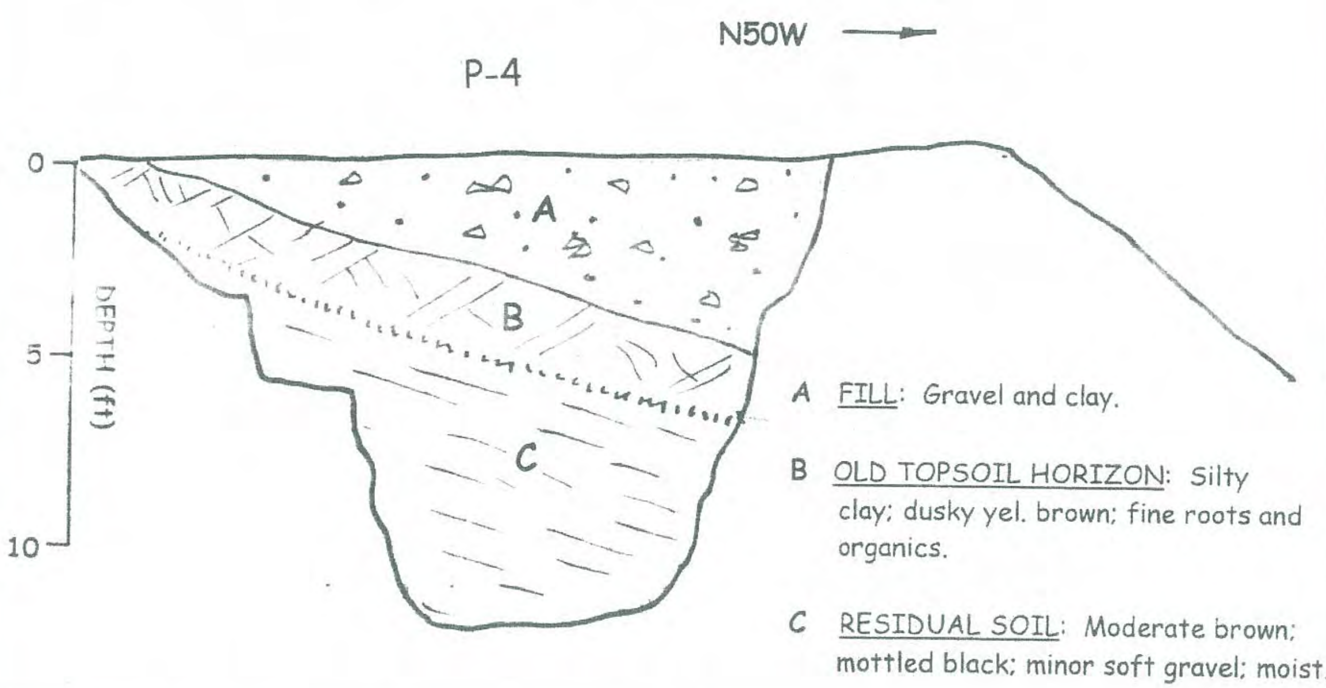
B SANDSTONE: Clayey; soft; sheared; contact w/shale dips 15 deg. NE.

PROPOSED RESIDENTIAL DEVELOPMENT
FASSLER AVENUE
PACIFICA, CALIFORNIA

TEST PIT LOGS

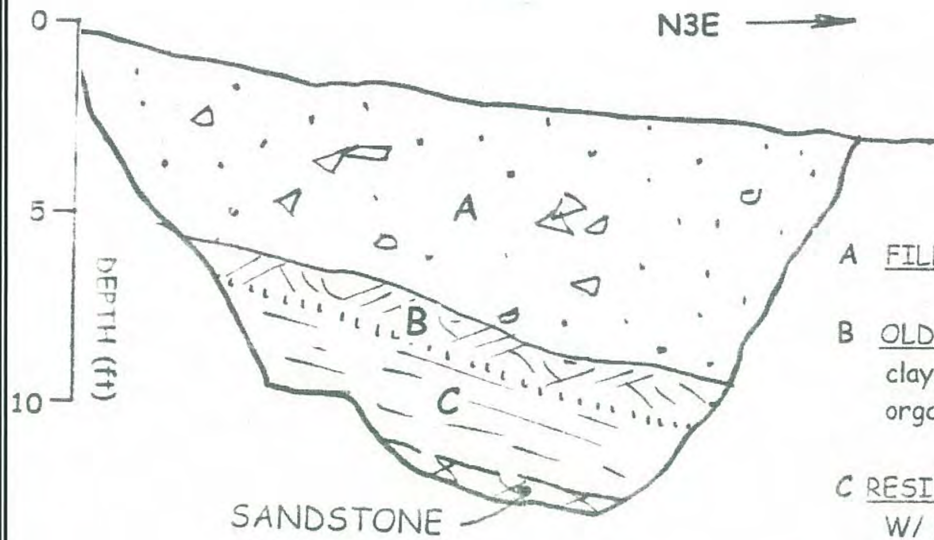
JOB NO.	SCALE	DATE	PLATE
PACIQ-01-00	1" = 5'	April 2005	18





PROPOSED RESIDENTIAL DEVELOPMENT FASSLER AVENUE PACIFICA, CALIFORNIA	TEST PIT LOGS			
	JOB NO. PACIQ-01-00	SCALE 1" = 5'	DATE April 2005	PLATE 19

P-5

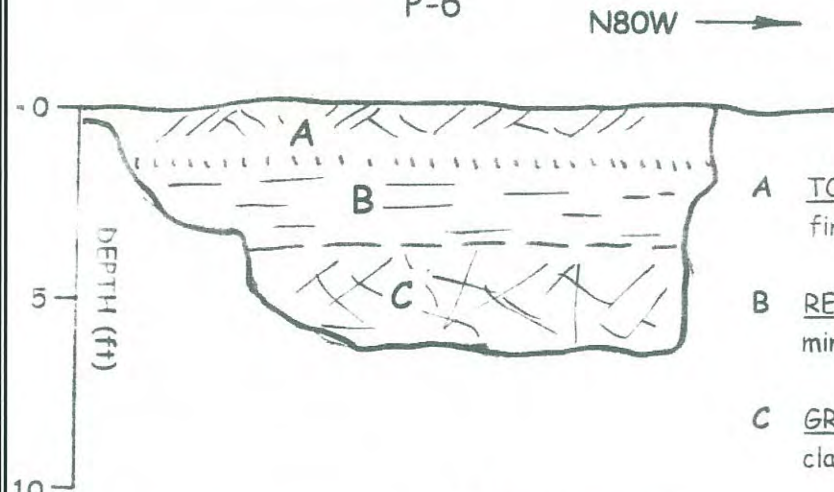


A FILL: Gravel and clay.

B OLD TOPSOIL HORIZON: Silty clay; dusky yel. brown; fine roots and organics.

C RESIDUAL SOIL: Clay; mod. brn. W/ black mottling; soft ss gravel.

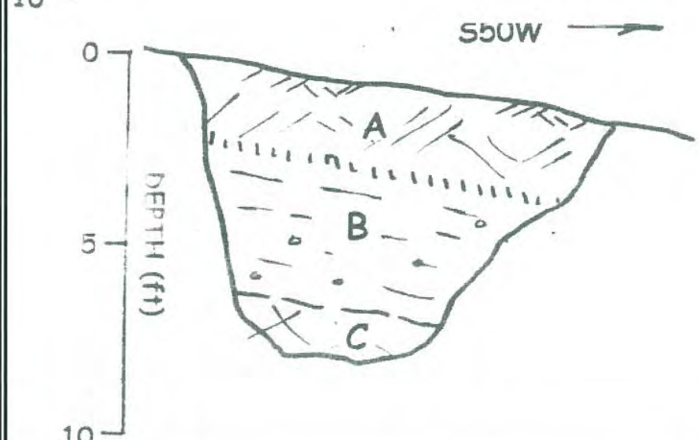
P-6



A TOPSOIL: Silty clay; dusky yel. brown; fine roots and organics.

B RESIDUAL SOIL: Clay; mod. yel. orange; minor soft gravel; moist.

C GREENSTONE: Moderately weathered; clayey; fractured.

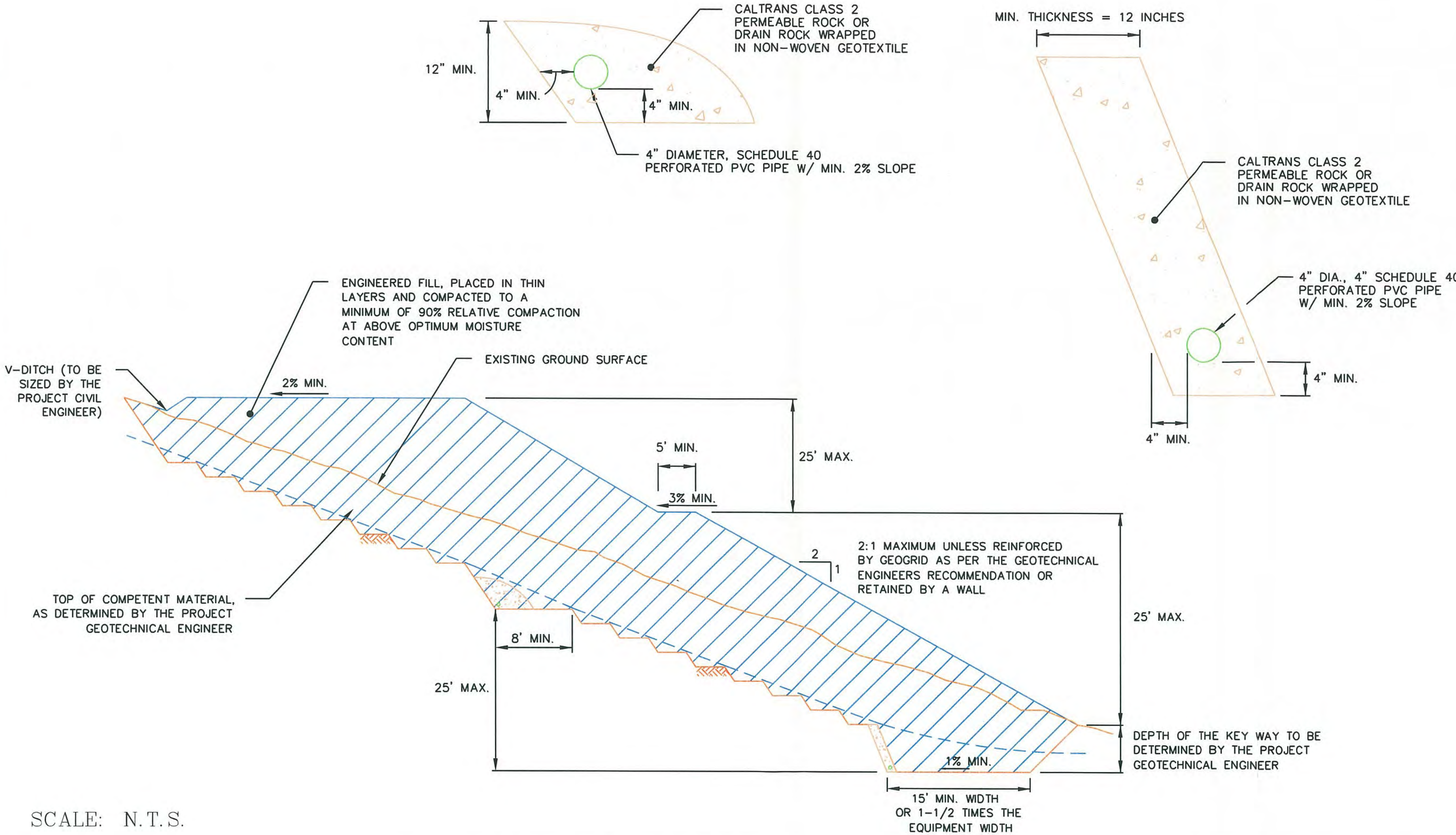


A TOPSOIL: Silty clay; dusky yel. brown; fine roots and organics.

B RESIDUAL SOIL: Gravelly, silty clay; dk. yel. brown; moist.

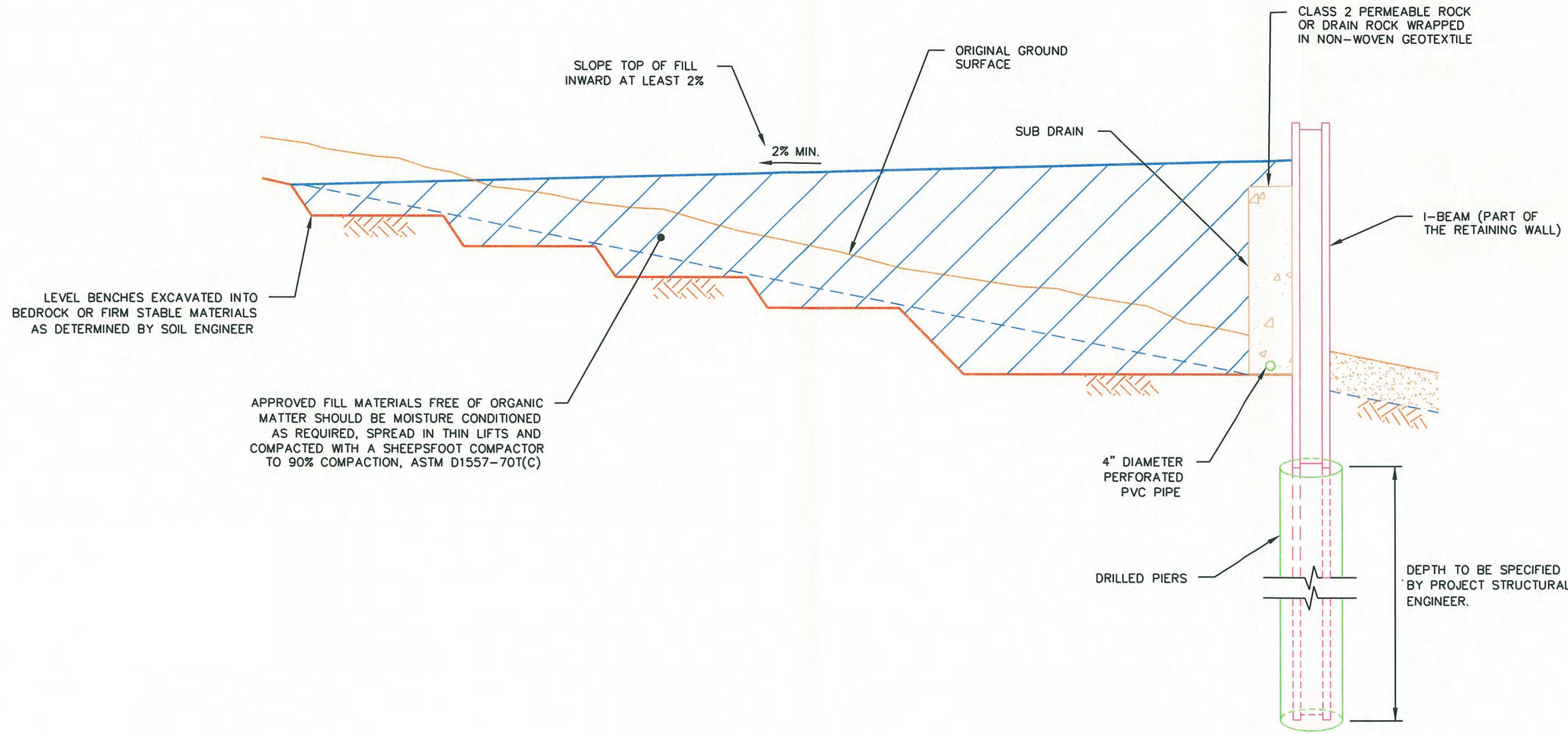
C SILTSTONE: Clayey; fractured; weak.

PROPOSED RESIDENTIAL DEVELOPMENT **FASSLER AVENUE** **PACIFICA, CALIFORNIA**	**TEST PIT LOGS**			
	JOB NO. PACIQ-01-00	SCALE 1" = 5'	DATE April 2005	PLATE 20



SCALE: N.T.S.

	PROPOSED RESIDENTIAL DEVELOPMENT FASSLER AVENUE PACIFICA, CALIFORNIA	BAGG	**SCHEMATIC FOR FILL PLACEMENT ON SLOPING GROUND**
			JOB NO. PACIQ-01-00 / **DATE** 4/21/2005 / **PLATE 21**



SCALE: N.T.S.

PROPOSED RESIDENTIAL DEVELOPMENT FASSLER AVENUE PACIFICA, CALIFORNIA	BAGG	RETAINING WALL BACKFILL

JOB NO.	DATE	
PACIQ-01-00	4/21/2005	PLATE 22

APPENDIX E–2

GEOTECHNICAL INVESTIGATION UPDATE LETTER
BAGG, DECEMBER 2015

December 7, 2015
BAGG Job No: PACIQ-01-00

Mr. Samir Sharma
SDG Architects, Inc.
3361 Walnut Boulevard
Brentwood, California 94513

UPDATE Geotechnical Investigation
Proposed Residential Development
801 Fassler Avenue
Pacifica, California

References:
1) BAGG Engineers, Report, Geotechnical Engineering Investigation, Proposed Residential Development, Fassler Avenue, Pacifica, California, dated April 21, 2005

2) Wood Rogers Developing Innovative Design Solutions, Preliminary Grading and Drainage Plan, 801 Fassler Avenue, Pacifica, California dated March 2015

Dear Mr. Sharma:

Per your request this update to our geotechnical investigation is based on our review of our previous geotechnical investigation report for the project dated April 21, 2005 (Reference 1), and the preliminary grading plans prepared by Wood Rogers, Inc. (Reference 2). In general, the recommendations presented in our previous report are appropriate for the subject project; however, we are presenting supplemental recommendations with respect to the current grading proposed on the sloping terrain located on the northeast side of the development area, where two existing landslides were mapped during our previous investigation of the site. Additionally, we have included supplemental recommendations pertaining to site surface drainage, drilled pier foundations, and updated seismic design parameters per the 2013

California Building Code. A general discussion of the preliminary grading and development plans is presented below followed by our supplemental recommendations.

PRELIMINARY GRADING/DEVELOPMENT PLAN

The current plan shows 22 residential units on a gently sloping finish grade with an average gradient of about 7% toward the west. Grading is shown to consist of cuts and fills of about 20 and 15 feet on the northwest and southeast portions of the development area, respectively. A 25 to 40-foot high fill slope on sloping terrain is shown on the north side of the development area, and a 50-foot high cutslope with gradients ranging from 2H:1V to 3H:1V is planned at the southeast corner of the site. A review of the referenced plan indicates a gently sloping graded area is planned on the east side of the development which will receive a significant amount of runoff from an old roadway extending some 800 feet upslope to the east of the site.

NORTH SIDE FILL SLOPE

The preliminary grading plan (Reference 2) shows a 2H:1V fill slope with a height of 25 to 40 feet on the northeast side of the development area. A review of our 2005 geotechnical investigation report indicates two previously mapped landslides situated at the toe of the proposed fill slope which could potentially be impacted by the runoff from the newly planned fill slope. We recommend a 10 foot wide drainage bench and concrete lined v-ditch be included at the lower third of the slope height to facilitate surface water collection and drainage. Surface water collected from the new fill slope should be routed away from the mapped landslides. We note, however, that adding the drainage bench would likely require moving the toe of the fill slope further northeast, onto steeper terrain, and to maintain a 2H:1V fill slope gradient, the toe would need to extend down to the property line where the natural gradient becomes less steep. A keyway and subdrain would need to be included at the toe of the slope per Plate 21 in our 2005 geotechnical report (Reference 1).

Extending the fill slope down to the property line would include extensive grading and removal of significant vegetation which may not be economically feasible; therefore, in lieu of extending the fill slope further downhill at a 2H:1V gradient, geogrid reinforcement could be incorporated in the construction of a steeper fill slope which would allow the toe to shift upslope and away from the existing landslides. We note that geogrid would extend back within the footprints of the residential units at the top of the fill slope and will conflict some with drilled piers for the foundations and to a lesser extent with utility trenches associated with the residential units; however, we do not anticipated such conflict would have an impact on a geogrid reinforced fill slope provided it is constructed properly. A drainage bench and concrete lined v-ditch would still need to be incorporated in the geogrid reinforced fill slope in order to facilitate collection and removal of runoff from the fill slope and away from the mapped landslides.

Alternately, a concrete lined J-ditch, without a drainage bench, could be incorporated in the construction of the fill slope. This approach would keep the toe of the fill slope as it is shown on the preliminary grading plan (Reference 2), however, the lack of a bench would exclude vehicular access should any significant slope maintenance be necessary in the future. We note that these recommendations for the northwest fill slope are conditional based on the northwest side sloping terrain and landslides being reviewed by a certified engineering geologist, as it has been some 10 years since a geologic reconnaissance was performed on the area.

SOUTHEAST SWALE

The preliminary grading plan (Reference 2) will include a gently slope swale-like area on the southeast side of the development area. The plan indicates the swale-like area will receive a significant amount of runoff from an old abandoned roadway that extends upon a ridgeline some 800 feet upslope and to the southeast from the development area. The swale-like area already contains significant erosion rills, therefore, we believe there is a need to collect, store and/or drain runoff away from this area to help reduce erosion potential.

SOUTH SIDE CUTSLOPE

The preliminary grading plans (Reference 2) indicate a 50-foot high cutslope with gradients ranging from 2H:1V to 3H:1V on the south side of the development area. Based on the review of our previous report (Reference 1), weathered sandstone was mapped in this area. This material may be susceptible to erosion; therefore, we recommend a drainage bench and concrete-lined V-ditch be included about midway up the cutslope. The concrete lined V-ditch for the cutslope could be continuous with the V-ditch recommended in the swale-like area. The drainage benches and V-ditches should convey runoff to an engineered discharge area designed to minimize erosion. BAGG should be contacted to have a certified engineering geologist observe the exposed bedrock cutslope to confirm that the sandstone exposed along the cutslope is competent and that adverse bedding is not present.

DRILLED PIERS

Our 2005 geotechnical report (Reference 1) included recommendations for drilled pier foundations which consisted of minimum 18 inch diameter piers advanced not less than 8 feet into competent bedrock material. Drilled piers are an appropriate foundation type for the subject project, however, we note that drilled piers in areas containing engineered fill will need to be advanced to deeper depths in order to achieve embedment into bedrock. BAGG Engineers should provide full time observation during the drilling of piers in the fill areas to confirm that adequate depth into bedrock is achieved.

2013 CALIFORNIA BUILDING CODE

The Structural Engineering Design Provisions in Chapter 16 of the 2013 California Building Code (CBC) have recently been revised to reflect the changing knowledge regarding earthquake shaking from major earthquakes. The new code uses mapped spectral acceleration values for periods of 0.2 and 1.0 seconds, to better represent the probabilistic shaking that can be expected at a given site. The "mapped" values generally represent "bedrock" shaking with a 2 percent probability of being exceeded in a 50-year period. The values are then modified for site-specific use based on classification of the soil profile at the site.

Based on the soil information obtained from our borings, and using the site coordinates of 37.6031^0 North Longitude and 122.4917^0 West Latitude, the earthquake ground motion parameters in accordance with 2013 California Building Code are tabulated below:

2013 CBC Seismic Design Parameters

2013 CBC Design Parameter	Value
Site Class, Table 1613.5.2	Soft Rock/Dense Soil Profile, C
North Latitude	37.6031°
West Longitude	122.4917°
Ss, Mapped Spectral Response Acceleration for short period	1.98g
S1, Mapped Spectral Response Acceleration for 1 second period	0.89g
$S_{MS,}$ Maxiumum Considered Earthquake Spectral Response Acceleration, short period	1.98g
S_{M1}, Maximum Considered Earthquake Spectral Response Acdeleration, 1-second period	1.16g
$S_{DS,}$ Design Spectral Response Acceleration, short period	1.32g
S_{D1}, Design Spectral response Acceleration, 1-second period	0.77g

We trust this update to our geotechnical engineering investigation provides you with the information required at this time. If you have any questions or require additional information, please feel free to contact us.

Very truly yours,
BAGG Engineers

Michael G. Matusich
Project Engineer

REGISTERED PROFESSIONAL ENGINEER MICHAEL MATUSICH GE 3013 EXP. 12/31/15 GEOTECHNICAL STATE OF CALIFORNIA

Ajay Singh
Senior Engineer

BAGG ENGINEERS