Geotechnical Investigation
Stanford Shopping Center Additions
Palo Alto, California

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Stanford Management Company
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GEOTECHNICAL INVESTIGATION
FOR
STANFORD SHOPPING CENTER ADDITIONS
PALO ALTO, CALIFORNIA

INTRODUCTION

In this report we present the results of our geotechnical investigation for the proposed additions to the existing Stanford Shopping Center located in Palo Alto, California, as shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the subsurface conditions at the site and to provide geotechnical recommendations for design of the proposed new buildings, parking structure and parking area improvements.

The project as presently planned consists of construction of three new retail buildings and a parking garage, as shown on the Site Plan, Figure 2. The new retail buildings will consist of two one-story 9,000 and 10,000 square foot structures and one two-story structure with a plan area of 30,500 square feet. The parking garage will consist of at grade parking with one level of raised overhead parking for a total of 1,529 cars. The new structures will be located adjacent to the existing shopping center building complex and within the existing parking areas. In addition improvements will be made to the existing parking areas.

Project Description
Our scope of services was presented in detail in our agreements with you dated May 8, August 24, and September 25, 1998. To accomplish this work, we have provided the following services:

1. Exploration of the subsurface soil conditions by drilling eight borings and retrieving relatively undisturbed soil samples for visual observation and laboratory testing and coring the existing parking area at 31 locations.

2. Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on selected samples.

3. Engineering analysis to evaluate site earthwork, building foundations, slabs-on-grade, retaining walls and pavements.

4. Preparation of this report as a summary of our findings and to present our conclusions and recommendations.

This report was prepared for the sole use of Stanford Management Company for application to the design of the proposed new buildings, parking structure, and parking area improvements in accordance with generally accepted geotechnical engineering practices at this time and location. No warranty is expressed or implied. Environmental services were excluded from our scope of work.

SITE CONDITIONS

Subsurface exploration was performed on May 12 and May 13, 1998, using conventional, truck-mounted drilling equipment to investigate, sample, and log the
subsurface soils. Eight exploratory borings were drilled to depths ranging from 20 to 35 feet. A representative bulk sample of the surface soil from the parking garage area was obtained for pavement design purposes.

Supplemental exploration was performed on October 9, 1998, where we cored through the asphalt and aggregate base sections at 31 locations throughout the parking lot.

The approximate locations of the borings and corings are shown on the Site Plan, Figure 2. Logs of our borings and details regarding our field investigation are included in Appendix A; the results of our laboratory tests are discussed in Appendix B.

We also performed a brief surface reconnaissance during our site exploration. The proposed building sites are located at the perimeter of the existing Stanford Shopping Center, located southwest of El Camino Real. The shopping center is bordered by Stanford University Hospital to the south, Hoover Pavilion to the east, and the Ronald McDonald Children's Hospital complex and San Francisquito Creek to the west. The areas of new construction are now occupied by asphaltic concrete parking lots.

The site is relatively flat and slopes down to the northeast with a change in elevation from 74 to 63 feet (datum unknown). The closest expression of the San Francisquito Creek exists approximately 300 feet northwest of the existing Macy's Men's Store.
Topographic information was based on the “Site Utilities Plan” drawn by Brian Kangas Foulk, dated February 27, 1998.

Generally the site is blanketed by 4 to 8½ feet of very stiff to hard silty clay. A layer of very stiff silty clay fill was encountered to a depth of approximately 4-feet-thick in Boring EB-5. The lateral extent of the fill is unknown. A Plasticity Index test (PI) was performed on a sample of the surficial clay from Boring EB-2 at a depth of approximately 2 feet. The test resulted in a PI of 16 which indicates low to moderate expansion potential. Interbedded very stiff to hard silty clays, sandy clays, clayey silts and medium dense to dense silty and clayey sands underlie the surficial clay to a depth of 35 feet, the maximum depth explored. Boring EB-3 encountered medium dense to very dense sand with a trace to some clay to a depth of 21½ feet, the maximum depth of this boring. The thicknesses of the asphaltic concrete and aggregate base sections are tabulated on Figure 2.

Free ground water was not encountered in the borings at the time of drilling to a depth of 35 feet (which corresponds to approximately Elevation 33½ feet), the maximum depth explored. The San Francisquito Creek elevation in the area of the Stanford Shopping Center is at approximately 50 feet. Ground water data from monitoring wells at a nearby site was provided by Stanford Management for our review. Water table measurements indicate ground water depths in the area of approximately 40 to 43 feet below grade.
Fluctuations in the level of the ground water may occur due to variations in rainfall, and other factors not in evidence at the time measurements were made.

GEOLOGIC HAZARDS

A brief qualitative evaluation of certain geologic hazards was made during this investigation. Our comments concerning these hazards follow.

A regional fault map illustrating known active faults relative to the site is presented in Figure 3. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone), and as shown on this map, no known surface expression of active faults are believed to exist within the site. Fault rupture through the site, therefore, is not anticipated.

Strong ground shaking can be expected at the site during moderate to severe earthquakes in the general region. This is common to virtually all developments in the San Francisco Bay Area. The “Seismicity” section that follows presents a detailed discussion regarding potential levels of ground shaking.

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained sands. Sands were not encountered in the borings for this investigation, or in our previous borings at the adjacent
site. The sands encountered in our borings were generally medium dense to very dense and contained a significant amount of fine-grained material. In addition, ground water was not encountered to the terminal depth of our borings. For these reasons and based upon engineering judgment, the potential for liquefaction is low during seismic shaking.

The near-surface soils at the site vary in composition both vertically and laterally. Major earthquake shaking could cause non-uniform compaction of the soil strata, resulting in movement of the near-surface soils. In our opinion, because the subsurface soils encountered are generally stiff to very stiff and do not appear to change in thickness or consistency abruptly over short distances, and provided that the fill encountered in the parking structure area is removed and replaced with engineered fill, the potential for distress to the buildings due to differential seismic compaction is low.

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or "free" face such as an open body of water, channel, or excavation. Generally in soils, this movement is due to failure along a weak plane, and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free. Generally, failure in this mode is analytically unpredictable, since it is difficult to
determine where the first tension crack will occur. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free.

The closest expression of the San Francisquito Creek exists at least 200 to 300 feet from the proposed structures. In our opinion, based on the low potential for liquefaction and the distance to San Francisquito Creek, the probability of lateral spreading occurring at the proposed building sites would be low.

SEISMICITY

The San Andreas fault, which generated the great San Francisco Earthquake of 1906, passes about 5 miles southwest of the site. Two other major active faults in the area are the Hayward fault, located about 13 miles northeast of the site, and the Monte Vista - Shannon fault, located about 4 miles southwest.

We performed a Deterministic Seismic Hazard Analysis (DSHA) for this site. This analysis utilizes the maximum Moment Magnitude (Mw) for the controlling faults, published attenuation curves, the shortest distance to the fault, and the site-specific response characteristics. Based on the attenuation methods of Campbell (1994), the mean peak ground acceleration (PGA) expected for this site is 0.54 from a magnitude 6.8 Mw event on the Monte Vista - Shannon fault, located approximately 4 miles to the south and 0.42 from a magnitude 7.1 Mw
event on the San Andreas fault, located approximately 5 miles to the southwest.

We performed a computer search of known historical earthquakes of Richter Magnitude 5 or greater within a 100 kilometer radius of the site using an abbreviated version of the California Division of Mines and Geology computerized earthquake catalog of events through December 1997. We also included data from Townley and Allen (1939) and the U.S. Geological Survey Earthquake Data Base System, giving 198 years of data in the search area. The results of our computer search indicate that 88 known earthquakes of Richter Magnitude 5 or greater have occurred within 100 kilometers of the site between 1800 and December 1997.

Four earthquakes of Richter Magnitude 7 or greater have occurred in the region, during the above noted time period, including the Loma Prieta Earthquake of 1989, centered about 32 miles from the site. Based on attenuation methods of Campbell (1994), the maximum historic site acceleration that may have been experienced at the site is computed at approximately 0.26g, resulting from a Richter Magnitude 8.25 earthquake in 1906 located about 25 miles from the site.

The computer-generated acceleration values and probabilities should only be considered reasonable best estimates. All of the influences affecting attenuation and occurrence rates are not yet known; furthermore, there are uncertainties in every parameter used to
obtain such results. At the present time there is no test available to verify the validity of the acceleration and probability data. Therefore, significant deviations from the indicated values are possible due to geotechnical and geological uncertainties and other specific site conditions.

The Working Group on California Earthquake Probabilities (1990) has concluded that the probability of a magnitude 7.0+ earthquake in the San Francisco Bay Area over the next 30 years is 67 percent. This probability is a low estimate since only three active faults in the area, the Hayward, San Andreas, and Rodgers Creek were included in the study. Schwartz (1994) concludes that the probability of the occurrence of one or more magnitude 7.0+ earthquakes in the Bay Area is substantially higher than 67 percent, possibly as high as 90 percent. Please note that significant earthquakes could occur on an active or a potentially active fault for which probabilities have not been estimated.

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. Our current understanding of earthquake activity indicates, however, that the site will likely be subject to at least one moderate to severe earthquake within 50 years following construction. During such an earthquake the danger of fault offset at the site is slight, but strong shaking of the site is likely to occur.
Based on our borings and alluvium thickness maps of Santa Clara County (Rogers and Williams 1974), the site is underlain by stiff soils extending to depths on the order of 100 feet below the ground surface. Based on this information, the site may be characterized as Type S_1 (S Factor of 1.0) in accordance with Table No. 16-J of the 1994 Uniform Building Code (UBC).

Based on the above information and local seismic sources, the site may also be characterized for design based on Chapter 16 of the 1997 UBC using the information in Table 1.

**TABLE 1. 1997 UBC Site Categorization and Site Coefficients**

<table>
<thead>
<tr>
<th>Categorization/Coefficient</th>
<th>Design Value</th>
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<tr>
<td>Soil Profile Type (Table 16-J)</td>
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<td>Seismic Zone Factor (Table 16-I)</td>
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</tr>
<tr>
<td>Near Source Factor N_v (Table 16-T)</td>
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<tr>
<td>Seismic Coefficient C_a (Table 16-Q)</td>
<td>0.48</td>
</tr>
<tr>
<td>Seismic Coefficient C_v (Table 16-R)</td>
<td>0.87</td>
</tr>
</tbody>
</table>

**CONCLUSIONS AND RECOMMENDATIONS**

From a geotechnical engineering viewpoint the proposed buildings, parking structure, and parking area improvements may be constructed as planned, in our opinion, provided the design is performed in accordance with the recommendations presented in this report.

**Conclusions**
The primary geotechnical concern is the presence of fill in the area of the proposed parking structure. The existing fill should be delineated by pot-holing during construction and removed within the parking structure footprint to a distance of 5 feet beyond the perimeter of the parking structure. Provided the existing fill material meets the requirements in the “Material for Fill” section below, it may be reused as engineered fill. Prior to replacing the fill the exposed subgrade should be prepared as presented in the “Subgrade Preparation” section. The fill should be replaced and compacted in accordance with the recommendations for fill presented in the “Compaction” section.

Because subsurface conditions may vary considerably from those predicted by relatively small diameter borings, and in order to check that our report recommendations have been properly implemented, we recommend that we be retained to 1) review the final construction plans and specifications and 2) observe the earthwork and foundation installation. Also, the assumed and/or actual geotechnical conditions can be greatly affected by the construction process. For the above reasons our geotechnical recommendations are contingent upon our firm providing geotechnical observation and testing services during construction.

EARTHWORK

Areas of new construction should be cleared of all surface and subsurface deleterious materials including any existing building foundations, slabs or pavements.
to be removed, buried utility and irrigation lines, fills, debris, designated trees, shrubs, and associated roots. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted to the recommendations presented in the "Compaction" section. We recommend that the backfilling be carried out under our observation. The existing pavements in the area of the parking garage could be left in place, if desired to use them for lower level parking, except where the removal and recompaction of fills are necessary.

After clearing, any vegetated areas should be stripped to sufficient depth to remove all surface vegetation and topsoil containing greater than 3 percent organic matter by volume. At the time of our field investigation, most of the proposed building areas are currently paved with asphaltic concrete paving. Landscape materials should be removed from medians and planting areas. The actual stripping depth required depends on site usage prior to construction and should be established in the field by us at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscaped areas, if desired.

If desired to reuse asphaltic concrete or a mixture of asphaltic concrete and aggregate base as engineered fill, we recommend that it be ground up and thoroughly mixed with on-site or import soil. In general, recycled asphalt or concrete should be ground down to less than 4 inches in greatest dimension, with no more than 25 percent larger than 2.5 inches. Recycled material

**Reuse of On-Site Recycled Materials**
should be thoroughly mixed with a sufficient amount of soil, such that there is no more than 40 percent by weight of recycled material in the final mix. Recycled fill containing recycled asphalt and concrete should be used within 2 feet of finished grade in building areas.

All fills should be removed down to native soil. Provided the fill material meets the requirements in the “Material for Fill” section, it may be reused as engineered fill. Side slopes of fill excavations in building and pavement areas should be sloped at inclinations no greater than 3:1 (horizontal to vertical) to minimize abrupt variations in fill thickness. All fill should be compacted in accordance with the recommendations for fill presented in the “Compaction” section.

After the site has been properly cleared, stripped, and necessary excavations have been made, the exposed surface soils in those areas to receive fill, slabs-on-grade, or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the “Compaction” section. The finished compacted subgrade should be firm and non-yielding under the weight of compaction equipment.

All on-site soils below the stripped layer having an organic content of less than 3 percent by volume are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with no more than 15 percent.
larger than 2.5 inches. Imported fill material should be predominantly granular with a Plasticity Index of 15 or less.

All fill, as well as scarified surface soils in those areas to receive fill or slabs-on-grade, should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness at a moisture content near the laboratory optimum. Each successive lift should be firm and non-yielding under the weight of the construction equipment.

The upper 6 inches of subgrade in pavement areas and all aggregate base should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition). Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum.

All utility trenches should be backfilled with compacted fill in accordance with local requirements or the recommendations contained in this section. Fill material should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) by mechanical means only. If imported sand is used as backfill, the upper 3 feet in building and pavement areas should be compacted to 95 percent. The upper 6 inches of backfill in all
pavement areas should be compacted to at least 95 percent relative compaction.

The contractor should be responsible for all temporary slopes and trenches excavated at the site and the design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards.

Positive surface gradients should be provided adjacent to the structures to direct surface water away from the foundations and slabs towards suitable discharge facilities. Ponding of surface water should not be allowed on or adjacent to structures, slabs-on-grade, or pavements. Roof runoff should be directed away from foundations and slabs-on-grade.

All grading and earthwork should be performed under the observation of our representative to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is essential. The project plans and specifications should incorporate all recommendations contained in the text of this report.

Variations in soil conditions are possible and may be encountered during construction. In order to permit correlation between the soil data obtained during our field and laboratory investigations and the actual
subsurface conditions encountered during construction and to observe conformance with the plans and specifications, it is essential that we be retained to perform continuous or intermittent review during the earthwork, excavation and foundation construction phases. Therefore, the conclusions and recommendations presented in this report are contingent upon us performing construction observation services.

FOUNDATIONS

We recommend that the proposed buildings and parking structure be supported on conventional continuous and/or isolated spread footings bearing on natural, undisturbed soil or compacted fill.

All footings should have the minimum footing dimensions shown in Table 2. Footing depths are taken from lowest adjacent finished grade, considered as the bottom of interior slab-on-grade or the finish exterior grade, excluding landscape topsoil, whichever is lower.

<table>
<thead>
<tr>
<th>Number of Stories</th>
<th>Minimum Footing Width (inches)</th>
<th>Minimum Footing Depth (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12</td>
<td>18</td>
</tr>
<tr>
<td>2</td>
<td>15</td>
<td>18</td>
</tr>
<tr>
<td>3</td>
<td>18</td>
<td>24</td>
</tr>
</tbody>
</table>

Footings constructed in accordance with the above recommendations would be capable of supporting
maximum allowable bearing pressures of 3,000 pounds per square foot (psf) for dead loads, 4,500 psf for combined dead and live loads, and 6,000 psf for all loads including wind or seismic. These allowable bearing pressures are based upon factors of safety of 3.0, 2.0, and 1.5 for dead, dead plus live, and seismic loads, respectively.

These maximum allowable bearing pressures are net values; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to permit spanning of local irregularities. It is essential that we observe the footing excavations before the reinforcing steel is placed.

**ONE- AND TWO-STORY BUILDINGS:** We estimate that post-construction differential settlement under static loads for the one- and two-story buildings will be less than ½-inch based on maximum interior column loads of 200 kips.

**PARKING STRUCTURE:** Preliminary structural loads for the parking structure were provided for our review by Charles Pankow Builders. Typical interior column dead plus live loads on the order of 270 kips and perimeter
column dead plus live load on the order of 140 kips were used for analysis. Based on these preliminary loads and the maximum allowable bearing pressures recommended above, we estimate that post-construction differential settlement should be less than approximately ¼-inch.

If the parking structure cannot tolerate the above estimated differential settlements between heavily loaded short-span interior columns, the interior column lines may alternately be supported by continuous strip footings. Continuous strip footings would be capable of supporting average allowable bearing pressures of 3,000 psf and maximum allowable bearing pressures of 4,500 psf for combined dead and live loads. These allowable bearing pressures are based on factors of safety of 3.0 and 2.0 for dead and dead plus live loads, respectively.

We estimate that post-construction differential settlement under static loads for continuous strip footings will be less than ½-inch between adjacent columns. Perimeter column loads are relatively light and could remain on isolated footings. We estimate that post-construction differential settlements between adjacent perimeter columns should be less than ½-inch. For the purpose of providing sufficient future clearance in the garage, total foundation settlement of 1 inch should be assumed.

The proposed slab-on-grade floors should be supported by at least 6 inches of non-expansive fill, prepared in
accordance with the recommendations presented in the "Subgrade Preparation" section of this report. Before slab construction, the subgrade surface should be proof-rolled to provide a smooth, firm surface for slab support. Slab reinforcing should be provided in accordance with the anticipated use and loading of the slab.

If desired to limit moisture rise through slabs-on-grade in habitable areas, we recommend that a moisture barrier be constructed beneath the slab. This should consist of 2 inches of clean, fine sand over a minimum 10-mil-thick vapor barrier. At least 4 inches of free draining gravel such as ½-inch or ¾-inch crushed rock with no more than 5 percent passing the ASTM No. 200 sieve should be placed beneath the vapor barrier to serve as a capillary break. Pea gravel should not be used as a capillary break as it provides weak lateral resistance during seismic ground shaking. The sand should be lightly moistened just prior to placing the concrete. The moisture barrier could be used as the 6 inches of non-expansive fill recommended above.

Adequate footing reinforcement should be provided to satisfy with the anticipated use and loading requirements. We recommend that a modulus of subgrade reaction of 60 pounds per cubic inch be used in design of continuous strip footings underlain by undisturbed native soils.

Lateral loads may be resisted by friction between the footings and the supporting subgrade. A maximum
allowable frictional resistance of 0.25 may be used for design. In addition, lateral resistance may be provided by passive pressures acting against foundations poured neat against competent soil. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot be used in design.

RETAINING WALLS

Any proposed retaining walls should be designed to resist lateral earth pressures from adjoining natural materials and/or backfill and from any surcharge loads. Provided that adequate drainage is provided as recommended below, we recommend that walls restrained from movement at the top be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot (pcf) plus a uniform pressure of $8H$ pounds per square foot, where $H$ is the distance in feet between the top of the footing and the top of the wall. Such walls should also be designed to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface. Any unrestrained retaining walls with adequate drainage should be designed to resist an equivalent fluid pressure of 45 pcf plus one-third of any surcharge loads.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the ground.
water level. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp-proofing of the walls should be included in areas where wall moisture would be problematic.

Adequate drainage may be provided by a subdrain system positioned behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material of Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 2 feet out from the wall and to within 2 feet of the outside finish grade. Alternatively, 1/2-inch to 3/4-inch drain rock may be used in place of the Class 2 Permeable Material provided the drain rock and pipe are enclosed in filter fabric, such as Mirafi 140N or equivalent. The upper 2 feet of wall backfill should consist of relatively impervious compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage matting may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. The drainage panel should be connected to the perforated drain pipe at the base of the wall.

Backfill placed behind the walls should be compacted to at least 90 percent relative compaction using light
compaction equipment. If heavy compaction equipment is used, the walls should be temporarily braced.

Retaining walls may be supported on spread footing foundations designed in accordance with the recommendations presented under the “Footings” section of this report. Lateral load resistance for the walls may be developed in accordance with the recommendations presented in the “Lateral Loads” section.

PAVEMENTS

We obtained a representative bulk sample of the surface soil from the parking area and performed an R-value test to provide data for pavement design. The results of the test are included in Appendix B and indicate an R-value of 7. Because surface soils vary across the site, we judged an R-value of 5 to be applicable for design. Using estimated traffic indices for various pavement loading requirements, we developed the following recommended pavement sections using Procedure 608 of the Caltrans Highway Design Manual, presented in Table 3.
TABLE 3. **Recommended Asphalstic Concrete Pavement Design Alternatives**  
**Pavement Components**  
**Design R–Value = 5**

<table>
<thead>
<tr>
<th>General Traffic Condition</th>
<th>Design Traffic Index</th>
<th>Asphalstic Concrete (Inches)</th>
<th>Aggregate Baserock* (Inches)</th>
<th>Total Thickness (Inches)</th>
<th>Full Depth Asphalstic Concrete Section (Inches)</th>
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<td>14.0</td>
<td>18.0</td>
<td>10.5</td>
</tr>
<tr>
<td></td>
<td>7.0</td>
<td>4.0</td>
<td>16.0</td>
<td>20.0</td>
<td>11.0</td>
</tr>
</tbody>
</table>

Caltrans Class 2 aggregate base; minimum R-value equal to 78.

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. The traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel load analysis or a traffic study.

Recommendations for Portland Cement Concrete (PCC) pavements are presented in Table 4. Since the expected Average Daily Truck Traffic (ADTT) is not known at this time, we have provided alternatives for minimum pavement thickness. An allowable ADTT should be chosen that is greater than expected for the development.
TABLE 4. **Recommended Minimum PCC Slab Thickness**

<table>
<thead>
<tr>
<th>Allowable ADTT</th>
<th>Minimum PCC Slab Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.8</td>
<td>5</td>
</tr>
<tr>
<td>13</td>
<td>5½</td>
</tr>
<tr>
<td>130</td>
<td>6</td>
</tr>
</tbody>
</table>

Our design is based on an R-value of 5 and a 28-day unconfined compressive strength for concrete of at least 3,500 pounds per square inch. In addition, our design assumes that pavements are restrained laterally by a concrete shoulder or curb and that all PCC pavements are underlain by at least 6 inches of Class 2 aggregate base. We recommend that adequate construction and control joints be used in design of the Portland Cement concrete pavements to control the cracking inherent in this construction.

Aggregate base should conform to the material requirements of Caltrans Standard Specifications, latest edition. Pavement subgrade and base material should be prepared and compacted as described in the “Earthwork” section of the report.

We recommend that exterior concrete sidewalks be at least 4 inches thick and underlain by at least 4 inches of Class 2 aggregate base compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition. If sidewalks are subject to wheel loads, they should be designed in accordance with the "Portland Cement Concrete Pavements" section of this report.
REFERENCES


VICINITY MAP
STANFORD SHOPPING CENTER ADDITIONS
Palo Alto, California

"Reproduced with permission granted by THOMAS BROS. MAP"

LOVNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services
<table>
<thead>
<tr>
<th>CORE NO.</th>
<th>THICKNESS (IN.)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>C-1</td>
<td>4</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>C-2</td>
<td>2½</td>
<td>6½</td>
<td></td>
</tr>
<tr>
<td>C-3</td>
<td>2</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>C-4</td>
<td>2</td>
<td>7</td>
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</tr>
<tr>
<td>C-5</td>
<td>2</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>C-6</td>
<td>5</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>C-7</td>
<td>2½</td>
<td>6½</td>
<td></td>
</tr>
<tr>
<td>C-8</td>
<td>5</td>
<td>4</td>
<td></td>
</tr>
<tr>
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<td>2</td>
<td>7</td>
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</tr>
<tr>
<td>C-10</td>
<td>2½</td>
<td>6½</td>
<td></td>
</tr>
<tr>
<td>C-11</td>
<td>2½</td>
<td>6½</td>
<td></td>
</tr>
<tr>
<td>C-12</td>
<td>2½</td>
<td>6½</td>
<td></td>
</tr>
<tr>
<td>C-13</td>
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<td>9</td>
<td></td>
</tr>
<tr>
<td>C-14</td>
<td>2</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>C-15</td>
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<td>6</td>
<td></td>
</tr>
<tr>
<td>C-16</td>
<td>3</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>C-17</td>
<td>2½</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>C-18</td>
<td>6</td>
<td>3</td>
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<td>C-20</td>
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<td>9</td>
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</tr>
<tr>
<td>C-21</td>
<td>3</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>C-22</td>
<td>2</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>C-23</td>
<td>2½</td>
<td>6½</td>
<td></td>
</tr>
<tr>
<td>C-24</td>
<td>2½</td>
<td>6½</td>
<td></td>
</tr>
<tr>
<td>C-25</td>
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<td>9</td>
<td></td>
</tr>
<tr>
<td>C-26</td>
<td>2</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>C-27</td>
<td>2½</td>
<td>6½</td>
<td></td>
</tr>
<tr>
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<td>8</td>
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</tr>
<tr>
<td>C-29</td>
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<td>7</td>
<td></td>
</tr>
<tr>
<td>C-30</td>
<td>3</td>
<td>5½</td>
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</tr>
<tr>
<td>C-31</td>
<td>7</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>EB-1</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>EB-2</td>
<td>4</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>EB-3</td>
<td>3</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>EB-4</td>
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</tr>
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<td>EB-5</td>
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<td>8</td>
<td></td>
</tr>
<tr>
<td>EB-7</td>
<td>4</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>EB-8</td>
<td>4</td>
<td>8</td>
<td></td>
</tr>
</tbody>
</table>

LEGEND

- Approximate location of casing
- Approximate location of previous

SITE PLAN
STANFORD SHOPPING CENTER ADDITIONS
Palo Alto, California

LOVNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

Base by Brian Kangas Foulk.
APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using a truck-mounted hollow-stem auger. Eight 8-inch diameter exploratory borings were drilled on May 12 and 13, 1998, to a maximum depth of 35 feet. Supplemental exploratory coring was performed on October 9, 1998. The approximate locations of the exploratory borings and corings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D-2488). The logs of the borings, as well as a key to the classification of the soil, are included as part of this appendix.

The locations of borings were approximately determined by pacing from existing site boundaries. Elevations of the borings were determined by interpolation from plan contours. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D-1586). In addition, 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

Field tests included an evaluation of the undrained shear strength of soil samples using a Torvane device, and the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depth.
### Primary Divisions

<table>
<thead>
<tr>
<th>GRAVELS</th>
<th>CLEAN GRAVELS (Less than 5% Fines)</th>
<th>GW</th>
<th>Well graded gravels, gravel—sand mixtures, little or no fines</th>
</tr>
</thead>
<tbody>
<tr>
<td>GM</td>
<td>Silty gravels, gravel—sand—silt mixtures, plastic fines</td>
<td></td>
<td></td>
</tr>
<tr>
<td>GC</td>
<td>Clayey gravels, gravel—sand—clay mixtures, plastic fines</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SW</td>
<td>Well graded sands, gravelly sands, little or no fines</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SP</td>
<td>Poorly graded sands or gravelly sands, little or no fines</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SM</td>
<td>Silty sands, sand—silt mixtures, non-plastic fines</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SC</td>
<td>Clayey sands, sand—clay mixtures, plastic fines</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ML</td>
<td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>OL</td>
<td>Organic silts and organic silt mixtures of low plasticity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CH</td>
<td>Inorganic clays of high plasticity, fat clays</td>
<td></td>
<td></td>
</tr>
<tr>
<td>OH</td>
<td>Organic clays of medium to high plasticity, organic silts</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PT</td>
<td>Peat and other highly organic soils</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Secondary Divisions

<table>
<thead>
<tr>
<th>Silt and Clay</th>
<th>Liquid Limit is Less Than 50%</th>
</tr>
</thead>
<tbody>
<tr>
<td>ML</td>
<td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity</td>
</tr>
<tr>
<td>OL</td>
<td>Organic silts and organic silt mixtures of low plasticity</td>
</tr>
<tr>
<td>CH</td>
<td>Inorganic clays of high plasticity, fat clays</td>
</tr>
</tbody>
</table>

### Fine Grained Soils

<table>
<thead>
<tr>
<th>Silt and Clay</th>
<th>Liquid Limit is Greater Than 50%</th>
</tr>
</thead>
<tbody>
<tr>
<td>MH</td>
<td>Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts</td>
</tr>
<tr>
<td>OH</td>
<td>Organic clays of medium to high plasticity, organic silts</td>
</tr>
</tbody>
</table>

### Highly Organic Soils

### Definition of Terms

<table>
<thead>
<tr>
<th>U.S. Standard Sieve Size</th>
<th>Clear Square Sieve Openings</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>40</td>
</tr>
<tr>
<td>40</td>
<td>10</td>
</tr>
<tr>
<td>10</td>
<td>4</td>
</tr>
<tr>
<td>4</td>
<td>3/4&quot;</td>
</tr>
<tr>
<td>3&quot;</td>
<td>3&quot;</td>
</tr>
<tr>
<td>12&quot;</td>
<td></td>
</tr>
</tbody>
</table>

### Grain Sizes

- **Terzaghi Split Spoon Standard Penetration**
- **Modified California**
- **D & M Underwater Sampler**
- **Piston Sampler**

### Samplers

- **At time of drilling**
- **Measured following drilling**

### Ground Water

<table>
<thead>
<tr>
<th>Sand and Gravel</th>
<th>Blows/foot*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>0—4</td>
</tr>
<tr>
<td>Loose</td>
<td>4—10</td>
</tr>
<tr>
<td>Medium Dense</td>
<td>10—30</td>
</tr>
<tr>
<td>Dense</td>
<td>30—50</td>
</tr>
<tr>
<td>Very Dense</td>
<td>OVER 50</td>
</tr>
</tbody>
</table>

### Relative Density

- **Key to Exploratory Boring Logs**

### Consistency

<table>
<thead>
<tr>
<th>Silt and Clay</th>
<th>Strength+</th>
<th>Blows/foot*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>0—1/4</td>
<td>0—2</td>
</tr>
<tr>
<td>Soft</td>
<td>1/4—1/2</td>
<td>2—4</td>
</tr>
<tr>
<td>Medium Stiff</td>
<td>1/2—1</td>
<td>4—8</td>
</tr>
<tr>
<td>Stiff</td>
<td>12</td>
<td>8—16</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>2—4</td>
<td>16—32</td>
</tr>
<tr>
<td>Hard</td>
<td>OVER 4</td>
<td>OVER 32</td>
</tr>
</tbody>
</table>

*Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. (1—3/8 inch I.D.) split spoon (ASTM D—1586). Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D—1586), point penetrometer, torque, or visual observation.

**Unified Soil Classification System (ASTM D—2487)**

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The attached boring logs and related information depict subsurface conditions only at the locations indicated and at the particular date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.
MATERIAL DESCRIPTION AND REMARKS

3 inches asphaltic concrete over 4 inches aggregate base

SILTY CLAY (CL)
hard, moist, reddish brown, low plasticity, trace very fine sand

SANDY CLAY (CL)
hard, moist, reddish brown, low plasticity, very fine sand, trace silt

CLAYEY SILT (ML)
very stiff to hard, moist, reddish brown, low plasticity, trace fine sand, trace claystone inclusions
increasing clay content

Bottom of Boring = 20 feet

Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual.

*Pocket Penetrometer Strength
MATERIAL DESCRIPTION AND REMARKS

4 inches asphaltic concrete over 8 inches aggregate base

Silty Clay (CL)
- hard, moist, reddish brown, low to moderate plasticity, trace fine sand
- Plasticity Index = 18, Liquid Limit = 38

Clayey Silt (ML)
- hard, moist, reddish brown, low plasticity, fine sand, trace claystone inclusions

Silty Clay (CL)
- hard, moist, reddish brown with gray mottling, moderate plasticity

Clayey Sand (SC)
- medium dense, moist, brown, fine to coarse sand, trace gravel
- 13 percent passing #200 sieve

Bottom of Boring = 20 feet

Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual.

*Pocket Penetrometer Strength
EXPLORATORY BORING – EB-3
STANFORD SHOPPING CENTER ADDITIONS
Palo Alto, California

MATERIAL DESCRIPTION AND REMARKS

3 inches asphaltic concrete over 7 inches aggregate base

CLAYEY SAND (SC)
dense, moist, reddish brown, fine to coarse sand, trace sub-angular
gravel to 1-inch
18 percent passing #200 sieve

SAND (SP-SC)
medium dense, moist, reddish brown, fine to coarse sand, trace gravel
to 1-inch and clay
9 percent passing #200 sieve
decreasing fines, increasing gravel

SAND (SP)
very dense, moist, reddish brown, fine to coarse, trace sub-rounded
gravel to 1.5-inch, trace clay

SAND (SP-SC)
medium dense, moist, brown, fine to coarse sand, trace sub-rounded
gravel to 1-inch and trace clay

Bottom of Boring = 21 1/2 feet

Note: The stratification lines represent
the approximate boundary between the soil
types. The transition may be gradual.

*Pocket Penetrometer Strength
<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Material Description and Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>CL</td>
<td>4 inches asphaltic concrete over 8 inches aggregate base</td>
</tr>
<tr>
<td></td>
<td>SILTY CLAY (CL) hard, moist, dark brown, low plasticity</td>
</tr>
<tr>
<td></td>
<td>Silt (ML) hard, moist, brown, low plasticity, trace clay, trace very fine sand and rootlets</td>
</tr>
<tr>
<td></td>
<td>trace claystone inclusions</td>
</tr>
<tr>
<td></td>
<td>hard, increasing sand and gravel</td>
</tr>
<tr>
<td>CL</td>
<td>Silt (ML) hard, moist, reddish brown with gray mottling, low plasticity, with fine sand and trace claystone inclusions</td>
</tr>
<tr>
<td></td>
<td>Increasing silt content, grades to silt</td>
</tr>
<tr>
<td>ML</td>
<td>Silty clay (CL) hard, moist, reddish brown, low plasticity, trace very fine to fine sand and trace clay</td>
</tr>
<tr>
<td></td>
<td>Grades to silty clay</td>
</tr>
<tr>
<td>CL</td>
<td>Silty clay (CL) hard, moist, reddish brown, low plasticity, trace fine sand</td>
</tr>
</tbody>
</table>

Bottom of Boring = 35 feet

Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual.

*Pocket Penetrometer Strength
### Material Description and Remarks

- **4 inches asphaltic concrete over 8 inches aggregate base**

- **Silty Clay (CL) [Fill]**
  - Very stiff to hard, moist, dark brown, moderate plasticity, fine to coarse sand, trace sub-angular gravel to 3/4-inch

- **Silty Clay (CH)**
  - Very stiff, moist, dark brown, moderate to high plasticity
  - Very stiff, moist, brown, moderate plasticity, trace fine sand
  - Hard
  - Increasing silt and fine sand
  - Decreasing silt, trace fine gravel

- **Bottom of Boring = 20 feet**

Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual.

*Pocket Penetrometer Strength*
**EXPLORATORY BORING - EB-6**

**STANFORD SHOPPING CENTER ADDITIONS**

Palo Alto, California

**LOWNEY ASSOCIATES**

Environmental/Geotechnical/Engineering Services

---

**MATERIAL DESCRIPTION AND REMARKS**

<table>
<thead>
<tr>
<th>SOIL TYPE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CL</strong></td>
<td>4 inches asphaltic concrete over 3 inches aggregate base</td>
</tr>
<tr>
<td><strong>ML</strong></td>
<td>SILTY CLAY (ML) stiff, moist, brown, low plasticity, trace clay and trace very fine sand</td>
</tr>
<tr>
<td><strong>CL</strong></td>
<td>SILTY CLAY (CL) very stiff, moist, brown, low plasticity, trace claystone inclusions</td>
</tr>
<tr>
<td><strong>CL</strong></td>
<td>very stiff, increasing coarse sand and gravel, trace sandstone fragments</td>
</tr>
</tbody>
</table>

**Bottom of Boring = 25 feet**

*Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual.*

*Pocket Penetrometer Strength*
<table>
<thead>
<tr>
<th>UNCONSOL. COMPR. STRENGTH</th>
<th>DENSITY AT TYPICAL SAMPLE</th>
<th>DRY DENSITY (pcf)</th>
<th>WATER CONTENT (%)</th>
<th>PENETRATION RESISTANCE</th>
<th>SAMPLER</th>
<th>SOIL TYPE</th>
<th>MATERIAL DESCRIPTION AND REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.7</td>
<td>1.6</td>
<td>98</td>
<td>25</td>
<td>30</td>
<td>CL</td>
<td>4 inches asphaltic concrete over 8 inches aggregate base</td>
<td></td>
</tr>
<tr>
<td>6.5</td>
<td>2.8</td>
<td>120</td>
<td>14</td>
<td>44</td>
<td>CL</td>
<td>SILTY CLAY (CL) very stiff, moist, brown with trace red mottling, moderate plasticity</td>
<td></td>
</tr>
<tr>
<td>9.0+</td>
<td>4.0</td>
<td>120</td>
<td>15</td>
<td>50</td>
<td>CL</td>
<td>SANDY CLAY (CL) hard, moist, reddish brown, low plasticity, fine to coarse sand, gravel to 1.5 inch, trace sandstone fragments</td>
<td></td>
</tr>
<tr>
<td>9.0+</td>
<td>2.8</td>
<td>111</td>
<td>14</td>
<td>37</td>
<td>SP</td>
<td>increasing gravel</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>SP</td>
<td>SAND (SP) very dense, moist, brown, fine to coarse sand and gravel, trace clay 4 percent passing #200 sieve</td>
<td></td>
</tr>
</tbody>
</table>

Bottom of Boring = 20 feet

Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual.

*Pocket Penetrometer Strength

EXPLORATORY BORING - EB-7
STANFORD SHOPPING CENTER ADDITIONS
Palo Alto, California

LOWNEY ASSOCIATES
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**EXPLORATORY BORING – EB-8**

**STANFORD SHOPPING CENTER ADDITIONS**

Palo Alto, California

---

### MATERIAL DESCRIPTION AND REMARKS

<table>
<thead>
<tr>
<th>UNCONSOLIDATED</th>
<th>SHEAR CONDUCTIVITY (NS)</th>
<th>DRY BULK DENSITY (PSP)</th>
<th>WATER CONTENT (%)</th>
<th>PENETRATION RESISTANCE (BMP)</th>
<th>DEPTH (FEET)</th>
<th>LEGEND</th>
<th>SOIL TYPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.0+</td>
<td>103</td>
<td>15</td>
<td>39</td>
<td></td>
<td></td>
<td>CL</td>
<td>SILTY CLAY (CL) hard, moist, reddish brown, moderate plasticity</td>
</tr>
<tr>
<td>9.0+</td>
<td>98</td>
<td>18</td>
<td>83</td>
<td></td>
<td>5</td>
<td>SM</td>
<td>SILTY SAND (SM) dense, moist, reddish brown, fine to medium sand, silty, trace coarse sand and clay 31 percent passing #200 sieve</td>
</tr>
<tr>
<td>9.0+</td>
<td>99</td>
<td>14</td>
<td>35</td>
<td></td>
<td>10</td>
<td>CL</td>
<td>SILTY CLAY (CL) hard, moist, reddish brown, moderate plasticity, trace very fine sand,</td>
</tr>
<tr>
<td>9.0+</td>
<td>104</td>
<td>9</td>
<td>76</td>
<td></td>
<td>15</td>
<td></td>
<td>Bottom of Boring = 20 feet.</td>
</tr>
</tbody>
</table>

Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual.

*Pocket Penetrometer Strength*
APPENDIX B
LABORATORY INVESTIGATION

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

**Moisture Content:** The natural water content was determined (ASTM D-2216) on 53 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

**Dry Densities:** In place dry density determinations (ASTM D-2937) were performed on 38 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

**Washed Sieve Analyses:** The percent soil fraction passing the No. 200 sieve (ASTM D-2217) was determined on six samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

**R-Value:** An R-value resistance test (California Test Method No. 301) was performed on a representative sample of the surface soils at the site to provide data for the pavement design. The test indicated an R-value of 7 at an exudation pressure of 300 pounds per square inch. The results of the test are presented on Table B-1.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Description of Material</th>
<th>Water Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Exudation Pressure (psi)</th>
<th>&quot;R&quot; Value</th>
<th>Expansion Pressure (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>Brown Silty Clay with Trace Gravel</td>
<td>25.2</td>
<td>96.1</td>
<td>165</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>19.5</td>
<td>106.2</td>
<td>295</td>
<td>6</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>17.1</td>
<td>110.0</td>
<td>382</td>
<td>9</td>
<td>0</td>
</tr>
</tbody>
</table>

* * * * * * * * *
PLASTICITY CHART AND DATA

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE B-1
350-163
REGIONAL FAULT MAP
STANFORD SHOPPING CENTER ADDITIONS
Palo Alto, California

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FIGURE 3
350-163