GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
VICINITY OF INTERSTATE 10 and HAVEN AVENUE
ONTARIO, CALIFORNIA

Project No. 21861

October 15, 2002

KLEINFELDER
An employee owned company

October 15, 2002
Project No. 21861

Mr. John Burroughs
Commerce Construction Company, L.P.
13191 Crossroads Parkway North, Sixth Floor
City of Industry, California 91746-3497

Subject: **Report of Geotechnical Investigation**
Proposed Commercial Development
Vicinity of Interstate 10 and Haven Avenue
Ontario, California

Dear Mr. Burroughs:

Kleinfelder, Inc. is pleased to present this report of geotechnical investigation performed for the proposed Commercial Development. The site is located south of Interstate 10 and west of Haven Avenue in Ontario, California.

In summary, it is our opinion that the proposed buildings may be constructed using conventional spread footings and slab-on-grade floors supported on engineered fill, provided the recommendations presented in the attached report are incorporated into design and construction. The upper onsite natural soils are loose to medium dense and appear to have a low to moderate potential for hydro-consolidation when subject to saturation by water. In order to provide uniform support of the structures and reduce the potential for excessive settlement, we recommend that the upper native soils be overexcavated and recompacted as engineered fill to provide support of foundations, floor slabs and pavements. The conclusions and recommendations presented in this report are subject to the limitations presented in Section 7.

We appreciate the opportunity of providing geotechnical engineering services to you on this project. If you should have any questions or require additional information, please contact us.

Respectfully submitted,

KLEINFELDER, INC.

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1 INTRODUCTION

Kleinfelder, Inc. was retained by Commerce Construction Company, L.P. to conduct a geotechnical investigation for the proposed commercial development in Ontario, California. Our services were performed in general accordance with our proposal dated June 26, that was authorize by Mr. John Burroughs on September 20, 2002.

The site is located south of Interstate 10 and east of Haven Avenue in Ontario, California. The location of the site is shown on Plate 1, Site Location Map. The proposed site layout is shown on Plate 2, Plot Plan.

1.1 PURPOSE AND SCOPE

The purpose of this geotechnical investigation was to evaluate the subsurface soil conditions at the subject site and provide geotechnical recommendations for design and construction of the project. A description of the scope of work performed is presented below.

Task 1 – Literature Review/Utility Clearance. We reviewed published and unpublished geologic literature in our files and the files of selected public agencies including publications prepared by the California Division of Mines and Geology and the U.S. Geological Survey. We reviewed available appropriate seismic and faulting information including designated earthquake fault zones and our in-house database of faulting in the general site vicinity.

Prior to conducting the field investigation program. Each of our proposed boring and test pit locations were cleared for known existing utility lines with the participating utility companies through Underground Service Alert (USA).

Task 2 – Field Exploration. A total of 9 hollow-stem auger borings (B-1 through B-9) were advanced in our current investigation at the project site to depths ranging from approximately 11½ feet to 51½ feet below existing grade. Eight test pits were also excavated to depths of approximately 6 to 8 feet below existing grades. The approximate locations of the borings and test pits are presented on Plate 2, Plot Plan. A Kleinfelder engineer supervised the field operations and logged the borings and test pits. Selected bulk, disturbed and relatively undisturbed samples were retrieved, sealed and transported to our laboratory for further evaluation. The number of blows necessary to drive both a Standard Penetration Test (SPT) sampler and a California-type sampler were recorded. A description of the field exploration and a Legend to the Logs of Borings is presented in Appendix A.

Task 3 – Laboratory Testing. Laboratory testing was performed on representative relatively undisturbed and disturbed samples to substantiate field classifications and to provide engineering parameters for geotechnical design. Testing consisted of:

- Moisture content and dry density
- #200 wash sieve
- Collapse potential
- R-Value
- Preliminary Corrosivity Tests

The results of our laboratory testing are presented in Appendix B.

Task 4 – Geotechnical Analyses. We evaluated the field and laboratory data in conjunction with the site plan and estimated building loads. We also evaluated potential foundation systems, lateral earth pressures, settlement, pavement design, and earthworks considerations. Potential geologic hazards were evaluated such as ground shaking, liquefaction potential, fault rupture hazard and seismically-induced settlement. Design recommendations for use with standard UBC (1997) seismic design were considered.

Task 5 – Report Preparation. This report was prepared presenting our findings, conclusions and recommendations for earthwork and foundation engineering. Recommendations for foundation type(s), allowable bearing pressure, estimated settlement, passive resistance, lateral earth pressures for retaining structures, pavements, earthwork, and seismicity are presented. This report also contains a site map, logs of the borings and laboratory test results.

1.2 PROPOSED PROJECT

Based on information provided by Commerce Construction Company and a site plan for the proposed development dated August 19, 2002, it is our understanding that the project consists of a commercial development with 7 proposed building pads and related parking. The proposed building pads along the northern property line adjacent to the 10 Freeway and on-ramp range in size from 15,000 to 25,000 square feet. One larger proposed building pad, located along the southern property line is approximately 170,000 square-feet. The total building area will consist of over 311,500 square feet and the development will include roughly 900 surface parking stalls. Construction is anticipated to be masonry, or concrete tilt-up buildings with concrete slab-on-grade floors. We understand that no basement levels are planned for the buildings in the development.

Detailed structural loads for the buildings are not available at this time. However, maximum column loads are anticipated to be on the order of 120 kips and maximum wall loads on the order of 4 to 5 kips per lineal foot. Slabs-on-grade are expected to support maximum floor loads of between 150 to 200 pounds per square foot.

Grading plans for the site were not available at the time of this report, however, site grading is anticipated to include cuts and fills of approximately 1 to 2 feet.

2 SITE AND SUBSURFACE CONDITIONS

2.1 SITE DESCRIPTION

The site of the proposed development is located south of Interstate 10 and east of Haven Avenue in Ontario, California. The location of the site is shown on Plate 2, Plot Plan. The site is currently divided into two portions by a chain-link fence that traverses the site from east to west as indicated on the Plot Plan. The northern portion of the site is currently a vacant field that is generally flat and covered with minor shrubs. There is an existing sewer line that runs from the southwest to northeast along the western portion of the property. There are existing Edison easements that run east to west, parallel to the chain-link fence and east to west along the southern property boundary and ends near the middle of the property. The southern portion of the site is currently occupied by a roofing distribution company. The majority of the southern portion of the site is paved with asphalt concrete and used as storage for the roofing materials. There is an existing building structure located just south of the chain-link fence as indicated on the Plot Plan.

2.2 SUBSURFACE SOIL CONDITIONS

2.2.1 General

The following paragraphs summarize the results of our field exploration. The boring logs (presented in Appendix A) should be reviewed for a more detailed description of the subsurface conditions at the locations explored.

2.2.2 Artificial Fill

Fill soils were identified in two of our test pits (TP-3 and TP-4) conducted at site. The fill encountered in TP-3 was encountered to a depth of approximately 6 feet below the current grade and consisted of silty sand with varying amounts of plastic debris. The fill at the site appears to be localized. Shallow fill of approximately 6 inches was encountered in TP-4. Although fill was not identified at the other locations explored at the site, we anticipate that some shallow fill should be expected within the previously paved areas and adjacent to and beneath existing structures or substructures. There is also a possibility of other localized deeper fill deposits in other areas between borings. Deeper and/or poorer quality fill may exist between the locations explored during our investigation.

2.2.3 Native Soils

The natural soils encountered in our borings and test pit explorations generally consisted of loose to medium dense silty sands and sand with varying amounts of gravel. The in-situ densities tested varied from approximately 100 to 108 pounds per cubic foot at moisture contents ranging from 0.5 to 10.6 percent. The upper native soils are considered to have a low to moderate potential for hydro-consolidation, meaning that the soils become significantly weaker and more compressible when wet or saturated. See Appendix A for a more detailed explanation of the field exploration.

2.3 SURFACE AND GROUNDWATER CONDITIONS

Surface water flow across the project site is via sheet flow towards the south, following local topography. The project site is not located within any currently designated flood hazard zone as currently designated by the FEMA Map Service Center Web Site (FEMA, 2002).

Groundwater was not encountered within the 9 borings advanced to a maximum depth of 51.5 feet for this investigation. The site of the project is located within the Chino Groundwater Basin. The depth to historic high groundwater within the proposed project site is reported at approximately 350 feet below the ground surface according to currently available information (WMWD, 2001 and Carson, Matti, 1985).

Fluctuations of the groundwater level, localized zones of perched water, and soil moisture content should be anticipated during and following the rainy season. Irrigation of landscaped areas can also cause a fluctuation of local groundwater levels.

3 GEOLOGIC CONDITIONS

3.1 GEOLOGIC SETTING

The site is located on the Chino Valley portion of the upper Santa Ana River Drainage of the Perris Block, within the Peninsular Ranges geomorphic province of California. Locally, this area lies near the transition zone between the Transverse Ranges geomorphic province to the north and the Peninsular Ranges geomorphic province on the south. The Peninsular Ranges are a northwest-southeast oriented complex of mountain ranges and valleys formed by sub-unit blocks that are separated by similarly trending strike slip faults.

3.2 GEOLOGIC MATERIALS

The site has been regionally mapped to be underlain by surficial sediments including minor amounts of surficial fills (unmapped), wind blown sand, and younger (Holocene) lower alluvial fan deposits. These younger alluvial fan deposits include materials locally derived from the San Gabriel Mountains to the north via Deer and Etiwanda Creeks. The fan deposits consist of mixtures of unconsolidated sands, silty sands and gravelly silty sands with cobbles. The wind blown deposits consist of fine sands. The soils encountered during the field investigation also consisted of mixtures of these soils. Beneath the younger alluvial fan deposits are the older alluvial fan materials both of which are estimated to be at combined thickness of least 400 feet thick (Fife et al, 1976).

3.3 GEOLOGIC HAZARDS

The site is currently not located within a State of California designated Earthquake Fault Zone. Therefore, the likelihood of ground surface rupture due to primary faulting from known faults is considered to be low. Based on the materials encountered at this site, the existing topographic conditions, analyses performed and the proposed site improvements, we do not expect seismic slope instability to be a concern. Also, due to the site's elevated inland location and proximity from any large bodies of impounded water, we believe that tsunamis and/or seiches should not be considered a potential hazard to the project. The site is not listed within a 100-year or 500-year flood hazard zone as designated by FEMA (FEMA web site, 2002).

In our opinion, the most significant geologic hazard to the project is the potential for moderate to strong ground shaking resulting from earthquakes generated on the faults within the vicinity of the site. In the vicinity of the site, approximately 39 known active faults have been mapped within a 62-mile (100-kilometer) radius of the site.

3.4 FAULTING AND SEISMICITY

We consider the most significant geologic hazard to the project to be the potential for moderate to severe seismic shaking that is likely to occur during the design life of the proposed project. The project site is located in the highly seismic Southern California region within the influence of several fault systems that are considered to be active or potentially active. An active fault is defined by the State of California as a "sufficiently active and well defined fault and has exhibited surface displacement within the Holocene time (about the last 11,000 years)". A potentially active fault is defined by the State as a fault with a history of movement within Pleistocene time (between 11,000 and 1.6 million years ago). These active and potentially active faults are capable of producing potentially damaging seismic shaking at the site. It is anticipated that the project site will periodically experience ground acceleration as the result of small to moderate magnitude earthquakes. Other active faults without surface expression (Blind faults) are also capable of generating an earthquake, or other potentially active seismic sources are not currently zoned.

Faults identified by the State as being either active or potentially active are not known to be present on-site. The site is not located within a State of California designated Earthquake Fault Zone for ground rupture (Hart and Bryant, 2000). Nor is the site located within any other currently designated seismic hazards zone for Liquefaction Potential or Seismically Induced Slope Instability (the Guasti Quad has yet to be mapped for these hazards).

We have listed within Table 1 the known faults in the region that, in our opinion, could significantly impact the site. In addition, recent experience and current research indicates that "blind faults" (faults that apparently have not broken the surface and display little or no surface expression) may underlie adjacent areas north of the site and portions of Los Angeles and

Orange Counties west and southwest of the site. Blind thrust faults are known to be responsible for both the M5.9 Whittier Narrows earthquake (1987) and the M6.7 Northridge earthquake (1994).

We have performed a computer-aided search of the known active and potentially active faults within a 62-mile (100 kilometer) radius of the site, researched available literature to assess the maximum credible earthquakes expected to be generated on each fault. Table 1 summarizes these parameters for the four out of the thirty-nine known active and potentially active faults within the searched radius of the site that in our opinion will have the greatest impact upon the site. Selection of the faults is based on their proximity to the site and their potential to generate strong ground motion on the site. Table 1 was generated using in part the EQFAULT computer program (Blake, 2000) as modified using the fault parameters from DMG Open File Report 96-08 and the 1997 UBC fault maps (ICBO 1998). This table does not identify the probability of reactivation or the onsite effects from earthquakes occurring on any of the other faults in the region. The site is located on the USGS Guasti, California 7½' Quadrangle Map, at Latitude 34.0654 °N and Longitude 117.5726°W, at approximately the 975 foot elevation (MSL).

Table 1
Significant Faults

Fault Name	Approx. Distance From Site Km (mi)	Maximum Event (Moment Mag.)	Fault Seismic Source Type
San Jose	13.2 (8.2)	6.5	B
Cucamonga	14.9 (9.3)	7.0	A
Chino - Central Avenue	16.7 (10.4)	6.7	B
Sierra Madre	17.4 (10.8)	7.0	B

A number of moderate earthquakes have occurred in the vicinity of the project site in the past 201 years, and the site is located in an area of high seismicity of low magnitude events from an unknown seismic source known as the Fontana trend. The parameters used to define the limits of the historical earthquake search include geographical limits (within 62 mi. of the site), dates (1800 through 2000), and magnitude (magnitudes above M 4). A summary of the historical search is presented below.

Time period (1800 to 2000)	201 years
Maximum Magnitude within 62 mi (100 km) radius (12/16/1858, 12/8/1812)	7.0
Calculated Max. Historic Site Acceleration During period, M7.0 (12/16/1858)	0.40g
Approximate distance to nearest historical earthquake, > M4.0	8.3 km
Number of events exceeding magnitude 4 within the search area	601

Under the current understanding of regional seismo-tectonics, the largest maximum credible event to impact the site may be generated by faults related to the Cucamonga Fault having moment magnitude of M7.0. The USGS indicates a 10% probability of exceedance in 50 years for an acceleration of 0.52g for alluvial sites within this area (USGS, 2002).

3.5 NEAR-SOURCE SEISMIC ZONE

In addition to the determination of fault activity, faults are also type classified as an A, B, or C for Near-Source Zone ground motion (Ca, Cv, Na and Nv) by the both State and ICBO in the Uniform Building Code (UBC) according to parameters of known slip rate and maximum earthquake magnitude. A "Type A" fault has a magnitude M≥7.0 and slip rate≥5mm/yr. A "Type B" has a magnitude 6.0<M<7.0 and slip rate between 2mm and 5mm/yr. A "Type C" has a magnitude M<6.5 and a slip rate of <2mm/yr, or is unrated under the current knowledge.) The site is located at approximately the 11.5 km Near Source Seismic zone distance for the Cucamonga Fault, a Type A fault as designated by the UBC (ICBO 1998). The site is located in Seismic Zone 4 of the latest edition of the Uniform Building Code (UBC). Structures should be designed in accordance with the values and parameters given within the UBC.

Please note that the fault distance presented in Table 1, Significant Faults, indicate the distance from the site to the nearest location where the fault trace is mapped at the ground surface. The Near Surface Zone Map distances are based on the shortest distance from the site to the fault plane projection to the surface from a depth of 10-km. In some cases the Near Source Zone Map distance is less than the distance shown in Table 1, because the site may be closer to the fault plane projection than the surface trace of the fault.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on our field exploration, laboratory testing and geotechnical analyses conducted for this study, it is our opinion that it is geotechnically feasible to construct the project as planned, provided the recommendations presented in this report are incorporated into project design and construction. There do not appear to be significant onsite geotechnical constraints that can not be overcome by proper planning, design and construction practices.

The upper natural soils at the site are relatively loose and exhibit a low to moderate potential for hydro-consolidation, and therefor are not considered suitable for the support of the proposed buildings. The upper native soils should be overexcavated and recompacted as properly compacted engineered fill to reduce the potential for settlement due to hydro-consolidation and to provide uniform support for the proposed structures.

If the grading recommendations are followed, the proposed buildings may be supported on the resulting engineered fill using shallow isolated and continuous spread footings and slab on grade floors.

4.2 SEISMIC DESIGN CONSIDERATIONS

The site is located in a seismically active region and the proposed facility can be expected to be subjected to strong seismic shaking during its design life. Potential seismic hazards include ground shaking, localized liquefaction, ground rupture due to faulting, and seismic settlement. The following sections discuss these potential seismic hazards with respect to this site.

4.2.1 Ground Shaking

Because this site is located in the seismically active Southern California region, we recommend that, as a minimum, the proposed development be designed in accordance with the requirements of the latest edition of the Uniform Building Code (UBC) for Seismic Zone 4. We recommend that a soil profile factor of S_D be used with the UBC design procedure (Table 16-J). Near source seismic coefficients for acceleration and velocity, Na and Nv (UBC Tables 16-S and 16-T) should be used for calculating the design. The site is located 11.5 Km from the Cucamonga Fault, a Type A Fault as designated by the 1997 UBC (ICBO, 1998). The site is within the 15 Km Active Fault Near-Source (Seismic) Zone for the Cucamonga Fault. A summary of the seismic parameters is presented below.

Design Fault	Cucamonga
Fault Type	A
Seismic Zone	4 (z = 0.4)
Soil Profile Factor (Table 16-J)	S_D
Near-Source Distance	11.5 km
Na (Table 16-S)	1.0
Nv (Table 16-T)	1.14
Ca (Table 16-Q) ($0.44N_A$)	0.44
Cv (Table 16-R) ($0.64N_X$)	0.73

4.2.2 Liquefaction

Liquefaction is a phenomenon associated with shallow groundwater, in combination with the presence of loose, fine sands or silts within a depth of about 50 feet below grade or less. Liquefaction occurs when these soils are subject to strong ground shaking resulting from earthquake induced ground motion. Liquefaction typically causes these soils to lose a portion or all of their shear strength. This strength is typically regained sometime after the shaking stops. Soil movements (both vertical and lateral) have been observed under these conditions due to consolidation of the liquefied soils and the reduced shear resistance of slopes. Liquefaction potential decreases with an increase in clay and gravel content and a decrease in grain size. However the potential increases with an increase in the duration of the earthquake induced ground shaking.

The site is not located within a state or county designated liquefaction hazard zone. Based on our research, the reported depth to historical groundwater appears to be greater than 100 feet below grade. Due to the depth to historic high groundwater at the site, the potential for liquefaction occurrence is considered low.

4.2.3 Seismically-Induced Settlement

Seismically-induced settlement is surface settlement caused by densification of non-saturated soils due to earthquake-induced ground shaking. Based on our field data and utilizing procedures proposed by Tokimatsu and Seed (1987), we estimated seismically-induced settlement to be on the order of ½ to ¾-inch. Differential seismically-induced settlement is anticipated to be on the order of ¼ to ½-inch or less.

4.3 EARTHWORK

4.3.1 Site Preparation

Site preparation and earthwork operations should be performed in accordance with applicable codes and the recommendation included herein. Based on our field investigation, the upper natural soils appear to be loose and moisture sensitive and are not considered suitable for uniform support of the proposed buildings. The moisture sensitive soils have a low to moderate probability to experience significant loss in strength and increase in compressibility when wet. These soils should be over-excavated and replaced as properly compacted-engineered fill to mitigate the collapse potential.

Building Pads

We recommend that the building pads be overexcavated a minimum of 5 feet below the finished pad grade. Where fill is to be placed at the site to raise site grades in building areas, the overexcavation should extend at least 3 feet below the bottom of footings or at least 3 feet below the existing grades, whichever is deeper. The excavation beneath the building pads should extend a minimum of 5 feet laterally beyond the building footprints or the edge of footings, whichever is greater.

Parking Areas

In the area where surface paving is planned, the depth of overexcavation may be reduced to about 1.5 feet below existing grade. The 1.5 foot excavation below existing grades is recommended for areas to receive fill and where there will be less than 6 feet of cut to establish final grades.

General

Existing fill should be overexcavated and compacted as engineered fill. Localized areas requiring deeper fill removal should be anticipated.

Following the over-excavation of the upper loose soils, the exposed subgrade should be inspected for all unsuitable soils, rubble, and debris and the excavation deepened if necessary. The exposed subgrade should then be scarified to a depth of 6-inches, moisture conditioned to within 0 to 4 percent over the optimum moisture content and rolled with heavy compaction equipment. All soils should be compacted to at least 90 percent of maximum dry density as obtainable by ASTM Designation D-1557-91 method of compaction.

Following the subgrade preparation, the removed soils should be replaced in loose lifts of not more than 8-inches thick, brought to within 0 to 3 percent above optimum moisture and be mechanically compacted using heavy equipment. All soils should be compacted to at least 90 percent of maximum dry density as obtainable by ASTM Designation D-1557-91 method of compaction. The upper six inches of the pavement subgrade should be compacted to a minimum of 95 percent relative compaction. All earthwork operations should be observed and tested by a representative of this firm.

4.3.2 Materials for Fill

The onsite soils, less any debris or organic materials may be used for any required fill soils. Imported soils should be granular in nature and be relatively non-expansive with an expansion index of less than 35. The imported soils should contain 10 to 40 percent fines (percent passing the No. 200 sieve) to provide a stable subgrade and maintain low to medium permeability characteristics.

4.3.3 Excavation Conditions

The borings advanced at the site were advanced using a truck-mounted, hollow-stem auger drill rig. Drilling was completed with slight to moderate effort through the subsurface soils. Conventional earth moving equipment is expected to be capable of performing the excavations required for site development.

4.3.4 Collapsible Soils

Based on our laboratory testing, the upper natural soils at the site appear to have a low to moderate collapse potential. Collapsible soils (or moisture sensitive soils) are defined as a soil with a potential for a significant decrease in strength and increase in compressibility when wet or saturated. The collapsible soils should be overexcavated and recompacted as properly compacted-engineered fill during the earthwork operations.

4.3.5 Excavations and Temporary Slopes

Excavations deeper than 4 feet deep should be sloped back at 1:1 (horizontal to vertical) or be shored or braced for safety. Excavations extending below a 1½:1 (horizontal to vertical) plane extending down from any adjacent footings should be shored for safety. All excavations should be inspected by a representative of the geotechnical engineer during construction to allow any modifications to be made due to variations in the soil conditions.

During wet weather, earthen berms or other methods should be used to prevent runoff water from entering all excavations. All runoff water and/or groundwater encountered within excavations should be collected and disposed outside the construction limits.

All excavations must comply with applicable local, state, and federal safety regulations including the current OSHA Excavation and Trench Safety Standards. Construction site safety generally is the sole responsibility of the Contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. We are providing the information below solely as a service to our client. Under no circumstances should the information provided be interpreted to mean that Kleinfelder is assuming responsibility for construction site safety or the Contractor's activities; such responsibility is not being implied and should not be inferred.

The Contractor should be aware that slope height, slope inclination, or excavation depths (including utility trench excavations) should in no case exceed those specified in local, state, and/or federal safety regulations (e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations).

4.3.6 Trench Backfill

All required trench backfill should be mechanically compacted in 8-inch (maximum) layers with mechanical compaction equipment. Jetting and flooding is not recommended. We recommend all backfill be compacted to at least 90 percent of maximum dry density based on ASTM Designation D1557-91. The moisture content of compacted backfill soils should be within 0 to 3 percent over the optimum at the time of compaction. Some settlement of the backfill may be expected and any utilities within the trenches should be designed to accept differential settlement.

If imported material is used for pipe or trench zone backfill, we recommend it consist of fine-grained sand. In general, coarse-grained sand and/or gravel should not be used for pipe or trench zone backfill due to the potential for soil migration into the relatively large void spaces present in this type of material and water seepage along trenches backfilled with coarse-grained sand and/or gravel.

Recommendations provided above for pipe zone backfill are minimum requirements only. More stringent material specifications may be required to fulfill local building requirements and/or bedding requirements for specific types of pipes. We recommend the project Civil Engineer develop these material specifications based on planned pipe types, bedding conditions, and other factors beyond the scope of this study.

4.4 DRAINAGE AND LANDSCAPING

It is important that positive surface drainage be provided to prevent ponding and/or saturation of the soils in the vicinity of foundations and concrete slabs-on-grade. We recommend that the site be graded to carry surface water away from the proposed buildings and that positive measures be implemented to carry away roof runoff. The upper soils at the site are slightly to moderately collapsible when saturated with water. The recommended overexcavation is intended to mitigate the collapse potential of the upper soils. Poor perimeter or surface drainage could allow migration of water beneath the building or pavement areas, which may result in distress to project improvements. The following supplemental suggestions are also provided if it is desirable to further reduce the potential for migration of water under the building pad: When possible, we suggest that planters adjacent to buildings be placed at least 10 feet from the building footprint. If planted areas adjacent to the structure are desired, we recommend that care be taken not to over-irrigate and to maintain a leak-free sprinkler piping system. If possible, we suggest that planters be sealed. In addition, it is recommended that planter areas next to buildings have a minimum of 5 percent positive fall away from building perimeters to a distance of at least 5 feet. Drain spouts should be extended to discharge a minimum of 5 feet from the building, or some other method should be utilized to prevent water from accumulating in planters. Landscaping after construction should not promote ponding of water adjacent to structures.

4.5 FOUNDATIONS

4.5.1 Allowable Bearing Pressures

The proposed buildings may be supported on shallow, reinforced concrete, spread footings founded over at least 3 feet of properly compacted engineered fill soils. Continuous and isolated spread footings should have minimum widths of 18 inches and be embedded at least 18 inches below the lowest final adjacent subgrade. Within this report, the lowest adjacent grade refers to the finished exterior grade for the perimeter footings and the finished pad grade for interior footings. Footings established as recommended may be designed using an allowable bearing pressure of 3,000 pounds per square foot for dead plus sustained live loading.

The allowable bearing pressure provided above is a net value; therefore, the weight of the concrete may be assumed to be 50 pounds per square foot. The weight of the soil backfill may be neglected when computing dead loads. The allowable bearing pressure may be increased by one-third for short-term loading due to wind or seismic forces.

Footings may experience an overall loss in bearing capacity or an increased potential to settle where located in close proximity to existing or future utility trenches. Furthermore, stresses imposed by the footings on the utility lines may cause cracking, collapse and/or a loss of serviceability. To reduce this risk, footings should extend below a 1:1 plane projected upward from the closest bottom corner of the trench.

Footings for minor structures (loading dock walls, minor retaining walls, free standing walls, etc.) that are structurally separate from the building can be designed using an allowable bearing pressure of 1,000 pounds per square foot at a minimum depth of 18 inches below the lowest adjacent grade. Such footings should be underlain by at least 18 inches of properly compacted fill soils.

4.5.2 Estimated Settlements

Based on anticipated loading conditions, we estimate the total static settlement for the proposed buildings supported in the manner recommended to be on the order of ½ to 1 inch. Differential settlements between adjacent columns are estimated to be on the order of ½-inch. Seismically induced dry settlement is expected to be on the order of ½ to ¾ inch with differential settlement on the order of ¼ to ½ inch. The seismically induced settlements are in addition to the static settlements presented above.

Static settlement of all foundations is expected to be primarily elastic and should be essentially completed shortly after initial application of structural loads.

4.5.3 Lateral Resistance

Resistance to lateral loads (including those due to wind or seismic forces) may be provided by frictional resistance between the bottom of concrete foundations and the underlying soil and by passive soil pressure against the sides of the foundations. A coefficient of friction of 0.35 may be used between cast-in-place concrete foundations and the underlying soil. Passive pressure available in engineered fill may be taken as equivalent to the pressure exerted by a fluid weighing 300 pounds per cubic foot.

The passive resistance of the subgrade soils will diminish or be non-existent if trench sidewalls slough, cave or are overwidened during or following excavations. If this condition is encountered, our firm should be notified to review the condition and provide remedial recommendations, if necessary.

4.5.4 Construction/Design Considerations

Prior to placing steel or concrete, footing excavations should be cleaned of all debris, loose or soft soil, and water. Footing excavations should be observed by the project Geotechnical Engineer just prior to placing steel or concrete to verify the recommendations contained herein are implemented during construction.

4.7 CONCRETE SLABS SUPPORTED-ON-GRADE

It is our opinion that concrete slab-on-grade floors may be used for the proposed structures. The slab-on-grade may be placed on engineered fill prepared as described in Section 4.3.1. Slab thickness and reinforcement should be designed by the structural engineer. All slabs should be designed for any specific loading conditions by the structural engineer. A modulus of subgrade reaction of 150 pounds per cubic inch may be used for design. All areas adjacent to buildings, including planters, should be designed to drain away from the structure to avoid accumulation of water beneath the slab or footings.

Subsurface moisture and moisture vapor naturally migrate upward through the soil and, where the soil is covered by a building or pavement, this subsurface moisture will collect. To reduce the impact of this subsurface moisture and the potential impact of future introduced moisture (such as landscape irrigation or precipitation) the current industry standard is to place a vapor retarder on the prepared subgrade. This membrane typically consists of visquene or polyvinyl plastic sheeting at least 10 mil in thickness. Other proprietary moisture vapor barrier products such as stego wrap or equivalent may also be used. It should be noted that although vapor barrier systems are currently the industry standard, this system may not be completely effective in preventing floor slab moisture problems. Such proprietary products should be installed as recommended by the manufacturer's specifications. These systems typically will not necessarily assure that floor slab moisture transmission rates will meet floor-covering manufacturer standards and that indoor humidity levels be appropriate to inhibit mold growth. The design and construction of such systems are totally dependent on the proposed use and design of the proposed building and all elements of building design and function should be considered in the slab-on-grade floor design. Building design and construction may have a greater role in perceived moisture problems since sealed buildings/rooms or inadequate ventilation may produce excessive moisture in a building and affect indoor air quality.

Various factors such as surface grades, adjacent planters, the quality of slab concrete and the permeability of the on-site soils affect slab moisture and control future performance. In many cases, floor moisture problems are the result of either improper curing of floor slabs or improper application of flooring adhesives. We recommend contacting a flooring consultant experienced in the area of concrete slab-on-grade floors for specific recommendations regarding your proposed flooring applications.

Special precautions must be taken during the placement and curing of all concrete slabs. Excessive slump (high water-cement ratio) of the concrete an/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking or curling of the slabs. High water-cement ratio and/or improper curing also greatly increase the water vapor permeability of concrete. We recommend that all concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) Manual.

4.8 PAVEMENT DESIGN

An R-value test was performed on a selected sample to provide data for the design of paving. The test result, presented in Appendix B, indicates the R-value of the sample tested to be nearly equivalent to that used in pavement design for aggregate base. Although the R-value of the soil tested is equivalent to that of aggregate base, we do not anticipate that the onsite soils will conform to the other requirements of aggregate base (class 2 aggregate base or crushed miscellaneous base) which include sand equivalent, durability, and gradation. These criteria are a function of the long-term strength and drainage characteristics of the materials. Local jurisdictional agencies typically request that sufficient testing (R-value, gradation, sand equivalent, durability) be performed before use of an aggregate base material can be substituted with on-site materials. Its has been our experience with the City of Ontario that the pavement section should include a minimum of 4 inches of aggregate base.

For design of the pavement structural section including aggregate base, we recommend that a minimum 4-inch layer of aggregate base be placed beneath pavements. Three alternative pavement sections are provided below for preliminary design. Option 1 is based on an R-value of 78 (equivalent to aggregate base) with a 4-inch minimum aggregate base layer. For this option, the actual R-value may be as low as 60 when considering the minimum aggregate base thickness. Option 2 is based is based on a reduced R-value of 50 to account for areas that may have soils that may be different or do not perform to the same R-value as those tested. Option 3 consists of a full-depth asphalt layer, based on an R-value of 60. For the full-depth asphalt layer, we recommend that an R-value of 60 be used to accommodate for the lower factor of safety in design due to not having an underlying base layer. Also, the lower degree of drainage, and durability of the onsite soils increase the potential for localized pavement failures if the full depth pavement option is used. In all cases the R-value of the subgrade soils should be evaluated by the geotechnical engineer of record following subgrade preparation to access the applicable final pavement section to be used.

Pavement sections are provided for Traffic Index (TI) values of 4.0 through 7.0 for the uses below:

Traffic Index	Suggested Typical Usage
4.0	Automobile Parking
5.0	Automobile Access Drives
6.0	Truck Traffic
7.0	Heavy Truck Traffic Areas of Truck Maneuvering

We have developed the following preliminary recommendations for asphalt pavement (Table 1), and rigid Portland concrete cement pavements (Table 2). These recommendations are applicable for the entire site.

Table 1
Recommended Asphalt Concrete Pavement Sections

Traffic Index (TI)	Option 1 (R-value = 60 to 78)		Option 2 (R-value = 50)		Option 3 (R-value = 60)
	Asphaltic Concrete (inches)**	Class 2 Aggregate Base or CMB* (inches)	Asphaltic Concrete (inches)	Class 2 Aggregate Base or CMB* (inches)	Full Depth Asphalt (inches)**
4.0	2	4	2	4	3
5.0	2.5	4	2.5	4	4
6.0	3	4	3	4	5
7.0**	3.5	4	3.5	5.5	6

*CMB – Crushed Miscellaneous Base
**The City of Ontario may require a minimum of 4 inches below pavement unless the subgrade meets all requirements of aggregated base.

Table 2

Recommended Portland Cement Concrete Pavement Sections

Traffic Index (TI)	PCC Pavement (inches)	
	R-value =50	R-value = 78
4.0 or less	6	5
4.1 - 5.0	6	5.5
5.1 - 6.0	6.5	5.5
6.1 - 7.0	6.5	6
7.1 - 8.0	7	6

6 LIMITATIONS

This preliminary report has been prepared for the exclusive use of Commerce Construction Company, LP and their agents for specific application to the proposed commercial development in Ontario, California. The findings, conclusions and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering practice. No other warranty, expressed or implied, is made. We should review the final location map and grading plans to verify that our borings were properly located, and to develop recommendations for additional exploration, if appropriate, and to provide additional information.

Our evaluation of subsurface conditions at the site has considered subgrade soil and groundwater conditions present at the time of our investigation. The influence(s) of post-construction changes to these conditions such as introduction of water into the subsurface will likely influence future performance of the proposed project. Whereas our scope of services addresses present groundwater conditions, future irrigation, broken water pipelines, etc. may adversely influence the project and should be addressed and mitigated, as needed, by specialized slab and flooring system designers having local knowledge.

The scope of our geotechnical services did not include any environmental site assessment for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater or atmosphere, or the presence of wetlands.

The client has the responsibility to see that all parties to the project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. This report contains information, which may be useful in the preparation of contract specifications. However, the report is not designed as a specification document and may not contain sufficient information for this use without proper modification.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of this report and the nature of the new project, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

7 REFERENCES

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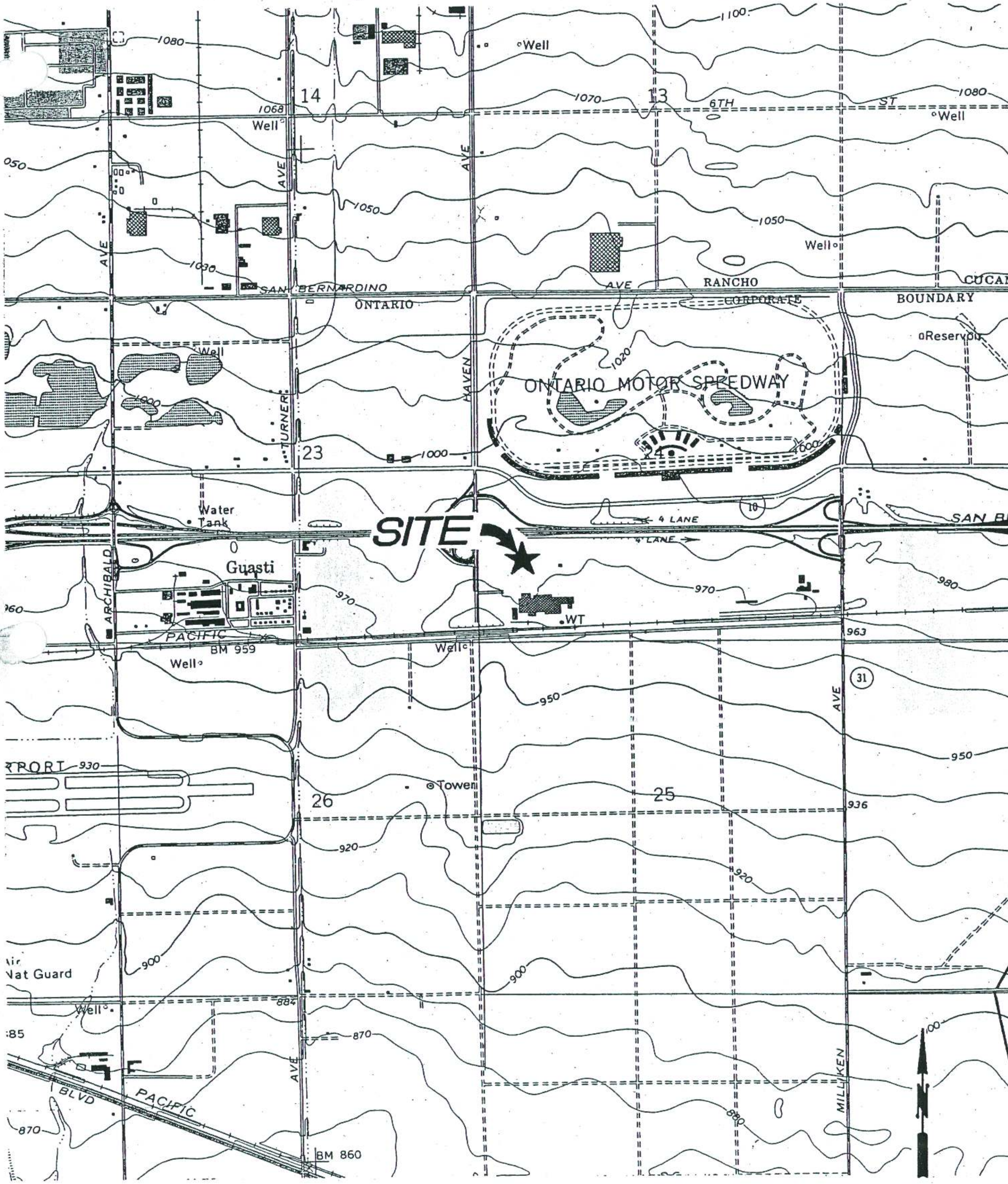
PLATES



RCE: U.S.G.S. 7.5' topographic series, Guasti, California quadrangle dated 1966, photorevised 1981.

0 FEET 2,000

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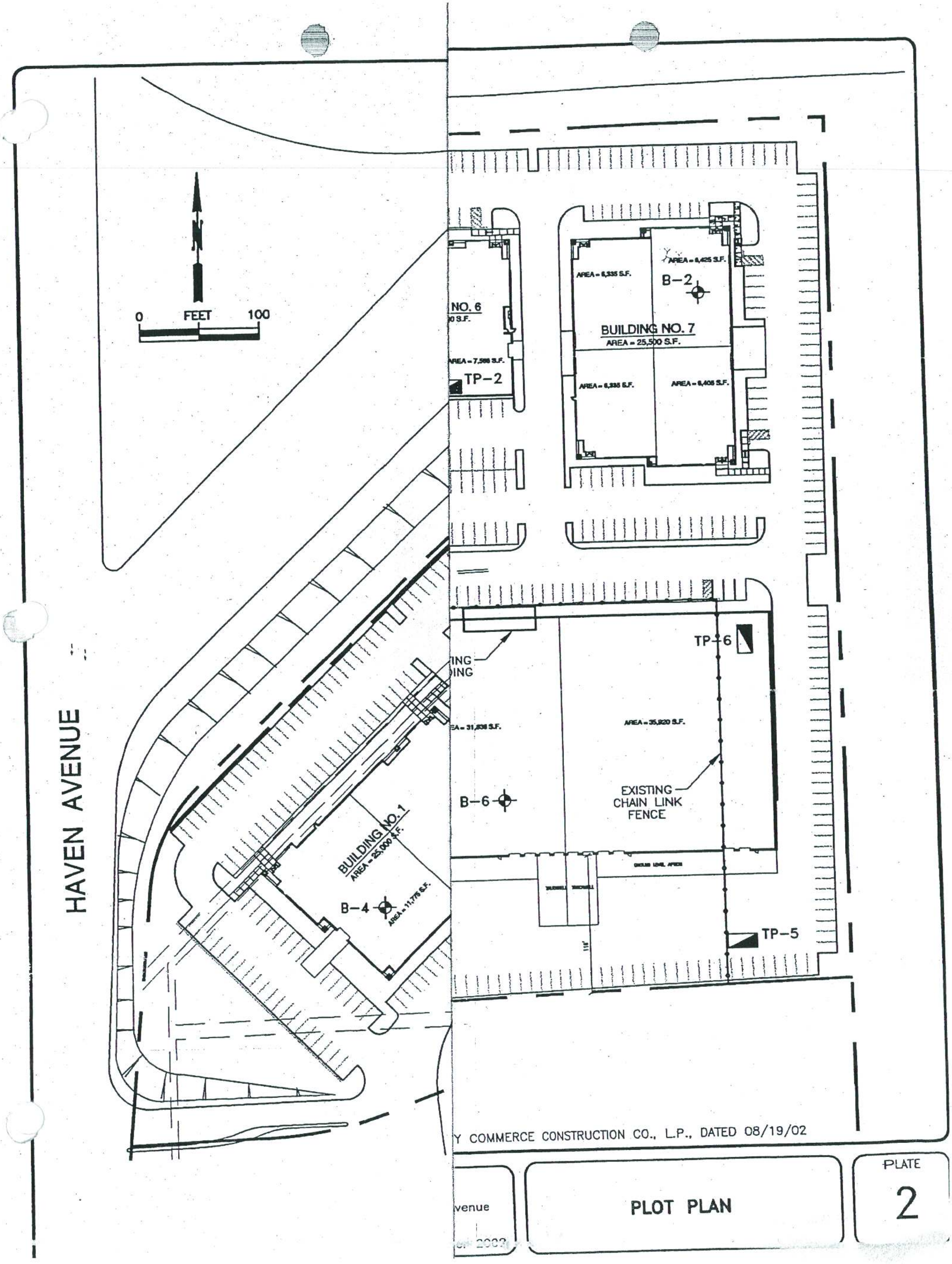
COMMERCIAL DEVELOPMENT SITE
Vicinity of Interstate 10 and Haven Avenue
Ontario, California

Project 21861 October 2002

SITE LOCATION MAP

PLATE 1



0 FEET 100
HAVEN AVENUE
NO. 6
TP-2
BUILDING NO. 7
AREA = 25,500 S.F.
B-2
TP-6
B-6
EXISTING CHAIN LINK FENCE
TP-5
BUILDING NO. 1
AREA = 25,000 S.F.
B-4
Y COMMERCE CONSTRUCTION CO., L.P., DATED 08/19/02
venue
PLOT PLAN
PLATE
2

APPENDIX A

FIELD EXPLORATION

APPENDIX A
EXPLORATORY BORINGS

A total of 9 hollow-stem auger borings were advanced for the project site by A&R Drilling of Signal Hill, California. Eight test pits were also excavated at the project site. The borings were drilled to depths ranging from 11½ to 51½ feet below existing grade. The test pits were excavated to depths approximately 6 to 8 feet below existing grade. Plate 2 presents the location of the borings and test pits.

The Logs of Borings are presented as Plates A-2 through A-10. The logs of test pits are presented as Plates A-11 through A-18. An explanation to the logs is presented as Plate A-1. The Logs of Borings and test pits describe the earth materials encountered, samples obtained and show field and laboratory tests performed. The logs also show the location, boring number, drilling date and the name of the logger and drilling subcontractor. The borings were logged by a geologist using the Unified Soil Classification System. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Bulk, relatively undisturbed and disturbed samples of representative earth materials were obtained from the borings at maximum intervals of about 5 feet.

All borings were backfilled using the soil from cuttings and tamped when the drilling was completed. Asphalt patch was used to cap the borings located within asphalt paved portions of the site. All test pits were backfilled with the excavated spoils.

A California sampler was used to obtain relatively undisturbed samples of the soil encountered. This sampler consists of a 3-inch O.D., 2.4-inch I.D. split barrel shaft that is pushed or driven a total of 12-inches into the soil at the bottom of the boring. The soil was retained in six 1-inch brass rings for laboratory testing. An additional 2-inches of soil from each drive remained in the cutting shoe and was usually discarded after visually classifying the soil. The sampler was driven using a 140-pound hammer falling 30-inches. The total number of hammer blows required to drive the sampler the 12-inches is termed blow count and is recorded on the Logs of Borings.

Samples were also obtained using a Standard Penetration Sampler (SPT). This sampler consists of a 2-inch O.D., 1-inch I.D. split barrel shaft that is advanced into the soils at the bottom of the drill hole a total of 18-inches. The sampler was driven using a 140-pound hammer falling 30-inches. The total number of hammer blows required to drive the sampler the final 12-inches is termed the blow count (N) and is recorded on the Logs of Borings. The procedures we employed in the field are generally consistent with those described in ASTM Standard Test Method D-1586-84.

Bulk samples of the surface soils were obtained directly from the auger blades and the test pits.

Date Drilled:
Drilled By:
Drilling Method:
Logged By:

Water Depth:
Date Measured:
Reference Elevation:
Datum:

Elevation (feet) Depth	Sample	Sample No.	Blow Count (Blows/ft.)	Graphic Log	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
		1	6			108	10	DS, SE
		2	12					GS
5								
	(1)	(2)	(3)	(4)	(5)	(6)	(6)	(7)
10								

NOTES ON FIELD INVESTIGATION

1. SAMPLE – Graphical representation of sample type as shown below.
   - Split Spoon – Standard Penetration Test Sample (SPT)
   - Drive Sample – California Sample (Cal)
   - Bulk Sample – Obtained by collecting cuttings in a plastic bag
   - Tube Sample – Shelby/Pitcher Tube Sample
2. SAMPLE NO. – Sample Number
3. BLOWS/FT – Number of blows required to advance sampler 1 foot (unless a lesser distance is specified). Samplers in general were driven into the soil at the bottom of the hole with a standard (140 lb) hammer dropping a standard 30 inches. Drive samples collected in bucket auger borings may be obtained by dropping non-standard weight from variable heights. When a SPT sampler is used the blow count conforms to ASTM D-1586.

   SCR/RQD – Sample Core Recovery (SCR) in percent (%) and Rock Quality Designation (RQD) in percent (%). RQD is defined as the percentage of core in each run which the spacing between natural fractures is greater than 4 inches. Mechanical breaks of the core are not considered.
4. GRAPHIC LOG – Standard symbols for soil and rock types, as shown on plate A-1b.
5. GEOTECHNICAL DESCRIPTION

   Soil – Soil classifications are based on the Unified Soil Classification System per ASTM D-2487, and designations include consistency, moisture, color and other modifiers. Field descriptions have been modified to reflect results of laboratory analyses where deemed appropriate.

   Rock – Rock classifications generally include a rock type, color, moisture, mineral constituents, degree of weathering, alteration, and the mechanical properties of the rock. Fabric, lineations, bedding spacing, foliations, and degree of cementation are also presented where appropriate.

   Description of soil origin or rock formation is placed in brackets at the beginning of the description where applicable, for example, Residual Soil.
6. DRY DENSITY, MOISTURE CONTENT: As estimated by laboratory or field testing.
7. ADDITIONAL TESTS – (Indicates sample tested for properties other than the above):
   - MAX – Maximum Dry Density
   - GS – Grain Size Distribution
   - SE – Sand Equivalent
   - EI – Expansion Index
   - CHEM – Sulfate and Chloride Content, pH, Resistivity
   - PM – Permeability
   - UU – Unconsolidated Undrained Triaxial
   - SG – Specific Gravity
   - HA – Hydrometer Analysis
   - AL – Atterberg Limits
   - RV – R-Value
   - CN – Consolidation
   - CU – Consolidation Undrained Triaxial
   - CD – Consolidated Drained Triaxial
   - PP – Pocket Penetrometer
   - WA – Wash Analysis
   - DS – Direct Shear
   - CP – Collapse Potential
   - UC – Unconfined Compression
   - T – Torvane
8. ATTITUDES – Orientation of rock discontinuity observed in bucket auger boring or rock core, expressed in strike/dip and dip angle, respectively, preceeded by a one-letter symbol denoting nature of discontinuity as shown below.

   B: Bedding Plane J: Jointing C: Contact F: Fault S: Shear

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EXPLANATION OF LOGS

PLATE A-1a

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

PRIMARY DIVISIONS			GROUP SYMBOLS	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIALS IS LARGER THAN #200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN #4 SIEVE	CLEAN GRAVELS (LESS THAN) 5% FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVEL WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN #4 SIEVE	CLEAN SANDS (LESS THAN) 5% FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN HALF OF MATERIALS IS SMALLER THAN #200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT IS LESS THAN 50	ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT IS GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
	HIGHLY ORGANIC SOILS		PT	PEAT, MUCK AND OTHER HIGHLY ORGANIC SOILS
TYPICAL FORMATIONAL MATERIALS	SANDSTONES		SS	
	SILTSTONES		SH	
	CLAYSTONES		CS	
	LIMESTONES		LS	
	SHALE		SL	

CONSISTENCY CRITERIA BASED ON FIELD TESTS

RELATIVE DENSITY - COARSE - GRAIN SOIL

RELATIVE DENSITY	SPT * (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	0 - 15
Loose	4 - 10	15 - 35
Medium Dense	10 - 30	35 - 65
Dense	30 - 50	65 - 85
Very Dense	>50	85 - 100

CONSISTENCY-FINE-GRAIN SOIL

		TORVANE	POCKET ** PENETROMETER
CONSISTENCY	SPT (# blows/ft)	UNDRAINED SHEAR STRENGTH (tsf)	UNCONFINED COMPRESSIVE STRENGTH (tsf)
Very Soft	<2	<0.13	<0.25
Soft	2 - 4	0.13 - 0.25	0.25 - 0.5
Medium Stiff	4 - 8	0.25 - 0.5	0.5 - 1.0
Stiff	8 - 15	0.5 - 1.0	1.0 - 2.0
Very Stiff	15 - 30	1.0 - 2.0	2.0 - 4.0
Hard	>30	>2.0	>4.0

* NUMBER OF BLOWS OF 140 POUND HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1 3/8 INCH I.D.) SPLIT BARREL SAMPLER (ASTM-1586 STANDARD PENETRATION TEST)

** UNCONFINED COMPRESSIVE STRENGTH IN TONS/SQ.FT. READ FROM POCKET PENETROMETER

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

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EXPLANATION OF LOGS

PLATE A-1b

Date Drilled:	9/26/02	Water Depth:	>31.5 feet
Excavated By:	A&R Drilling	Date Measured:	9/26/02
Drilling Method:	Hollow Stem Auger 8 inch	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
		1			**Silty Sand (SM):** light brown, dry, loose, fine to medium sand			RV
		2	11			104	2.1	
5		3	14			108	0.5	CP
10		4	23		-- medium dense, less silt			
15		5	31		**Sand (SP):** brown, dry, dense, with fine to coarse gravel			
					Sandy Silt (ML): brown, moist, stiff, some calcium carbonate mottling, trace clay			
20		6	16					
					Silty Sand (SM): brown, moist, fine to medium sand			
25		7	12		-- medium dense, trace gravel, trace clay, calcium carbonate mottling			

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LOG OF BORING B-1

PLATE **A-2a**

Drafted by:_____ Reviewed by:______ Explanation To Logs On Plate A-1

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
30		8	46		-- moist, dense, fine to coarse sand, fine to coarse gravel up to 2 inches diameter			
					Total depth of boring: 31.5 feet No groundwater encountered Boring backfilled with soil cuttings and patched with asphalt			

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LOG OF BORING B-1

PLATE

A-2b

Drafted by:_____Reviewed by:______Explanation To Logs On Plate A-1

Date Drilled:	9/26/02	Water Depth:	>21.5 feet
Excavated By:	A&R Drilling	Date Measured:	9/26/02
Drilling Method:	Hollow Stem Auger 8 inch	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
		A			**Silty Sand (SM):** light brown, dry, loose, fine to coarse			
		1	8			100	2.8	WA
5		2	17		-- medium dense, trace fine gravel	104	1.9	
10		3	24		-- increase in coarse sand content			
15		4	23		-- increase in sand and gravel content			
20		5	18		-- fine to coarse gravel			

Total depth of boring: 21.5 feet
No groundwater encountered
Boring backfilled with soil cuttings

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Commerce Construction
Ontario, California

LOG OF BORING B-2

PLATE

A-3

Drafted by:_____ Reviewed by:______ Explanation To Logs On Plate A-1

Date Drilled:	9/26/02	Water Depth:	>21.5 feet
Excavated By:	A&R Drilling	Date Measured:	9/26/02
Drilling Method:	Hollow Stem Auger 8 inch	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
					Silty Sand (SM): light brown, dry, loose, fine to medium sand, some gravel up to 3/4 inches diameter			
		1	18					CHEM
5		2	20		-- medium dense, fine gravel	106	1.7	
10		3	24		-- poor sample recovery, decrease in gravel content			
					Sand (SP): light brown, moist, medium dense, fine to medium sand, trace silt			
15		4	28					
20		5	17		-- increase in silt and fine sand content			
					Total depth of boring: 21.5 feet No groundwater encountered Boring backfilled with soil cuttings			

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PROJECT NO. 21861

Commercial Development Site
Commerce Construction
Ontario, California

LOG OF BORING B-3

PLATE A-4

Drafted by:_____ Reviewed by:______ Explanation To Logs On Plate A-1

Date Drilled:	9/26/02	Water Depth:	>31.5 feet
Excavated By:	A&R Drilling	Date Measured:	9/26/02
Drilling Method:	Hollow Stem Auger 8 inch	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
					ASPHALTIC CONCRETE (AC): approximately 3 inches thick			
		1	40		**Silty Sand (SM):** light brown, dry, medium dense, fine to coarse sand, trace fine gravel, sample at 0.5 feet disturbed	86	2.2	
		2						
5		3	18		-- increase in moisture content	105	1.8	
					-- increase in gravel content			
10		4	51		-- dense, with gravel up to 2 inches diameter			
15		5	38		-- decrease in silt content, fine to coarse gravel			
20		6	19		-- medium dense, decrease in gravel content			
					Sandy Silt (ML): light brown, moist, fine sand, trace clay			
					Silty Sand (SM): brown, moist, medium dense, fine to coarse sand, trace fine gravel			
25		7	20					

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Ontario, California

LOG OF BORING B-4

PLATE A-5a

Drafted by:_____Reviewed by:______Explanation To Logs On Plate A-1

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
30		8	26					
					Total depth of boring: 31.5 feet No groundwater encountered Boring backfilled with soil cuttings and patched with asphalt			

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Commercial Development Site
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Ontario, California

LOG OF BORING B-4

PLATE

A-5b

Drafted by:_____Reviewed by:______Explanation To Logs On Plate A-1

Date Drilled:	9/26/02	Water Depth:	>51.5 feet
Excavated By:	A&R Drilling	Date Measured:	9/26/02
Drilling Method:	Hollow Stem Auger 8 inch	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
					Silty Sand (SM): brown, moist, fine to medium sand			
		1						
5		2	12		-- loose, sample disturbed			
					Sand (SP): gray brown, moist, fine to coarse sand, fine to coarse gravel			
10		3	78		-- very dense			
15		4	23		-- medium dense			
					Silty Sand (SM): brown, moist, medium dense, fine to medium sand, trace clay			
20		5	12					
25		6	14					WA

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LOG OF BORING B-5

PLATE **A-6a**

Drafted by:_____Reviewed by:______Explanation To Logs On Plate A-1

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
30		7	20					
35		8	22		-- coarse sand and fine gravel at tip of sampler -- trace fine gravel			
					Sandy Silt (ML): brown, moist, medium dense, fine sand, trace clay			
40		9	12					WA
45		10	18					WA
50		11	18		-- silty sand interbeds common, trace fine gravel			
					Total depth of boring: 51.5 feet No groundwater encountered Boring backfilled with soil cuttings and patched with asphalt			

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LOG OF BORING B-5

PLATE

A-6b

Drafted by:_____ Reviewed by:______ Explanation To Logs On Plate A-1

Date Drilled:	9/26/02	Water Depth:	>31.5 feet
Excavated By:	A&R Drilling	Date Measured:	9/26/02
Drilling Method:	Hollow Stem Auger 8 inch	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
		1	24		**ASPHALTIC CONCRETE (AC):** approximately 3 inches thick	107	5.1	
					Silty Sand (SM): brown, moist, medium dense, fine to medium sand, with gravel			
		2						
5		3	15		-- decrease in gravel content	100	10.6	
					-- trace clay			
					-- increase in silt content			
10		4	14					
15		5	24					
					Sand (SP): gray brown, moist, medium dense, fine to coarse, some fine to coarse gravel			
					-- increase in gravel content			
					Silty Sand (SM): motled brown, moist, fine to medium sand, trace clay			
20		6	23					
25		7	14		-- trace gravel and calcium carbonate stringers			

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LOG OF BORING B-6

PLATE A-7a

Drafted by:_____Reviewed by:______Explanation To Logs On Plate A-1

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
30		8	17					
					Total depth of boring: 31.5 feet No groundwater encountered Boring backfilled with soil cuttings			

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LOG OF BORING B-6

PLATE
A-7b

Drafted by:_____Reviewed by:______Explanation To Logs On Plate A-1

Date Drilled:	9/26/02	Water Depth:	>11.5 feet
Excavated By:	A&R Drilling	Date Measured:	9/26/02
Drilling Method:	Hollow Stem Auger 8 inch	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
					ASPHALTIC CONCRETE (AC): approximately 3 inches thick			
		1	38		**Silty Sand (SM):** light brown, dry, medium dense, fine to coarse sand, trace gravel			
					-- increase in silt content			
5		2	15		-- decrease in silt content	106	4.0	
10		3	22		**Sand (SP):** gray brown, moist, medium dense, fine to coarse sand, fine to coarse gravel up to 1 inch diameter			
					Total depth of boring: 11.5 feet No groundwater encountered Boring backfilled with soil cuttings and patched with asphalt			

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LOG OF BORING B-7

PLATE A-8

Drafted by:_____Reviewed by:______Explanation To Logs On Plate A-1

Date Drilled:	9/26/02	Water Depth:	>11.5 feet
Excavated By:	A&R Drilling	Date Measured:	9/26/02
Drilling Method:	Hollow Stem Auger 8 inch	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
		1	24		**ASPHALTIC CONCRETE (AC):** approximately 3 inches thick			
					Silty Sand (SM): brown, moist, medium dense, fine to medium sand, trace fine gravel			
					Sand (SP): light brown, dry, fine to medium sand			
5		2	12		-- loose, sample disturbed	81	2.4	
10		3	19		-- light gray, medium dense, increase in sand content, fine gravel			
					Total depth of boring: 11.5 feet No groundwater encountered Boring backfilled with soil cuttings and patched with asphalt			

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LOG OF BORING B-8

PLATE **A-9**

Drafted by:_____ Reviewed by:______ Explanation To Logs On Plate A-1

Date Drilled:	9/26/02	Water Depth:	>11.5 feet
Excavated By:	A&R Drilling	Date Measured:	9/26/02
Drilling Method:	Hollow Stem Auger 8 inch	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
		1	32		**ASPHALTIC CONCRETE (AC):** approximately 3 inches thick			
					Silty Sand (SM): light brown, dry, medium dense, fine to medium sand, trace gravel, roots			
					-- increase in gravel up to 1 inch diameter			
5		2	26			107	3.0	CP
					-- increase in gravel content			
10		3	36					

Total depth of boring: 11.5 feet
No groundwater encountered
Boring backfilled with soil cuttings and patched with asphalt

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LOG OF BORING B-9

PLATE
A-10

Drafted by:_____Reviewed by:______Explanation To Logs On Plate A-1

Date Trenched:	9/26/02	Water Depth:	>7 feet
Trenched By:	Staib Backhoe	Date Measured:	09/26/2002
Trenching Method:	24 inch Bucket	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
5					**NATIVE:** **Silty Sand (SM):** light brown, dry, fine to medium sand, some coarse sand, trace fine gravel -- at 2 feet: reddish brown, roots, firm, slightly cemented			
					Total depth of excavation: 7 feet No groundwater encountered Backfilled with excavated soils			

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LOG OF TEST PIT TP-1

PLATE A-11

Drafted by_____ Reviewed by_____

Explanation To Logs On Plate A-1

Date Trenched:	9/26/02	Water Depth:	>6.5 feet
Trenched By:	Staib Backhoe	Date Measured:	09/26/2002
Trenching Method:	24 inch Bucket	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
					NATIVE: **Silty Sand (SM):** light brown, dry, fine to medium sand, some coarse sand, trace fine to coarse gravel			
					-- at 2.5 feet: roots, firm			
5					-- at 5 feet: pinhole porosity, slightly cemented			
					Total depth of excavation: 6.5 feet No groundwater encountered Backfilled with excavated soils			

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LOG OF TEST PIT TP-2

PLATE A-12

Drafted by_____ Reviewed by_____

Explanation To Logs On Plate A-1

Date Trenched:	9/26/02	Water Depth:	>8 feet
Trenched By:	Staib Backhoe	Date Measured:	09/26/2002
Trenching Method:	24 inch Bucket	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
					ARTIFICIAL FILL (Af): **Silty Sand (SM):** light brown, dry, fine to medium sand, trace fine gravel, plastic debris -- at 2 feet: roots			
5								
					NATIVE: **Sand (SP):** light brown, slightly moist, fine to coarse sand, with fine to coarse gravel, trace cobbles up to 5 inches diameter			
					Total depth of excavation: 8 feet No groundwater encountered Backfilled with excavated soils			

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LOG OF TEST PIT TP-3

PLATE A-13

Drafted by_____Reviewed by_____

Explanation To Logs On Plate A-1

Date Trenched:	9/26/02	Water Depth:	>8.5 feet
Trenched By:	Staib Backhoe	Date Measured:	09/26/2002
Trenching Method:	24 inch Bucket	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
					NATIVE: **Silty Sand (SM):** light brown, dry, fine to medium sand -- at 2 feet: roots, firm, slightly cemented, trace pinhole porosity			
5					**Sand (SP):** light brown, slightly moist, fine to coarse sand, with fine to coarse gravel, trace cobbles up to 4 inches in diameter			
					Total depth of excavation: 8.5 feet No groundwater encountered Backfilled with excavated soils			

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LOG OF TEST PIT TP-3A

PLATE A-14

Drafted by_____Reviewed by_____

Explanation To Logs On Plate A-1

Date Trenched:	9/26/0	Water Depth:	>7.5 feet
Trenched By:	Staib Backhoe	Date Measured:	09/26/2002
Trenching Method:	24 inch Bucket	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
					ARTIFICIAL FILL (Af): **Silty Sand (SM):** dark brown, dry, fine to coarse sand			
					NATIVE: **Silty Sand (SM):** light brown, dry, fine to medium sand -- at 1.5 feet: slightly cemented, trace pinhole porosity			
5								
					Sand (SP): light brown, dry, fine to coarse sand, with fine to coarse gravel, trace cobbles up to 4 inches diameter			
					Total depth of excavation: 7.5 feet No groundwater encountered Backfilled with excavated soils			

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LOG OF TEST PIT TP-4

PLATE A-15

Drafted by_____ Reviewed by_____

Explanation To Logs On Plate A-1

Date Trenched:	9/26/0	Water Depth:	>6 feet
Trenched By:	Staib Backhoe	Date Measured:	09/26/2002
Trenching Method:	24 inch Bucket	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
5					**NATIVE:** **Silty Sand (SM):** light brown, dry, fine to medium sand, some coarse sand, with fine gravel, rootlets throughout -- at 1.5 feet: slightly cemented, firm, slight increase in moisture content and silt content -- at 4 feet: increase in moisture and silt content			
					Total depth of excavation: 6 feet No groundwater encountered Backfilled with excavated soils			

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LOG OF TEST PIT TP-5

PLATE A-16

Drafted by_____ Reviewed by_____

Explanation To Logs On Plate A-1

Date Trenched:	9/26/0	Water Depth	>7 feet
Trenched By:	Staib Backhoe	Date Measured:	09/26/2002
Trenching Method:	24 inch Bucket	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
5					**NATIVE:** **Silty Sand (SM):** light brown, dry, fine to medium sand, some coarse sand, with fine gravel -- at 1.5 feet: firm, slight increase in moisture content, roots, increase in silt content			
					Total depth of excavation: 7 feet No groundwater encountered Backfilled with excavated soils			

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LOG OF TEST PIT TP-6

PLATE A-17

Drafted by_____ Reviewed by_____

Explanation To Logs On Plate A-1

Date Trenched:	9/26/02	Water Depth:	>6 feet
Trenched By:	Staib Backhoe	Date Measured:	09/26/2002
Trenching Method:	24 inch Bucket	Elevation:	N/A
Logged By:	J. Norum	Reference Datum:	N/A

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
5					**NATIVE:** **Silty Sand (SM):** light brown, dry, fine to medium sand, some coarse sand, with fine gravel, roots in upper 2 feet -- at 2 feet: reddish brown, slightly cemented, firm			
					Total depth of excavation: 6 feet No groundwater encountered Backfilled with excavated soils			

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LOG OF TEST PIT TP-7

PLATE A-18

Drafted by_____ Reviewed by_____

Explanation To Logs On Plate A-1

APPENDIX B

LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

Laboratory tests were performed on representative relatively undisturbed and bulk soil samples to estimate engineering characteristics of the various earth materials encountered.

LABORATORY MOISTURE AND UNIT WEIGHT DETERMINATIONS

In situ moisture content and dry unit weight tests were performed on soil samples collected from the borings in accordance with ASTM D2216-92 and D2937-94, respectively. The results are presented on the Logs of Borings and are summarized in Table B-1, Moisture and Unit Weight.

WASH SIEVE

The percent passing #200 sieve of four selected soil samples were performed by wash sieving in accordance with ASTM Standard Test Method D 1140-92. The test results are summarized in Table B-2, Wash Sieve Test Results.

COLLAPSE POTENTIAL

Collapse potential testing was performed on selected relatively undisturbed samples in general accordance with ASTM Standard Test Method D-5333. The test results are presented on Plates B-2 and B-3, Collapse Potential Test.

R-VALUE TESTING

R-value testing was performed on one sample of the near-surface soils encountered at the site. The test was performed in general accordance with Caltrans Standard Test Method 301. The test result is presented in Table B-3, R-Value Test Results.

CORROSIVITY TESTS

A series of chemical tests were performed on one selected sample collected from a depth approximately 2.5 feet below the existing grade to estimate pH, resistivity, and sulfate and chloride contents. The test results may be used by a qualified corrosion engineer to evaluate the general corrosion potential with respect to the construction materials. The results of the tests are presented in Table B-4, Corrosion Test Results.

Table B-1
Moisture and Unit Weight

Boring	Depth (ft)	Moisture Content (%)	Dry Unit Weight (pcf)
B-1	2.5	2.1	104
B-1	5.0	0.5	108
B-2	2.5	2.8	100
B-2	5.0	1.9	104
B-3	5.0	1.7	106
B-4	0.5	2.2	---*
B-4	5.0	1.8	105
B-6	0.5	5.1	107
B-6	5.0	10.6	100
B-7	5.0	4.0	106
B-8	5.0	2.4	---*
B-9	5.0	3.0	107

*Sample disturbed.

Table B-2
Wash Sieve Test Results

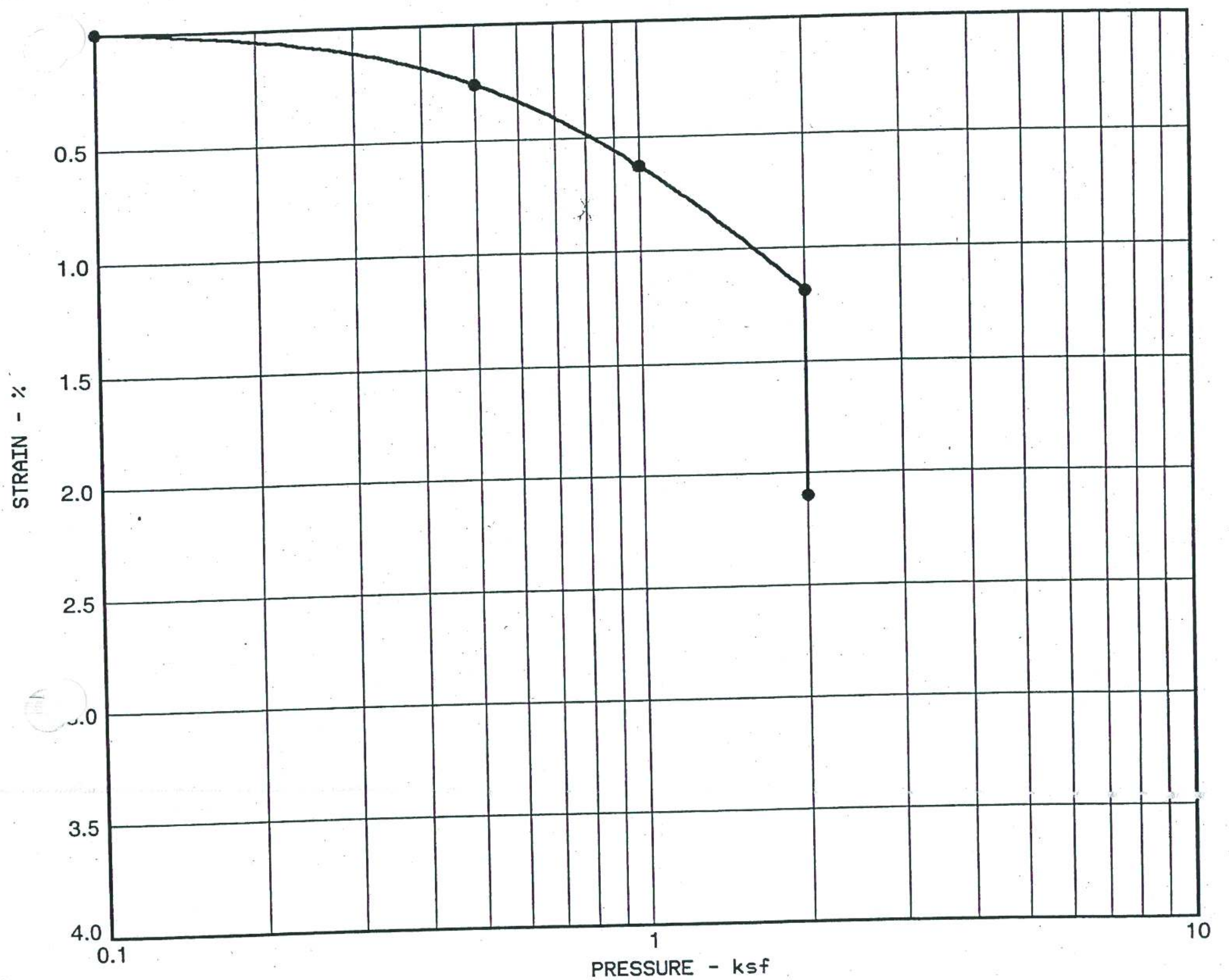
Boring	Depth (ft)	Percent Passing No. 200 (0.075 mm)
B-2	2.5	18
B-5	25.0	48
B-5	40.0	51
B-5	45.0	58

Table B-3
R-Value Test Results

Boring	Depth (ft)	R-Value
B-1	0 – 2.5	76

Table B-4
Corrosion Test Results

Boring	Depth (ft)	pH	Sulfate (ppm)	Chloride (ppm)	Resistivity (Ω-cm)
B-3	2.5	8.7	7	121	18,000



Sample	B-1
Depth (ft)	6.0
Description	Silty Sand
Classification	SM
Collapse Potential (%)	0.9
Severity of Collapse	Slight

Moisture Content Before = 0.5 %
Moisture Content After = 20.1 %
Dry Density Before = 108 pcf

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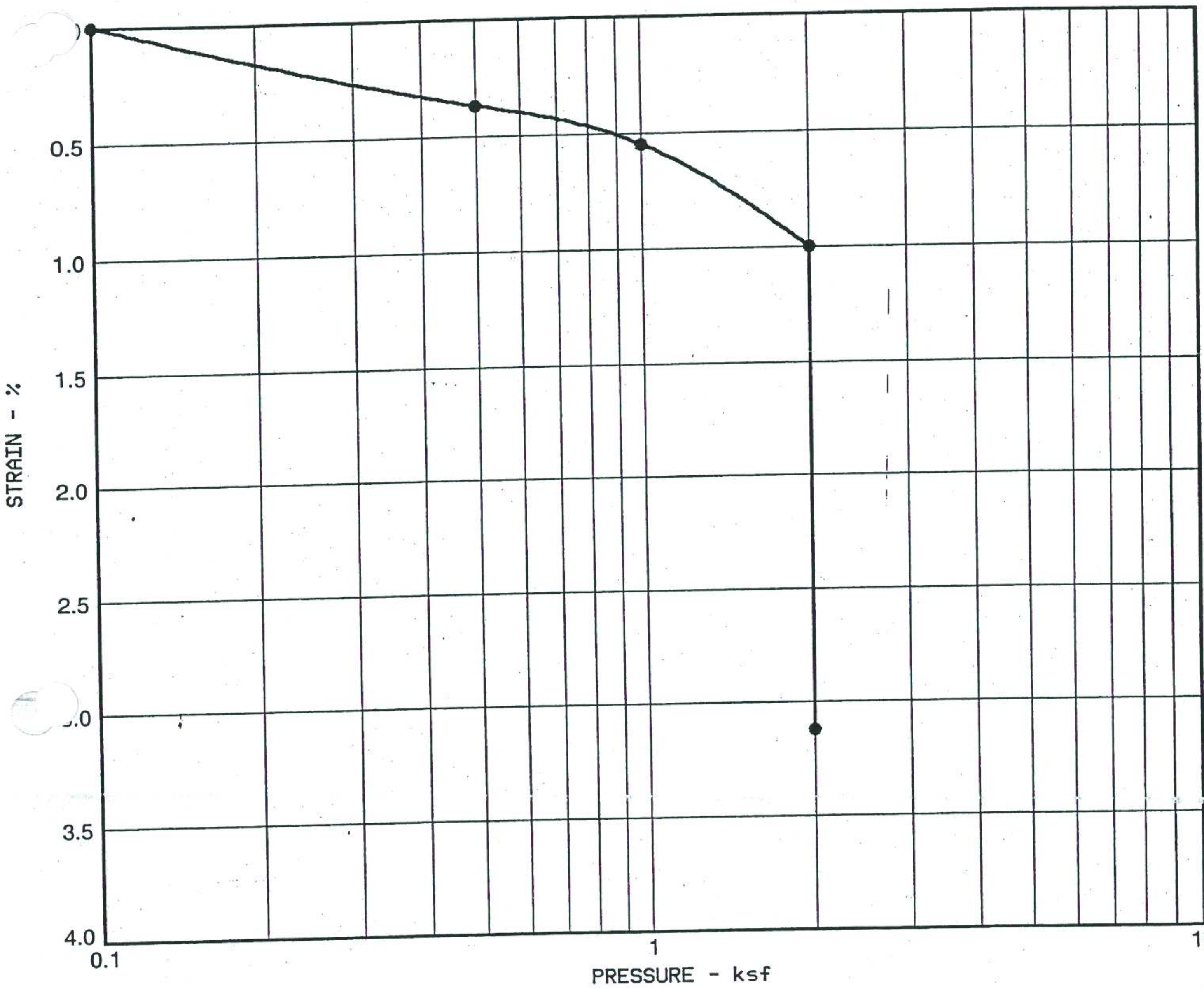
COLLAPSE POTENTIAL TEST

PLATE

B-1

PROJECT NO. 21861



Sample	**B-9**
Depth (ft)	**6.0**
Description	**Silty Sand**
Classification	**SM**
Collapse Potential (%)	**2.1**
Severity of Collapse	**Moderate**

Moisture Content Before = 3.0 %
Moisture Content After = 20.3 %
Dry Density Before = 107 pcf

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COLLAPSE POTENTIAL TEST

PLATE **B-2**

PROJECT NO. **21861**

APPENDIX C

ASFE INSERT

Important Information About Your

Geotechnical Engineering Report

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

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

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

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

APPENDIX D

APPLICATION FOR AUTHORIZATION TO USE

APPLICATION FOR AUTHORIZATION TO USE

Geotechnical Investigation
Commercial Development
Vicinity of Interstate 10 and Haven Avenue
Ontario, California
File Number: 21861
Report Date: October 15, 2002

KLEINFELDER, INC.
1370 Valley Vista Drive, Suite 150
Diamond Bar, California 91765
(909) 396-0335

To whom it may concern:

Applicant understands and agrees that the Geotechnical Investigation (Report) for the subject site is a copyrighted document, that Kleinfelder, Inc. is the copyright owner and that unauthorized use or copying of the Report for the site is strictly prohibited without the express written permission of Kleinfelder, Inc. Applicant understands that Kleinfelder, Inc. may withhold such permission at its sole discretion, or grant permission upon such terms and conditions, as it deems acceptable.

Applicant agrees to accept the contractual terms and conditions between Kleinfelder, Inc. and Commerce Construction Company LP originally negotiated for preparation of this Report. Use of this Report without permission releases Kleinfelder, Inc. from any liability that may arise from use of this report.

To be Completed by Applicant

______________________ By: ______________________
(company name) *(Print Name)*

______________________ ______________________
(address) *(Signature)*

______________________ Title: ______________________
(city, state, zip)

__________ __________ Date: ______________________
(telephone) *(FAX)*

Approval of Original Client

By: ______________________ Date: ______________________
(Print Name)

____
(Signature)

For Kleinfelder, Inc.'s use only

_______ approved for re-use with additional fee of $__________

_______ approved for re-use with applicant's agreement to following conditions: Applicant agrees to above terms and understands that findings discussed in report were based on available information and site conditions as noted at time of ESA.

_______ disapproved, report needs to be updated

By: ______________________ Date: ______________________
(Kleinfelder, Inc. Project Manager)