# Appendix D:
# Geotechnical Investigation Report

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A Report Prepared for:

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**GEOTECHNICAL INVESTIGATION REPORT**
**WINDSOR MILL II PROPERTY**
**WINDSOR, CALIFORNIA**

**Kleinfelder Job No: 43125**

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**APPENDIX A**

Preliminary Corrosivity Test Data

**APPENDIX B**

Important Information About Your Geotechnical Engineering Report


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# GEOTECHNICAL INVESTIGATION REPORT
# WINDSOR MILL II PROPERTY
# WINDSOR, CALIFORNIA

## 1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for proposed residential development at the Windsor Mill II property in Windsor, California. The objective of this report is to provide our client, Cal-waii Molding Company, Inc., Work Force Housing Associates, LLC, and their design team with findings, conclusions and recommendations regarding the geotechnical engineering aspects of project development.

### 1.1 LOCATION AND SITE DESCRIPTION

The Windsor Mill property is located at the south terminus of Bell Road, in Windsor, California. The site is comprised of three adjoined parcels as follows:

- 8703 Bell Road, Assessor Parcel Numbers APN 066-170-011 (Parcel 1).
- 8820 Larson Way, APN 164-010-020 (Parcel 2).
- 8777 Bell Road, APN 066-170-013 (Parcel 3).

For the purposes of this report, the aforementioned property will be collectively referred to as the "site." The location of the site is shown on Plate 1, Site Plan and Vicinity Map.

### 1.2 PROJECT DESCRIPTION

Conceptual development plans were not available during our investigation or preparation of this report. We assume development will generally consist of one- or two-story single-family residences with appurtenant streets, underground utilities and driveways. Since the site is nearly level, new site preparation and grading will most likely consist of removal of the old structures;


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foundations; utilities, isolated pockets of existing, non-engineered fill and porous surface soils; and the placement and compaction of relatively shallow new fills of approximately 2 to 3 feet thick to provide relatively level building pads. Additional details of the planned project are not known at this time. If the actual project differs significantly from our present understanding, we should be contacted to review and, if necessary, revise our recommendations.

## 1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation is to evaluate the site for proposed single-family residential development with respect to site geologic and geotechnical characteristics and to provide opinions and recommendations concerning the following:

- Regional and site-specific geologic and seismic conditions.
- Geotechnical feasibility of residential development.
- Site preparation and grading, including fill quality and compaction specification.
- Foundation types, including geotechnical engineering design parameters and estimates of settlement.
- Support for concrete slab-on-grade floors.
- Surface and subsurface drainage improvements.
- Preliminary pavement design.

Our scope of services, as outlined in our January 30, 2004 proposal, consisted of field exploration, laboratory testing, engineering analyses and preparation of this report.

## 1.4 PREVIOUS INVESTIGATION

Kleinfelder conducted a geotechnical investigation of the adjacent Windsor Mill and a parcel east of Bell Road documented in a report titled "Geotechnical Investigation Report, Windsor Mill Property, Windsor, California," dated December 12, 2003. Kleinfelder has also conducted preliminary environmental site assessments (ESAs) for this property and the Windsor Mill parcel. The ESA for the Windsor Mill II site is documented in a report titled "Phase I Environmental Site Assessment, Windsor Mill, Phase II Site, Windsor California," dated May 12, 2004. Each parcel has its own environmental concerns and each ESA should be reviewed for


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associated environmental findings, conclusion and recommendations, which are not included in this report.

## 1.5 AUTHORIZATION

This investigation was authorized by Kleinfelder's Professional Services Agreement dated January 30, 2004, executed by Mr. Gale Paddock of Kleinfelder, Inc. and a financial representative of Cal-waii Molding Company, Inc.


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# 2.0 FIELD INVESTIGATION

## 2.1 SUBSURFACE EXPLORATION

Our field exploration was performed on April 16 and April 20, 2004 and consisted of excavating 10 backhoe test pits (labeled TP-1 through TP-10) and drilling six (6) soil borings (labeled K-1 through K-6) at the approximate locations shown on the Site Plan, Plate 2. The borings were located by Kleinfelder personnel by pacing and taping from existing landmarks. As such, the locations shown on Plate 1 should be considered accurate only to the degree implied by the method used. The exploration program was conducted under the supervision of our Geotechnical Engineer and Certified Engineering Geologist.

The test pits were excavated with a Takeuchi TB-145 track-mounted excavator to depths up to approximately 7 feet below the existing ground surface. Our geologist observed the excavations and logged the conditions encountered. Upon completion, the test pits were loosely backfilled and track-walked.

The borings were drilled and sampled using a Mobile B-53 truck mounted drill rig equipped with 6-inch-diameter, solid-flight augers. The borings were drilled to depths ranging from 13.5 to 15.5 feet below existing grades. Materials encountered in each test boring were visually classified in the field and a log was recorded. Visual classifications were made in accordance with the Unified Soil Classification System presented on the Boring Log Legend, Plate 2. The Logs of Exploration Borings K-1 through K-6 showing soil classifications and sample depths are presented on Plates 3 through 8. Log of Test Pits TP-1 through TP-10 are presented on Plates 9 and 10.

Relatively undisturbed samples were obtained from each major soil type encountered in each boring by driving a 2.5-inch (inside diameter) Modified California sampler containing thin brass liners at selected depths in each boring. The sampler was advanced by dropping a 140-pound


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hammer 30 inches per blow. The number of blows required to drive the last 12 inches of an 18-inch-drive and recorded as resistance in blows per foot and were converted to equivalent SPT blow counts (conversions for sampler type and hammer weight were applied to the raw blow counts) on the boring logs. When the sampler was withdrawn from each boring, the brass liners containing the samples were removed, examined for logging, labeled and sealed to preserve their natural moisture content for possible laboratory testing.

## 2.2 LABORATORY TESTING

Selected soil samples were tested in our laboratory to evaluate pertinent engineering and physical properties. The testing program consisted of moisture density determination, particle size analysis, an UBC Expansion Index test, Atterberg Limits, and unconfined compression strength tests. The results of our laboratory tests are presented graphically on Plates 11 through 13 or at the corresponding depth on the log of borings. One sample of near-surface soil from Borings K-3 was submitted to Environmental Technical Services (ETS) of Petaluma, California for preliminary corrosivity analysis. The results of their tests are presented in Appendix A.


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# 3.0 FIELD EXPLORATION

## 3.1 REGIONAL GEOLOGY

Geologic literature and maps pertinent to the site vicinity were reviewed as part of this study. The site is located in the northern Santa Rosa plain within the Coast Range Geomorphic Province of Northern California. This province is generally characterized by northwest-trending mountain ranges and intervening valleys, which are a reflection of the dominant northwest structural trend of the bedrock in the region. The basement rock in the northern portion of this province is presumed to consist of the Franciscan Complex, a diverse group of igneous, sedimentary and metamorphic rocks of Upper Jurassic to Cretaceous age (140 to 65 million years old). The Franciscan Complex is part of a northwest trending belt of material, immediately adjacent to the eastern edge of the San Andreas fault system, which is located approximately 19 miles southwest of the site. In the site vicinity, the Franciscan Complex rocks have been unconformably overlain by Tertiary age continental and marine sedimentary and volcanic rocks. These Tertiary age rocks have been locally overlain by younger Quaternary alluvial and colluvial deposits.

## 3.2 SITE GEOLOGY

The local geology in the site vicinity has been mapped by Huffman and Armstrong (1980, California Division of Mines and Geology, Special Report 120). Huffman and Armstrong (1980) indicate that the site is underlain by sand, gravel, mudstone and interbedded tuff of Plio-Pleistocene age Glen Ellen Formation. The area immediately east of the site is mapped as underlain by undifferentiated alluvium consisting of sand, gravel, silt and clay.

## 3.3 FAULTING AND SEISMICITY

The site is located within the seismically active North Bay/North Coast region of California, and is subject to seismically induced ground shaking from nearby and distant faults. Several faults


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have been mapped in the general site vicinity. The San Andreas fault zone, located approximately 19 miles southwest of the site, is the boundary between two tectonic plates, the Pacific Plate (west of the fault) and the North American Plate (east of the fault). At this boundary, the Pacific Plate is moving north relative to the North American Plate. In the North Coast region of California, this movement is distributed across a complex system of predominantly strike-slip, right-lateral, parallel, and sub-parallel faults that include the San Andreas, Healdsburg/Rodgers Creek and the Maacama, among others.

The site is not located within a state-mandated Earthquake Fault Zone as defined by the California Geological Survey (CGS, formerly the Division of Mines and Geology, CMDG), in accordance with the Alquist-Priolo Earthquake Fault Zone Act of 1972. The nearest Earthquake Fault Zone is that of the Healdsburg/Rodgers Creek fault, which is located approximately 1.8 miles northeast of the site.

Moderate to major earthquakes generated on the Healdsburg/Rodgers Creek fault can be expected to cause strong ground shaking at the site. In addition, strong ground shaking can be expected from moderate to major earthquakes generated on other faults in the region such as the Maacama fault, located approximately 6.2 miles northeast of the site. A number of large earthquakes have occurred within this region in the historic past. Some of the significant nearby events include two 1969 Santa Rosa earthquakes (M5.6, 5.7), the 2000 Napa earthquake (M5.0), and the 1906 San Francisco earthquake (M8+). Future seismic events in this region can be expected to produce strong seismic ground shaking at this site. Time, location and magnitude of earthquakes are not accurately predictable with existing technology.

## 3.4 FLOODING

According to the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map panel 0603750545 B dated April 2, 1991, the majority of this site is located in an area outside of the 500-year flood plain.


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## 3.5 SURFACE CONDITIONS

Proposed residential development will be constructed on the approximately 5.3-acre irregularly shaped site bordered by: Johnson Street and existing private residences on the north and northeast; Bell Road and Windsor Mill facility parking area on the east; and the Northwestern Pacific Railroad easement along the entire southwest edge of the site. The parcel located at 8703 Bell Road, referred to herein as Parcel 1, is located at the central and western portions of the site and encompasses approximately 2.55-acres. No structures currently exist at this parcel; however, the site once contained wood-framed, metal-sided warehouses and a smaller carport/storage building which we understand have been removed within the last 4 years. An approximately 350-foot long asphalt concrete driveway extends from Johnson Street and cross the parcel (Plate 1). Vegetation on the parcel consists of a few trees and annual grasses and weeds.

8820 Larson Way, referred to herein as Parcel 2, is located at the northeast portion of the site and encompasses approximately 0.6-acres. The parcel contains a wood-framed residence, carport, several small outbuildings and an approximately 120-foot long gravel driveway. Vegetation on the parcel consists of a few small tress, shrubs, and annual grasses and weeds.

8777 Bell Road, referred to herein as Parcel 3, is located at the eastern and southern portion of the site and encompasses approximately 2.2-acres. The site is occupied by the Cash-and-Carry warehouse which consists of a reinforced masonry structure and carport. An asphalt concrete parking lot is present north of the warehouse, and gravel, concrete and asphalt concrete parking and delivery areas are present south of the warehouse. An undeveloped area exists at the southern tip of the parcel. Access to the parcel is provided off the south terminus of Bell Street. Vegetation at the site consists of numerous shrubs and trees along Bell Street and in the undeveloped area. Annual grasses and weeds are present along the western boundary of the parcel and in the undeveloped area.

A topographic survey of the site provided to us for our investigation, as well as observations made in the field, indicate the entire site is relatively flat, with elevations ranging from 116 feet


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(National Vertical Geodetic Datum, NGVD) near the middle and northern areas of the site to 107 feet in the undeveloped area at the southern tip of the site.

## 3.6 SUBSURFACE CONDITIONS

The majority of the site is blanketed with a veneer of fill, typically ranging in thickness from 1 to 4 feet thick. Beneath and beyond the fill areas, the subsurface conditions typically consist of thin residual/alluvial soils ranging in thickness from 1.5 to 5.5 feet, underlain by sand, clayey sand, silty clayey sand, sandy clay and silty sandy clay of the Glen Ellen Formation. For the purposes of this report, the Glen Ellen Bedrock (designated QTge on the boring and test pit logs) has been classified as a soil on our boring and test pit logs, since this formation is weakly consolidated to the depths explored and generally has similar engineering properties to those of medium stiff to hard clays and silts, and/or medium dense to dense sands and gravels.

The soil and groundwater conditions described in the following sections have been simplified for ease of report presentation. A more detailed description of the conditions encountered will be found on the log of exploration borings and test pit logs presented on Plates 4 through 11.

### 3.6.1 Parcel 1

In this area of the site, the surface material encountered typically consist of approximately 1.5 to 3.5 feet of highly variable fill. The fill material is dry to moist, highly variable in consistency (soft to hard/loose to dense), is comprised of a mixture of sand, silt, clay and gravel and contains minor amounts of organics and assorted debris such as plastic pipe, concrete and brick fragments. Three areas of fill (Qaf) were identified on this parcel. Test pit TP-2 encountered up to 5 feet of fill adjacent to the NWPR right-of-way that consisted of wet, soft, sandy, gravelly clay that contained construction debris with a maximum dimension of up to 2 feet, which may correspond to an infilled drainage channel. In the other two areas, up to 2 to 3 feet of loose sandy gravel fill was encountered. The approximate limits of the fill are shown on Plate 1. The fill material typically overlies a silty sand and sandy silt layer, encountered to depths of up to 3.5 feet below the ground surface. The layer is typically dry to moist, soft to stiff/medium dense and porous. This silty sand and sandy silt layer is typically underlain by a clay layer with variable


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amounts of sand encountered to depths of up to 5.5 feet. The layer is typically moist, stiff to very stiff and moderately plastic. This clay layer is underlain by deposits consisting of clay and sand with varied amounts of silt and gravel (weathered Glen Ellen Formation materials). In soil description terms, these deposits are typically moist, very stiff to hard/dense to very dense and were encountered to the maximum depth explored on this area of the site.

### 3.6.2 Parcel 2

In this area of the site, the surface soils encountered typically consist of brown sandy silt. This deposit is moist, very stiff, porous, and was encountered to a depth of 1.25 feet in Boring K-3. This surface silt overlies sandy clay layers, encountered to depths of up to 2.75 feet below the ground surface. The layer is moist, very stiff and moderately plastic. This clay layer is underlain by Glen Ellen Formation deposits comparable to those encountered in Parcel 1 and were encountered to the maximum depth explored on this area of the site.

### 3.6.3 Parcel 3

The north parking area and the south delivery and parking areas of the Cash-and-Carry warehouse are overlain by approximately 12 inches of aggregate base. Additionally, the aggregate base is overlain by 2 inches of asphalt concrete pavement in the north parking area and the southwest delivery and parking areas. The aggregate base and pavement section overlies clayey sand fill and sandy clay to depths up to 6.5 feet. The layer is moist, stiff to hard/medium dense to dense, slightly porous and moderately plastic. This layer is underlain by Glen Ellen Formation deposits comparable to those encountered in Parcel 1 and were encountered to the maximum depth explored on this area of the site.

### 3.6.4 Groundwater

Groundwater was observed in boring K-2 at a depth of approximately 13 feet below the existing ground surface. As reported in our ESA for this site, previous environmental studies have reported that the depth to groundwater ranges between 9 and 14 feet.


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# 4.0 DISCUSSION AND CONCLUSIONS

## 4.1 GENERAL

Based on the results of our geotechnical investigation, we judge that the proposed residential project is feasible from a geotechnical engineering standpoint. The main geotechnical engineering considerations affecting design and construction of the proposed development are the presence of undocumented fills, porous and compressible surficial alluvial soils, moderately plastic and expansive near-surface soils, and the potential for strong seismic ground shaking from future earthquakes in the region.

## 4.2 UNDOCUMENTED FILL AND ALLUVIAL DEPOSITS

As previously discussed in Section 3.6, many areas of the site are underlain by variable thicknesses of undocumented fill and alluvial deposits. These fills and alluvial soils are porous and can undergo significant strength loss and settlement under load, especially when saturated. In order to mitigate the adverse affects associated with these surficial soils, it will be necessary to remove and replace these soils as engineered fill and/or replace these soils with imported, non-expansive select fill to support pavements, structures and associated improvements. The fill materials contain deleterious materials locally. The fill materials will need to be removed and segregated with unsuitable material disposed of off-site. The recent alluvial soils will need to be reworked by excavating, moisture conditioning and compacting the soils to prepare the site for development. The depth of reworking is anticipated to typically be 3 to 4 feet but may be locally deeper. The near-surface soils consist of sands, silts and moderate plasticity clays. The moderate plasticity clays are moderately expansive and are not the most desirable soils on which to found structures. Accordingly, the near surface sands and silts may be selectively excavated and stockpiled for use as select fill beneath the proposed structures. Recommendations for site grading are provided in Section 5.0 of this report. Special considerations of expansive soils are discussed below.


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## 4.3 EXPANSIVE SOIL

Based on the results of our field exploration and previous experience with similar soils, we judge the clayey near-surface soils located across the site are moderately expansive. Expansive soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from sun, wind, rainfall, landscape irrigation, utility leakage, roof drainage, drought or other factors and will most likely result in unacceptable settlement or heave of structures supported in the near-surface clays in their present condition, if not mitigated or designed for. Results of laboratory testing indicate that the near surface plastic clay has a Plasticity Index of 23 (boring K-5 at a depth of 4-feet) and falls within the upper range of the very low expansion potential (boring K-2 at a depth of 2-feet). At a minimum the clayey soils should be properly moisture conditioned and compacted to reduce the shrink swell behavior coupled with the appropriate foundation design. These soils can also be replaced with low expansion select fill and can be placed beneath the structures, concrete slabs-on-grade and pavements. Recommendations for suitable support of foundations, concrete slabs-on-grade and pavements, along with related grading and site preparation, are presented in Section 5.1 of this report.

Foundations, slabs and pavements placed directly on, or sufficiently close to, the expansive on-site soils in their present condition will undergo detrimental and erratic movement. Our experience indicates that where the surface or near-surface soils are expansive, lightly-loaded foundations and slabs will experience unacceptable distress (heave and cracking) as the expansive materials shrink and swell with changes in moisture content. Therefore, we judge that the expansive soils are not suitable for support of foundations or concrete floor slabs unless mitigating measures to reduce shrink and/or swell movement are considered in project design and construction.

Within building and adjacent exterior slab areas, the adverse effects of expansive soils can be most effectively reduced by initially moisture conditioning them to a high moisture content (to cause expansion to occur before construction), and then blanketing the expansive soils with at least 36 inches (depth of significant seasonal moisture change in expansive soils) of imported, select fill having a low expansive potential (i.e., Plasticity Index of 15 or less). Structures can


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then be supported on shallow spread footings bottomed in the properly compacted select fill. Concrete floor slabs, where used, can also gain suitable support on the select fill.

Alternatively, the on-site expansive soils can be lime-treated to reduce the effects of the expansive soils and eliminate the need for removal and replacement of the expansive soils. Similar to imported select fill, the upper 36 inches of soil (as measured from planned soil subgrade) might be treated with high-calcium lime, preferably in relatively thin lifts. For planning purposes, we suggest the addition of 5% (volume by dry weight) high-calcium lime be used. If a lime-treatment alternative is chosen, the actual percentage of lime to be used should be verified by additional laboratory testing of the on-site expansive soils.

Drilled, cast-in-place, concrete pier-and-grade-beam foundations can also provide suitable support below the depth of significant seasonal moisture change in the expansive soils. Use of pier-and-grade-beam foundations can eliminate the need for select fill placement for the majority of the building areas. However, select fill will still be required under floor slabs (including garages) and exterior slabs. If differential heave or uplift on the order of 1 inch across slab joints is acceptable, select fill beneath floor slabs and exterior slabs can be reduced to 18 inches as measured from planned soil subgrade; however, this range of movement and related cracking is not usually acceptable at interior living spaces. Concrete patios, floor slabs, and sidewalks will be subject to expansive soil movements on the order of 1 to 2 inches during seasonal moisture changes unless select fill grading and/or special construction procedures are used. An effective method of reducing differential movement (caused by shrinking and swelling of expansive clays) to an acceptable level within these areas would be to:

- Initially moisture condition the clay to a high moisture content.
- Blanket the clay with at least 18 inches of imported, select fill.
- Provide a continuous perimeter moisture barrier to reduce the risk of seasonal moisture changes at the unprotected (exposed to the climate) concrete flatwork edges.

In asphalt-paved areas, the most practical method of lessening the detrimental effects of expansive soils is to thoroughly moisture condition them to close all shrinkage cracks for their full depth and to maintain them at a high moisture content until covered with the planned


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pavement section. Seasonal moisture changes at unprotected pavement edges can most effectively be controlled by installing a continuous perimeter moisture cutoff barrier. Streets that are bordered by planter areas may also need a moisture cutoff barrier, unless some risk of edge cracking in the asphalt concrete surfacing is acceptable. The moisture barrier should consist of a 4-inch-wide trench extending a minimum of 42 inches below finished grade or 6 inches below the street subgrade whichever is deeper. The trench should be filled with lean concrete consisting of ½-inch maximum aggregate, 2-sack slurry mix with a 6- to 8-inch slump. Asphalt pavement section recommendations are presented in Section 5.5 of this report.

During the dry season, shrinkage cracks in expansive soils may be as deep as four to five feet below existing grades. Therefore, prolonged watering and flooding, and the possible use of wetting agents, may be necessary to close shrinkage cracks for their full depth. Alternatively, the grading contractor may elect to over-excavate the dry, expansive soils to facilitate moisture conditioning them. Since the exact depth of required moisture conditioning will not be known prior to site grading, we suggest that the contract documents contain provisions for either alternative at the contractor's option.

## 4.4 SITE SEISMIC CHARACTERIZATION

The site and Sonoma County in general are subject to strong ground shaking during future earthquakes in the region. At a minimum, it will be necessary to design and construct the project in accordance with the earthquake-resistant provisions of the locally adopted Uniform Building Code (UBC). This section contains seismic design parameters that have been established in accordance with provisions of the 1998 California Building Code (CBC), which is based on the 1997 Uniform Building Code (UBC). Field and laboratory test data indicate that the site and proximity can be assigned a soil profile type $S_C$ based on average soil properties in the top 100 feet and Table 16A-J of the CBC. $S_C$ is defined as a profile consisting of very dense soil or rock, with a shear wave velocity between 1,200 and 2,400 feet per second, a Standard Penetration Test, N (blows/foot), greater than 50 and an undrained shear strength greater than 2000 pounds per square foot, which is characteristic of the Glen Ellen Formation Bedrock beneath the site.


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According to Figure 16-2 of the 1997 UBC, the site is within Seismic Zone 4; therefore, a Seismic Zone Factor, Z, of 0.4 should be used. Sheet D-14 of the Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada (ICBO, 1998) indicates that the site is located approximately 4.0 kilometers west/southwest of the Healdsburg/Rodgers Creek fault, which is classified as a Seismic Source Type A. Using the above information, the near-source factors $N_a$ and $N_v$ are 1.30 and 1.73, respectively, based on interpolation between values shown in Tables 16-S and 16-T of the 1997 UBC. The 1997 UBC seismic coefficients $C_a$ and $C_v$ are both used to determine the total design lateral force or shear at the base of a building or structure. The seismic coefficients $C_a$ and $C_v$ can be obtained from Tables 16-Q and 16-R of 1997 UBC, respectively, based on the soil profile type, Seismic Zone Factor and near-source factors presented herein. Thus, based on the tables and procedures provided in the 1997 UBC, the following relationships apply: $C_a = 0.40N_a$ and $C_v = 0.56\ N_v$. These site parameters for UBC seismic design are summarized in the following table.

| SEISMIC SITE PARAMETERS | | | | | |
|---|---|---|---|---|---|
| Zone | Soil Profile | $N_a$ | $N_v$ | $C_a$ | $C_v$ |
| 0.40 | $S_C$ | 1.30 | 1.73 | 0.52 | 0.97 |

## 4.5 LIQUEFACTION POTENTIAL

During severe vibration from earthquakes, liquefaction can occur in loose, cohesionless silt and sand where groundwater is shallow. Sowers and others (1995) map the site vicinity as having moderate to high liquefaction susceptibility. Data from the borings indicate that the site is predominantly underlain by stiff clays and weathered Glen Ellen bedrock deposits. Therefore, we consider the potential for liquefaction to occur at this site is considered to be low. However, it may be possible that there are some isolated, thin zones of loose cohesionless silt or sand within the near-surface and possible within the bedrock deposits at the site that were not detected in our borings or are at depths deeper than our borings, that could liquefy under very strong ground motion. However, we judge that the surface confinement provided by the recommended site preparation, the recommended foundations, and the silt/clay content of most of the natural


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surface and subsurface soils will reduce the potential for surface effects from liquefaction at the site to negligible.

## 4.6 DESIGN CONSIDERATIONS

### 4.6.1 Groundwater

Groundwater was encountered in only one of the six boring drilled during our investigation (boring K-2 at a depth of 13 feet). However, previous studies conducted at the site by others indicate that groundwater levels can vary between depths of 9 and 14 feet. Our experience in the project vicinity indicates that temporary perched groundwater is not uncommon within the upper five feet below the ground surface during and after periods of prolonged rainfall. Local groundwater levels can fluctuate depending on factors such as seasonal rainfall, groundwater withdrawal and construction activities on this or adjacent properties. In general, winter to early summer construction can experience extra earthwork costs related to the presence of groundwater or seepage, depending on the magnitude of prior seasonal rainfall.

Of particular note, although not encountered during this investigation, our ESA indicated that groundwater beneath the Cash and Carry store and to the west has been impacted from past site uses. Refer to our May 2004 ESA report for discussion on the limits and type of chemical impact.

### 4.6.2 Structure Foundations

Due to the presence of moderately expansive surficial soils, we judge that the structures can be founded on the following foundation systems:

- Post-tensioned Slab-On-Grade.
- Pier-and-Grade Beam Foundations.
- Spread Footings underlain by non-expansive select fill.

Each foundation system requires special subgrade preparation and grading requirements to reduce the affects of the underlying expansive soils. Both grading and structural requirements in conjunction with project performance criteria should be considered when selecting a foundations


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system. Recommendations for site grading are provided in Section 5.1 and recommendations for use in foundation design are provided in Section 5.3.

Provided the site is graded, and foundations are designed and constructed in accordance with the recommendations presented herein, we estimate that maximum post-construction settlement resulting from anticipated foundation loads will be less than approximately ½-inch. We judge that post-construction differential settlement along or between adjacent foundation elements will be less than about ¼-inch. Loose fill or disturbed soil that is not removed from footing excavations prior to placing concrete may result in increased settlement.

### 4.6.3 Preliminary Corrosivity Results

One sample of the near-surface soils (boring K-3 at 2.5-foot depth) was sent to Environmental Technical Services in Petaluma for preliminary corrosivity testing. The test results are presented in Appendix A. It should be noted that Kleinfelder does not practice corrosion engineering. As such, the corrosivity test results are presented for the preliminary use of our client and their design team. Our review of comments contained in the preliminary test results indicates that the near surface alluvial soils from Boring K-3 have favorable resistivity and acceptable chloride and sulfate concentrations. Sulfate concentration is low and should not have a direct effect on concrete, cement, grout, or mortar. Chloride concentrations are very low so not to negatively impact steel reinforcement. The soil was found to be moderately acidic and is corrosive to steel. A corrosion specialist should be contacted to review these results and determine if additional testing is warranted.

## 4.7 CONSTRUCTION CONSIDERATIONS

### 4.7.1 Grading

Grading operations during or shortly following the wet season, or in areas where the soils materials are saturated, will likely require provisions for drying of these materials prior to achieving suitable compaction. If the project necessitates fill placement and compaction in wet conditions, we can provide alternatives to conventional drying of the fill materials. Conversely, additional moisture will likely be required during the dry months as previously discussed. Water


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trucks should be available in sufficient number to provide water to achieve the specified moisture conditioning during compaction.

### 4.7.2 Temporary Slopes

The design and maintenance of temporary slopes is generally considered to be the responsibility of the contractor. The contractor should be familiar with applicable local, state, and federal regulations, including the current OSHA Excavation and Trench Safety Standards. In no case should slope height, inclination, and excavation depths exceed those specified in local, state, or federal safety regulations. Specifically, the contractor should be familiar with the current Cal/OSHA regulations as described in California Code of Regulations, Title 8.

The soil conditions, as encountered in our investigation, vary across the site. The majority of the near surface soils are soft to medium stiff clays and loose to medium dense granular soils that may be considered to be Type "B" under OSHA regulations. The Contractor should verify soil conditions throughout the proposed limits of construction.

The Contractor's "responsible person," as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. The contractor's responsible person should establish a minimum lateral distance from the crest of the slope for all vehicles, equipment, and spoil piles. Likewise, the contractor's responsible person should establish protective measures for exposed slope faces. We recommend that the contractor, or his specialty subcontractor, design temporary construction slopes to conform to OSHA's "Guidelines for Excavations and Temporary Sloping." The temporary slope inclination should be determined by the contractor or responsible subcontractor based on the soil conditions exposed at the time of construction. If an excavation, including trenches, is extended to a depth of more than twenty (20) feet, it will be necessary to have the side slopes designed by a Civil Engineer registered in the State of California.


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### 4.7.3 Temporary Shoring

In areas where temporary construction slopes will not be feasible, proper temporary shoring should be provided during construction to reduce potential damage to the existing street pavements, adjacent buried utilities, and other facilities. The design, installation and maintenance of temporary shoring are generally considered to be the contractor's responsibility.

We anticipate that shoring will likely consist of conventional hydraulic shoring systems and/or braced or cantilevered sheet-piles. The shoring should be adequately rigid to minimize deformation of the adjacent ground surface. Care must also be exercised to avoid impacting adjacent existing utilities and/or foundations.

The design excavation depth should include an additional depth below the pipe invert to account for bedding material and any anticipated over-excavation requirements. The contractor or specialty subcontractor should be responsible for design of the temporary shoring in accordance with all applicable regulatory requirements. Bracing should be removed from the trench as the backfilling proceeds. Shoring should be removed as trench zone backfilling proceeds and site conditions allow.

The upper 10 feet of shoring should also be designed for additional surcharge loads caused by construction traffic, equipment, spoil piles, and/or storage loads. To minimize excessive surcharging of the shoring, we recommend that heavy construction equipment vehicles and soil stockpiles be kept at least 10 feet from the top of the excavation. Where this is not possible, shoring should be designed to resist surcharging. In addition, the shoring should be designed with sufficient rigidity to minimize movement of adjacent utilities or pavements.

### 4.7.4 Groundwater Control

Groundwater was generally, depending on the time or season, encountered at depths below the proposed foundations but possible within typical depth of utility installations. Groundwater may be encountered within the upper 5-10 feet. It should be the contractor's responsibility to monitor the groundwater elevation and control groundwater during construction.


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# 5.0 RECOMMENDATIONS

## 5.1 SITE PREPARATION AND GRADING

Conceptual grading plans were not available at the time this report was prepared. We anticipate that required grading will consist mainly of minor cuts and fills of approximately 1 to 4 feet in maximum vertical height to provide relatively level building pads and pavement areas having positive surface drainage. Finished construction areas should generally be sloped and drainage gradients maintained to carry surface water off the site. Ponding of water or concentrated seepage should not be allowed under structures, or adjacent to foundation systems. Grades adjacent to foundations should be sloped a minimum of 2 percent away from the foundation for a minimum distance of 5 feet. Final grading plans should be reviewed by our firm for conformance to our geotechnical design recommendations prior to construction bidding. Transition lots, consisting of cuts and fills, should be over-excavated to create fill lots.

### 5.1.1 Clearing and Site Preparation

Following demolition and removal of existing structures and their foundations, the site should be stripped of vegetation and organic or construction debris before grading commences. We anticipate that the stripping operation will require the removal of 2 to 3 inches of topsoil in most areas. Deeper stripping or grubbing will be required where concentrations or pockets of organic-laden soil, or old fill and/or structures are encountered. Cleared construction and vegetative debris should be removed from the site and properly disposed. The stripped, organic-rich material may be stockpiled and used for future landscaping purposes; however, this material should not be used as engineered fill. Undocumented fills should be removed for their entire depth. Low plasticity alluvial soils and/or fill materials free of organics or other deleterious substances could possibly be segregated and stockpiled for potential re-use as select fill, subject to observation by Kleinfelder. Additional laboratory testing should be conducted to confirm the material is suitable for use as select fill.


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It is possible that other underground facilities such as abandoned pipelines, underground storage tanks, leach fields, septic tanks, or water wells may exist on the site. Where encountered or present, these items should be plugged and abandoned according to the regulations set forth by the County of Sonoma Public Health Department and/or City of Windsor Building Department. Excavations for the removal of abandoned underground facilities should be cleaned of loose materials and widened as necessary to permit compaction equipment access. The excavations should be subsequently backfilled with properly compacted soil as specified in the following sections of this report.

### 5.1.2 Subgrade Preparation

Within proposed structure and related concrete slab areas, weak, compressible and expansive surface soils should be removed to a depth of 36 inches (as measured from planned soil subgrade) and, depending on the selected structure foundation system, reworked or replaced with compacted imported, select fill having a low expansion potential (if conventional spread footing foundations are to be used). Excavation limits should extend at least 5 and 3 feet beyond building and adjacent exterior slab edges, respectively. The expansive soils exposed by stripping and required excavation should be scarified to a depth of at least 6 inches, moisture conditioned to at least 4 percent above optimum moisture content, and re-compacted to at least 88 percent relative compaction[1]. Following the dry summer months, the moisture conditioning effort should be increased to close all shrinkage cracks to their full depth in addition to providing a moisture content at least 4 percent above optimum.

To allow for possible variation in the actual excavation quantities required, we recommend that the contract documents provide for "add" unit prices for excavation of existing expansive or otherwise unsuitable surface soils. In addition, we recommend that an "add" unit price be provided for imported select or on-site compacted fill.

[1] Optimum moisture content is the water content (percentage by dry weight) corresponding to the maximum dry density. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil, as determined by the ASTM D1557 test procedure.


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### 5.1.3 Structural Fill

On-site soils that are free of organic matter and do not contain rocks over 4 inches in diameter will generally be satisfactory for reuse as general fill. However, on-site expansive soils (typically boring log classification symbols CH and CL with plasticity indices greater than 15 percent) should not be used as select fill.

Imported select fill should be of low expansion potential and free of organic matter, and should conform, in general, to the following requirements:

| | |
|---|---|
| Plasticity Index | less than 15% |
| Liquid Limit | less than 40% |
| Percent Soil Passing #200 Sieve | between 15% and 60% |
| Maximum Aggregate Size | 4 inches |

### 5.1.4 Fill Placement

Fill should be spread in thin lifts; moisture conditioned to near optimum moisture content and compacted to the relative compaction specifications presented below. Fill (or cut) subgrade soils should be finished to present smooth, unyielding surfaces. Subgrade soils should be maintained at their moist or above optimum moisture contents and be free of shrinkage cracks, until covered by permanent construction. A summary of our compaction requirements is presented in the following Table 1.


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**TABLE 1**
**SUMMARY OF COMPACTION REQUIREMENTS**

| Area | Compaction Requirements |
|---|---|
| General and Select Engineered Fill | In lifts, a maximum of 8 inches loose thickness, compact to a minimum of 90 percent relative compaction at or within 2 percent of the optimum moisture content for select (non-expansive) fill. On-site expansive soil, if used outside of, or below, select fill zones, should be moisture conditioned to, and maintained at, 4 percent or more above optimum moisture content and compacted to between 88 and 92 percent relative compaction. |
| Trenches | In lifts, a maximum of 8 inches loose thickness, compact to at least 90 percent relative compaction at or within 2 percent of the optimum moisture content for select, non-expansive fill. On-site expansive soil, if used below select fill zones, should be moisture conditioned to, and maintained at, 4 percent or more above optimum moisture content and compacted to between 88 and 92 percent relative compaction. |
| Parking and Access Driveway | Compact the top 6 inches of subgrade soil to at least 95 percent relative compaction at or within 2 percent of optimum moisture content for select (non-expansive) fill. On-site expansive soil should be moisture conditioned to at least 4 percent above optimum moisture content, and compacted between 90 and 92 percent relative compaction. Soils should be maintained at their moist or above optimum moisture contents until covered by permanent construction. |
| Concrete Slab-On-Grade Floors | Compact the top 6 inches of select subgrade soil to at least 90 percent relative compaction at or within 2 percent of the optimum moisture content. |
| Exterior Flatwork | Compact the top 6 inches of select subgrade soils to at least 90 percent relative compaction at or within 2 percent of the optimum moisture content. |

5.1.5 Cut Slopes

Finished cut or fill slopes (if any) should be trimmed to expose a firm surface free of loose material, and should be no steeper than 2.5:1 (horizontal to vertical) and no higher than five feet where native materials are used or exposed, or 2:1 if select material is used in fill slope construction. Upon completion of grading, all denuded slopes should be planted with fast-growing, deep-rooted groundcover to reduce the risk of erosion.

Grading operations during or shortly following the wet season, or in areas where the soils are saturated, will likely require provisions for drying of the soil prior to achieving suitable compaction. If the project necessitates fill placement and compaction in wet conditions, we can


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provide alternatives to conventional drying of the soils. Conversely, additional moisture will likely be required during the dry months. Water trucks should be available in sufficient number to provide water to achieve the specified moisture conditioning during compaction.

In general, site preparation and grading operations should be observed by a representative of Kleinfelder. This will allow us to check whether unforeseen or detrimental materials are exposed by the construction equipment and to modify our recommendations, if necessary.

## 5.2 TEMPORARY EXCAVATION AND BACKFILL

Shallow excavations for footings and utility trenches can readily be made with either a backhoe or trencher. We expect the walls of trenches less than 5 feet deep to remain in a near-vertical configuration during utility construction, provided equipment or excavated spoil surcharges are not located near the top of the excavation. Where trenches extend deeper than 5 feet, the excavation can become unstable. In addition, our test borings indicate that the site is crossed by former drainage channels that contain relatively clean sand that have the potential to retain perched groundwater. Groundwater may also be encountered in shallow excavations for utilities depending on the time of year construction commences. As such, the contractor should be prepared to dewater the utility excavations by pumping from temporary sumps or by other similar methods, at the contractor's option. All trenches, regardless of depth, should be evaluated for stability prior to personnel entering them. Shoring or sloping of the deeper trench walls will be necessary to protect personnel and to provide stability. At a minimum, all trenches should conform to the current CAL-OSHA requirements for worker safety.

We recommend a minimum compaction of trench backfill as previously presented in Section 5.1.4, Table 1. Care should be taken to adequately compact utility trench backfill in all structure areas including pavements. Poor compaction will likely cause subsequent settlement of the trench, resulting in possible distress cracking to the overlying structure or pavement.


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## 5.3 FOUNDATIONS

Based on the field and laboratory data obtained during this investigation, building loads may be adequately supported by pier-and-grade-beam or post-tensioned slabs-on-grade foundations on properly prepared native soils, or alternatively, by relatively conventional spread footings on 36 inches of imported select fill or lime-treated native soils.

### 5.3.1 Drilled Pier Design Criteria

Drilled piers should be designed to gain support by skin friction in the firm near-surface soils using an allowable skin friction of 500 pounds per square foot (psf) per foot of depth for dead plus long-term live loads. This value can be increased by 50% for total loads, including wind or seismic forces. Skin friction in the upper three feet of the expansive surface soils should be neglected in the pier load capacity computations. Pullout capacity of piers can be assumed to be ½ the downward capacity.

Drilled piers should be at least 12 inches in diameter and should extend at least 8 feet below lowest adjacent grade, regardless of load. Pier spacing should be at least three pier diameters, center to center. In areas where fills are greater than one foot, the piers should be deepened an amount equivalent in excess of the one-foot. An air space (2-inch minimum) should be provided under grade beams connecting the drilled piers, or alternatively, the grade beams should be designed to withstand an expansive soil uplift pressure of 1000 psf. Pier depths and exposed soils in pier holes should be checked by Kleinfelder before reinforcing steel is installed and concrete is placed.

### 5.3.2 Lateral Resistance

Resistance to lateral loads can be obtained from passive soil pressure acting against the face of drilled piers using a value of 1000 psf (rectangular distribution). However, neglect passive pressure in the top three feet of expansive surface soils. In addition, passive pressure can be assumed to act on a width equal to 1½ times the pier diameter.


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The existence of localized loose sands (channel deposits) might cause significant local sloughing of pier holes. As such, some pier holes may need to be cased and should be anticipated and accounted for by the contractor.

### 5.3.3 Post-Tensioned Slabs-on-Grade

Structures can also be supported on post-tensioned slabs designed by a structural engineer experienced in post-tensioned slab design for expansive soil conditions. We recommend the structural engineer use the design parameters listed below. These parameters are based on our experience, and the procedures presented in the Uniform Building Code (currently adopted) and the Post Tensioning Institute's publication "Design and Construction of Post-Tensioned Slabs on Ground" (1996).

| **Parameter** | **Estimated Value** |
|---|---|
| Edge Moisture Variation Distance ($e_m$) **- Center Lift** | 4.0 feet |
| Edge Moisture Variation Distance ($e_m$) **- Edge Lift** | 5.0 feet |
| Differential Soil Movement ($y_m$) **- Center Lift** | 0.40 inches |
| Differential Soil Movement ($y_m$) **- Edge Lift** | 0.35 inches |

Post-tensioned slabs can be designed to impose dead plus code live load and total design load (including wind or seismic forces) bearing pressures of 2,000 and 3,000 pounds per square foot (psf), respectively. A modulus of subgrade reaction value of k = 100 pounds per cubic inch (pci) can also be used for design purposes. Where an aggregate base material is used as a capillary moisture break beneath the slabs, the k value can be increased to 130 and 140 pci for aggregate base thicknesses of 4 to 6 inches, respectively. Resistance to lateral forces is available from soil-to-concrete friction using a factor of 0.3 along the underside of the post-tensioned slab and the supporting native soil or imported granular materials. This value contains a factor of safety equal to approximately 1.5. Where the post-tensioned slabs are placed directly on plastic


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sheeting, as discussed below, then the frictional resistance along the slab undersides should be ignored.

The subgrade for a post-tensioned slab foundation system will also have to be removed to a depth of 36 inches and reworked in accordance with the recommendations presented in Section 5.1.4. Excavation limits should extend at least 5 and 3 feet beyond building and adjacent exterior slab edges, respectively. The subgrade materials should not be allowed to dry out prior to post-tensioned slab construction. If drying does occur, the subgrade should be re-moisture conditioned to wet of optimum, and to close shrinkage cracks for their full depth, before the under-slab membrane is installed.

### 5.3.4 Moisture Barriers

We recommend a minimum slab thickness of 10 inches. To lessen moisture propagation through slabs, at least 4 inches of under-slab rock should be provided to form a capillary moisture break; this rock should be graded so that 100 percent passes the one-inch sieve and no more than 5 percent passes the No. 4 sieve. Alternatively, an extra layer of plastic could be used in lieu of the recommended 4-inch rock layer, along with taping of all protrusions through the top layer of plastic and installing moisture cutoff collars (such as compacted native soil or lean concrete) where utility trenches cross under the foundations. It is common to cover the membrane with 2 inches of clean sand for protection during construction. The slab subgrade materials should not be allowed to dry out prior to post-tensioned slab construction. If drying does occur, the subgrade should be re-moisture conditioned to wet of optimum before under-slab rock or concrete is placed.

Subsurface moisture and moisture vapor naturally migrate upward through 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 moisture that could be introduced in the future (such as landscape irrigation, precipitation or leaking pipes) the current industry standard is to provide a vapor retardant membrane. This membrane typically consists of visquene or polyvinyl plastic sheeting at least-10 mil in thickness. It should be noted that


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although vapor barrier systems are currently the industry standard, this system may not be completely effective in preventing floor slab moisture problems. 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 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 can 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 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 concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) Manual.

It should be emphasized that Kleinfelder personnel are not moisture proofing experts for floors. We make no guarantee, nor provide any assurance, that use of the capillary break/vapor retardant system will reduce concrete slab-on-grade floor moisture penetration to any specific rate or level, particularly those required by floor covering manufacturers. The builder and designers should consider all available measures for slab moisture protection and should consult with the project architect for appropriate moisture barrier design.


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Because the post-tensioned mat slabs are much thicker than conventional slabs, the moisture dissipation will take longer than that of thin slabs. With a typical construction schedule, the moisture content of the slab prior to the placement of the flooring may exceed that specified. Therefore, measures should generally be taken to evaluate whether the moisture, prior to placement of the flooring, is below that specified by the flooring manufacturer. Actual test results should generally be obtained, checked for acceptability by the contractor and kept in the project construction files.

### 5.3.5 Spread Footing Design Criteria

Recommended spread footing bearing pressures, depth of embedment and footing widths are presented in Table 2 below. ***The recommended pressures and depths assume that footings will bear on engineered, select (low expansion) fill or on firm, native soils located below the engineered, select fill, if necessary.***

The allowable soil bearing pressures are net values. The weight of the foundation, and the backfill weight over the foundation, may be neglected when computing dead loads. Allowable soil bearing pressures may be increased by one-third for transient forces, such as wind or seismic loads.

**TABLE 2**

**SPREAD FOOTING DESIGN CRITERIA**

| Spread Footing Type | Allowable Soil Bearing Pressure (pounds per square foot)* | Minimum Embedment (inches)** | Minimum Width (inches) |
|---|---|---|---|
| Continuous Wall | | | |
| One-story | 2,000 | 12 | 12 |
| Two-story | 2,000 | 18 | 18 |
| Isolated Column | | | |
| One-story | 2,000 | 12 | 18 |
| Two-story | 2,000 | 18 | 18 |

*Dead plus live load

**Below lowest adjacent grade.


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Resistance to lateral forces may be computed using soil-to-concrete friction or passive earth pressure. A friction factor of 0.3 is recommended between the undersurface of concrete structures and the supporting soils. A passive earth pressure equivalent to that exerted by a fluid weighing 400 pounds per cubic foot (pcf) is recommended against the face of foundations for foundations elements founded in engineered select fill. If friction and passive pressure resistances are combined, the lesser value should be reduced by 50 percent. Also, neglect passive pressure in the top one (1) foot of soil unless foundations are confined by slabs or pavements.

Footing concrete should be placed neat against undisturbed soil, if possible. The soils exposed in footing excavations should not be allowed to dry before placing concrete. If shrinkage cracks appear in the footing excavation soils, these soils should be thoroughly moistened to close all cracks prior to concrete placement. We should observe all footing excavations prior to placement of concrete to check that the conditions exposed are as anticipated, or to modify our recommendations, if necessary.

## 5.4 CONCRETE SLABS-ON-GRADE

Concrete slabs-on-grade for this project may consist of interior garage floors, exterior driveways, walkways and possibly porches. As previously discussed, the near-surface soils at the site are moderately expansive and will be subject to shrink/swell cycles with fluctuations in moisture content. To minimize the potentially adverse affects of expansive soil shrink/swell movement on slabs, we recommend that imported select fill or lime-treated native soils be placed and compacted as previously described in Section 5.1.4 of this report.

Concrete slab-on-grade garage floors, where moisture propagation through the slab is of concern, should be supported on at least four inches of under-slab rock to provide a capillary moisture break; this rock should be graded so that 100 percent passes the one-inch sieve and no more than 5 percent passes the No. 4 sieve. If the subgrade materials dry out prior to slab-on-grade construction, the subgrade should be re-moisture conditioned to wet of optimum as stated above before concrete is placed.


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We recommend that garage floor slabs be a minimum of four inches thick and be reinforced according to the recommendations set forth by the structural designer. During construction, care should be taken such that reinforcement is placed at the slab mid-height, particularly when using welded-wire fabric. The garage floor slabs should be separated from adjacent foundations by low-friction felt or mastic materials to allow for some differential movement at this interface.

## 5.5 ASPHALT PAVEMENT SECTIONS

Pavement design sections presented in Table 3 below were calculated using the State of California Department of Transportation (Caltrans) flexible pavement design method and the an assumed design R-value of 5, representative of the clayey alluvial soils found at the site. Traffic Indices of 4, 5 and 6 were assumed for design of automobile parking and roadway areas, respectively. These choices for Traffic Indices should be reviewed by the project civil engineer to evaluate their suitability for the proposed development; changes in Traffic Indices will require modification of the pavement sections recommended below. At pavement edges that are unprotected from moisture gain and loss, installation of perimeter moisture barriers as described in Section 5.3.4 of this report are recommended. This will significantly reduce future maintenance of pavement edge cracking.

**TABLE 3**

**RECOMMENDED PAVEMENT SECTIONS**
**R-Value = 5**

| Assumed Traffic Index | Asphalt Concrete (Inches) | Aggregate Base (Inches) |
|---|---|---|
| 4 | 2.5 | 7.5 |
| 5 | 2.5 | 11 |
| 6 | 3.0 | 13.5 |

These recommended pavement sections assume the following conditions:


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1. Expansive soil subgrades are compacted to at between 90 and 92 percent relative compaction at a moisture content of at least 4 percent above optimum for the top 6 inches. Subgrade soils should have shrinkage cracks closed for their full depth, and be maintained at their above optimum moisture content, until covered by permanent construction and protected by perimeter moisture barriers.
2. Aggregate base meets with Caltrans Standard Specifications requirements for Class 2 materials.
3. All imported select fill subgrade (if any) and aggregate base materials are compacted to at least 95 percent relative compaction at near optimum moisture contents.
4. Pavement areas are sloped to drain so that the risk of subgrade soil saturation, and corresponding strength loss, is reduced.
5. Subgrade soils are firm and unyielding before aggregate base materials are placed.
6. Asphalt concrete surfacing meets with Caltrans Standard Specifications requirements, and is placed and compacted on firm and unyielding aggregate base materials.

If the on-site expansion soils are lime-treated, the pavement section aggregate base thickness can be reduced. Actual pavement design for lime-treated subgrade soils can be calculated after completion of additional lime-soil laboratory testing including possible R-value tests.


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# 6.0 ADDITIONAL SERVICES AND LIMITATIONS

## 6.1 ADDITIONAL SERVICES

The conclusions and recommendations contained in this report are based on six (6) borings, ten (10) test pits, limited laboratory testing and engineering analysis. Additional information on subsurface conditions at the site will become available during the course of construction. As such, the review of project plans and specifications, along with field observation and testing during earthwork and building construction by Kleinfelder, are an integral part of the conclusions and recommendations made in this report. If Kleinfelder is not retained for these services, then our client, Cal-waii Molding Company, Inc., will be assuming Kleinfelder's responsibility for any potential claims that may arise during or after construction. The recommended tests, observations, and consultation by Kleinfelder prior to and during construction include, but are not limited to:

- Review of plans and specifications.
- Observation of grading, fill removal, fill placement and foundation excavations.
- Observation and testing of engineered fill, finished subgrade and aggregate base.

We have provided Cal-waii Molding Company, Inc. with six (6) bound original copies of this report. If additional copies are required, we can provide them at an additional fee (in accordance with our current fee schedule) after receipt of a written request from our client. **Under no circumstances will we provide a copy of the report to other design consultants or contractors without written permission from our client.**

The above additional services are not included as part of our agreement for this investigation but can be provided by our firm on a time-and-expense basis when requested.


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## 6.2 LIMITATIONS

The recommendations contained in this report are subject to the limitations presented herein. In addition, a brochure prepared by ASFE (Association of Firms Practicing in the Geosciences) has been included in Appendix B of this report. We recommend that all individuals reading this report also read this attached brochure.

Recommendations contained in this report are based on our field observations, data from six (6) borings, ten (10) test pits, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that subsurface conditions could vary between or beyond the points explored. If soil and groundwater conditions are encountered during construction that differ from those described herein, our firm should be notified immediately in order that a review may be made and supplemental recommendations provided, if warranted. If the scope of the proposed construction, including the type of structures and planned grading, changes from that described in this report, our recommendations should also be reviewed and modified, where necessary.

Our firm has prepared this report for the exclusive use of Cal-waii Molding Company, Inc. and Workforce Housing, LLC or their design professionals in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our investigation. No warranty is expressed or implied. The recommendations provided in this report are based on the assumption that our firm will conduct an adequate program of testing and observation during subsequent earthwork and building construction phases in order to evaluate compliance with our recommendations. If we are not retained for these services, our client must assume Kleinfelder's responsibility for any potential claims that may arise during or after construction and Kleinfelder will cease to be the Geotechnical Engineer-of-Record.

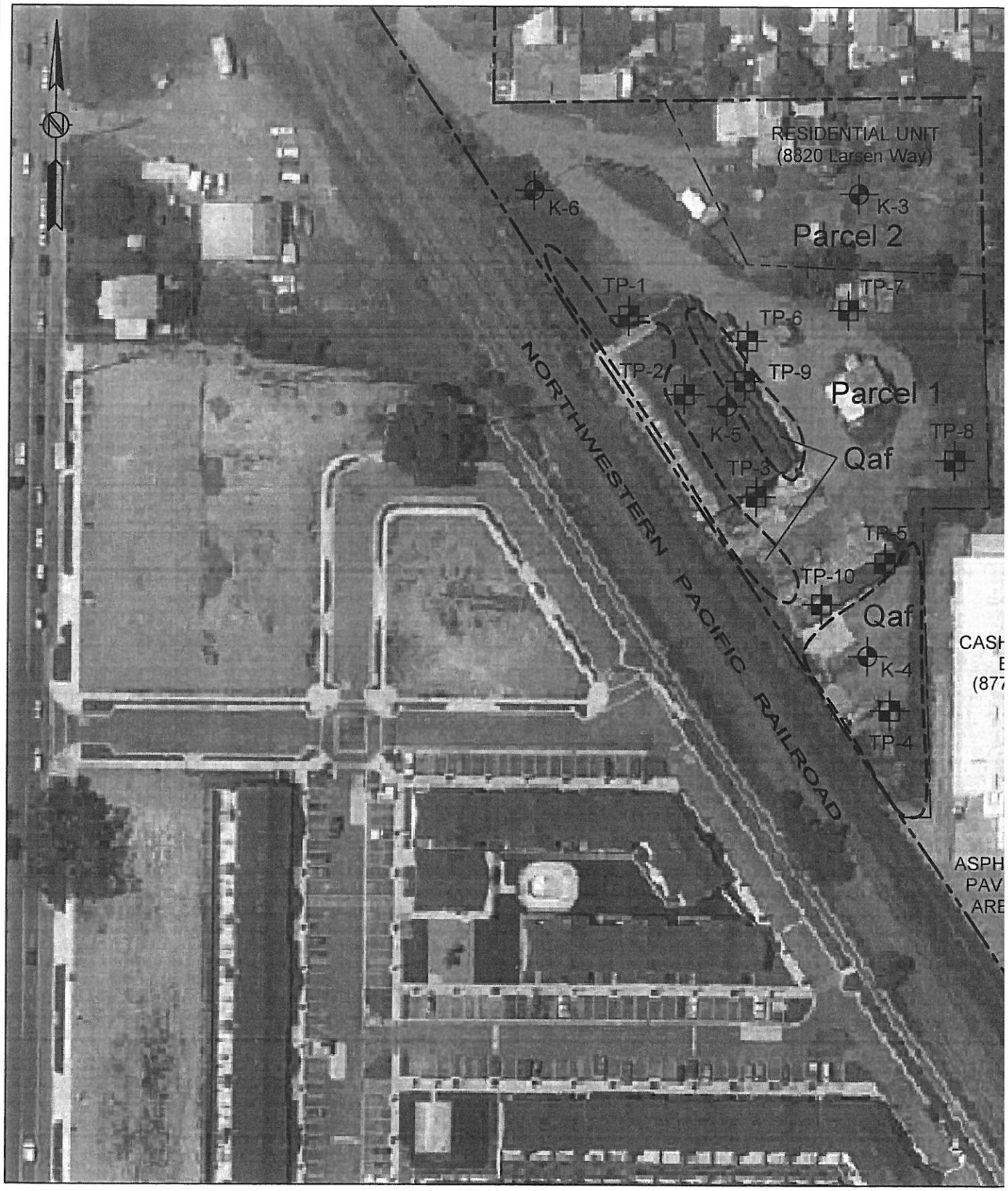
This study specifically did not include an analysis of environmental conditions or an evaluation of possible toxic materials that may be present on, at or below this site. A preliminary environmental assessment (PSA) has been provided under separate cover.


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This report is issued with the understanding that Cal-waii Molding Company, Inc. has assumed the risk they wish to bear by the design approach, construction expenditures and scheduling that are chosen. Homeowners should be made aware of the need to maintain proper drainage away and from around structures and to keep the soils near the foundation in a moist condition year-round. Additionally, the planting of trees close to foundations can result in detrimental loss of moisture in the soils adjacent to the structure, which can result in drying of the soils and distress to buildings and improvements. This is especially significant for shallow foundations and improvements. If trees are proposed near foundations or improvements, an arborist may need to be consulted to recommend tree types or distances away from improvements. It is our Client's responsibility to see that all parties to the project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety including the Additional Services and Limitations sections.

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Source: Sonoma County Aerial Photography

## UNIFIED SOIL CLASSIFICATION SYSTEM

| MAJOR DIVISIONS | | | | DESCRIPTIVE NAMES |
|---|---|---|---|---|
| COARSE GRAINED SOILS More than Half > #200 sieve | GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE | CLEAN GRAVELS WITH LITTLE OR NO FINES | GW | WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES |
| | | | GP | POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES |
| | | GRAVELS WITH OVER 12% FINES | GM | SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES |
| | | | GC | CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES |
| | SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE | CLEAN SANDS WITH LITTLE OR NO FINES | SW | WELL GRADED SANDS, GRAVELLY SANDS |
| | | | SP | POORLY GRADED SANDS, GRAVELLY SANDS |
| | | SANDS WITH OVER 12% FINES | SM | SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES |
| | | | SC | CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES |
| FINE GRAINED SOILS More than Half < #200 sieve | SILTS AND CLAYS LIQUID LIMIT LESS THAN 50 | | ML | INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY |
| | | | CL | INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS |
| | | | OL | ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY |
| | SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 | | MH | INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, 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 AND OTHER HIGHLY ORGANIC SOILS |

### *FIELD SAMPLING*

- MODIFIED CALIFORNIA SAMPLE
- DISTURBED, BAG OR BULK SAMPLE
- STANDARD PENETRATION TEST
- SHELBY TUBE SAMPLE
- 3-1/2" I.D. CONTINUOUS CORE SAMPLE
- UNRETAINED PORTION OF SAMPLE
- WATER LEVEL OBSERVED IN BORING (at given post-drilling time)
- WATER LEVEL OBSERVED IN BORING (at time of drilling)

### *LABORATORY TESTS*

| | |
|---|---|
| LL | LIQUID LIMIT |
| PI | PLASTICITY INDEX |
| SA | SIEVE ANALYSIS |
| #200 | PERCENT PASSING #200 SIEVE |
| RV | RESISTANCE VALUE |
| EI | EXPANSION INDEX |
| DS | DIRECT SHEAR |
| Tx/UU | TRIAXIAL SHEAR-UNCONSOLIDATED UNDRAINED |
| UC | UNCONFINED COMPRESSION |
| SG | SPECIFIC GRAVITY |
| Consol | CONSOLIDATION |

NOTES: Blow counts represent the number of blows of a 140-pound hammer falling 30-inches required to drive a sampler the last 12-inches of an 18-inch penetration. The blow counts have been converted to standard N-value blow counts.

The lines separating strata on the logs represent approximate boundaries only. The actual transition may be gradual. No warranty is provided as to the continuity of soil strata between borings. Logs represent the soil strata and groundwater observed at the boring location on the date of drilling only.

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PROJECT NUMBER **43125-1** DATE **MAY 2004**

**BORING/TEST PIT LOG LEGEND**

**Windsor Mill II**

**Windsor, California**

| Other Laboratory Tests | Dry Density (pcf) | Moisture Content (%) | Blows/ Foot * | DEPTH (FEET) | |
|---|---|---|---|---|---|
| | | | | 0 | **SILTY SANDY GRAVEL** -gray, wet, dense, fine to coarse grained sand, angular gravels to 1-1/2" diameter, geotextile fabric (Aggregate Base) |
| | | | 48 | 1 | |
| | | | | 2 | **CLAYEY GRAVELLY SAND** - gray-brown, moist, dense, fine to coarse grained sand, subangular to subrounded gravels to 1/2" diameter (Fill) |
| | | | | 3 | |
| | | | 35 | 4 | **CLAYEY SAND** - olive-brown to yellow-brown, moist, dense, fine to coarse grained sand (QTge) |
| UC = 3760 psf | 102 | 20.2 | | 5 | |
| | | | 34 | 6 | |
| | | | | 7 | harder drilling with gravels at 7 feet |
| | | | | 8 | **SILTY SANDY CLAY WITH SOME GRAVEL** - light olive, moist, hard, fine grained sand, subrounded gravel to 1/4" diameter (QTge) |
| | | | | 9 | |
| | | | | 10 | |
| | | | 40 | 11 | |
| | | | | 12 | |
| | | | | 13 | **SANDY CLAY(STONE)** - olive-brown, moist, hard, fine to medium grained sand (QTge) |
| | 99 | 23.4 | 89 | 14 | |
| | | | | | BOTTOM OF BORING K-1 @ 14.5 FEET<br>No Free Water Encountered |

EQUIPMENT: **6" Flight Auger** ELEVATION: ******

LOGGED BY: **B. Conder** START DATE: **4-20-04**

FINISH DATE: **4-20-04**

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of drilling.

KLEINFELDER

PROJECT NUMBER **43125-1** DATE **MAY 2004**

# LOG OF BORING K-1

**Windsor Mill II**

**Windsor, California**

| Other Laboratory Tests | Dry Density (pcf) | Moisture Content (%) | Blows/ Foot * | DEPTH (FEET) | |
|---|---|---|---|---|---|
| | | | | 0 | ~2" ASPHALT CONCRETE PAVEMENT |
| | | | | 1 | **SANDY SILTY GRAVEL** - dark brown, moist, fine to coarse grained sand, angular gravel to 1" diameter (Aggregate Base) |
| EI = 16 very low | | | 14 | 2 | **CLAY WITH SAND** - dark gray, moist, stiff, fine grained sand, moderately plastic |
| | | | | 3 | |
| | | | 22 | 4 | **CLAYEY SAND** - gray, moist, medium dense, fine to coarse grained sand, slightly porous |
| | | | | 5 | **SANDY CLAY** - gray-brown, moist, very stiff, fine to coarse grained sand, plastic (QTge) |
| | 96 | 29.1 | 26 | 6 | |
| | | | | 7 | **CLAYEY SAND** - olive-brown, moist, medium dense, fine to medium grained sand (QTge) |
| | | | | 8 | |
| | | | | 9 | |
| | | | 25 | 10 | |
| | | | | 11 | |
| | | | | 12 | **SILTY CLAYEY SAND** - olive-brown, wet, very dense, fine to medium grained sand (QTge) |
| | | | | 13 | ▽ |
| | 95 | 27.5 | 57 | 14 | |
| | | | | 15 | BOTTOM OF BORING K-2 @ 15 FEET |

EQUIPMENT: **6" Flight Auger** ELEVATION: **110.0' +/- ****

LOGGED BY: **B. Conder** START DATE: **4-20-04**

FINISH DATE: **4-20-04**

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of drilling.

KLEINFELDER

PROJECT NUMBER **43125-1** DATE **MAY 2004**

# LOG OF BORING K-2

**Windsor Mill II**

**Windsor, California**

| Other Laboratory Tests | Dry Density (pcf) | Moisture Content (%) | Blows/ Foot * | DEPTH (FEET) | |
|---|---|---|---|---|---|
| | | | | 0 | **SANDY SILT** - brown, moist, very stiff, fine to medium grained sand, porous, rootlets |
| | 91 | 8.8 | 20 | 1 | |
| | | | | 2 | **SANDY CLAY** - brown, moist, very stiff, fine to coarse grained sand, moderately plastic, rootlets (QTge) |
| Corrosivity | | | 68 | 3 | **SANDY CLAY** - light olive-brown, moist, hard, fine grained sand (QTge) |
| | | | | 4 | |
| | | | 75 | 5 | |
| | | | | 6 | |
| | | | | 7 | **CLAYEY SAND** - olive-brown with orange mottles, moist, dense, fine to medium grained sand (QTge) |
| | 98 | 22.7 | 48 | 8 | |
| | | | | 9 | |
| | | | | 10 | **SANDY CLAY** - olive-brown with orange mottles, moist, hard, fine grained sand (QTge) |
| | | | | 11 | |
| | | | | 12 | |
| | | | | 13 | |
| | | | | 14 | |
| | | | 36 | 15 | **CLAYEY SAND** - olive-brown, moist, dense, fine to coarse grained sand (QTge) |

BOTTOM OF BORING K-3 @ 15 FEET
No Free Water Encountered

EQUIPMENT: **6" Flight Auger** ELEVATION: **112' +/- ****
LOGGED BY: **B. Conder** START DATE: **4-20-04**
FINISH DATE: **4-20-04**

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of drilling.

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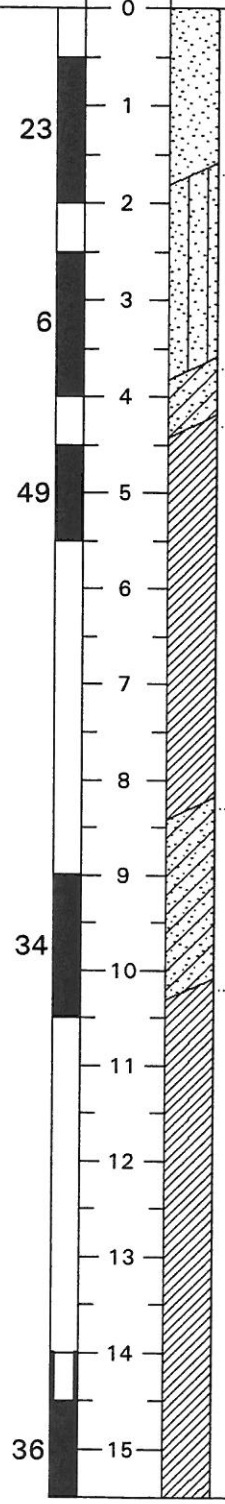
PROJECT NUMBER **43125-1** DATE **MAY 2004**

# LOG OF BORING K-3

**Windsor Mill II**

**Windsor, California**

| Other Laboratory Tests | Dry Density (pcf) | Moisture Content (%) | Blows/ Foot * | DEPTH (FEET) | EQUIPMENT: 6" Flight Auger | ELEVATION: 112.5' + /- ** |
|---|---|---|---|---|---|---|
| | | | | | LOGGED BY: B. Conder | START DATE: 4-20-04 |
| | | | | | | FINISH DATE: 4-20-04 |




**GRAVELLY SAND** - brown, moist, medium dense, fine to coarse grained sand, subrounded gravel to 1" diameter (Fill)

**SILTY SAND WITH GRAVEL** - dark gray-brown, wet, loose, fine to medium grained sand, subrounded gravel to 1/4" diameter (Fill)

105 19.6

**CLAYEY SAND** - olive-brown, moist, loose, fine to medium grained sand

**SANDY CLAY** - olive-brown, moist, hard, fine grained sand, plastic (QTge)

easier drilling at 8 feet

**CLAYEY SAND** - olive-brown, moist, dense, fine to medium grained sand (QTge)

-#200 = 32%

**SILTY SANDY CLAY** - light olive-brown, moist, hard, fine grained sand (QTge)

BOTTOM OF BORING K-4 @ 15.5 FEET
No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of drilling.

KLEINFELDER

PROJECT NUMBER 43125-1 DATE MAY 2004

# LOG OF BORING K-4

Windsor Mill II

Windsor, California

| Other Laboratory Tests | Dry Density (pcf) | Moisture Content (%) | Blows/ Foot * | DEPTH (FEET) | |
|---|---|---|---|---|---|
| | | | | 0 | **SILTY SANDY GRAVEL** - gray-brown, dry, loose, fine to coarse grained sand, subangular gravel to 1" diameter (Fill) |
| | | | 41 | 1 | |
| | 108 | 17.4 | | 2 | **CLAYEY SAND WITH GRAVEL** - brown, moist, dense, fine to coarse grained sand, subangular gravel to 3/4" diameter (QTge) |
| | | | | 3 | |
| LL = 41, PI = 23 | | | 40 | 4 | **CLAYEY SAND/SANDY CLAY** - olive, moist, dense/hard, fine to medium grained sand, moderately plastic (QTge) |
| | | | | 5 | |
| | 103 | 23.3 | 59 | 6 | olive-brown, very dense |
| | | | | 7 | **SILTY CLAY WITH SAND** - light olive with orange mottles, moist, hard, fine grained sand (QTge) |
| | | | | 8 | |
| | | | 52 | 9 | |
| | | | | 10 | |
| | | | | 11 | |
| | | | | 12 | |
| | | | | 13 | **CLAYEY SAND** - olive-brown, moist, dense, fine to coarse grained sand (QTge) |
| | | | 50 | 14 | **SILTY SANDY CLAY** - olive, moist, hard, fine grained sand (QTge) |
| | | | | | BOTTOM OF BORING K-5 @ 14.5 FEET<br>No Free Water Encountered |

EQUIPMENT: **6" Flight Auger** ELEVATION: **115.2' +/- ****

LOGGED BY: **B. Conder** START DATE: **4-20-04**

FINISH DATE: **4-20-04**

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of drilling.

KLEINFELDER

PROJECT NUMBER **43125-1** DATE **MAY 2004**

# LOG OF BORING K-5

**Windsor Mill II**

**Windsor, California**

EQUIPMENT: **6" Flight Auger** ELEVATION: **115.0' +/- ****

LOGGED BY: **B. Conder** START DATE: **4-20-04**

FINISH DATE: **4-20-04**

| Other Laboratory Tests | Dry Density (pcf) | Moisture Content (%) | Blows/ Foot * | DEPTH (FEET) | |
|---|---|---|---|---|---|
| | | | | 0 | **CLAYEY GRAVELLY SAND** - dark gray, moist, dense, fine to coarse grained sand, subangular gravel to 1/2" diameter (Fill) |
| | | | 37 | 1 | |
| | | | | 2 | **SANDY SILT WITH CLAY AND GRAVEL** - dark brown, moist, stiff, fine to coarse grained sand, subangular gravel to 1/2" diameter |
| | | | 13 | 3 | **SILTY SAND WITH GRAVEL** - brown, moist, medium dense, fine to coarse grained sand, subrounded gravel to 1/2" diameter |
| | | | | 4 | **SANDY CLAY** - olive-brown, moist, stiff, fine grained sand, plastic (QTge) |
| UC = 4403 psf | | | 50 | 5 | |
| | 112 | 18.3 | | | grades to sandy clay with gravel, hard, fine to coarse grained sand, subrounded gravel to 1/2" diameter |
| | | | | 6 | |
| | | | | 7 | grades to clay with sand, fine grained sand |
| | | | 47 | 8 | |
| | | | | 9 | |
| | | | | 10 | |
| | | | | 11 | **SILTY CLAY WITH SAND** - olive-brown, moist, hard, fine grained sand (QTge) |
| | | | | 12 | |
| | | | 31 | 13 | |

BOTTOM OF BORING K-6 @ 13.5 FEET
No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of drilling.

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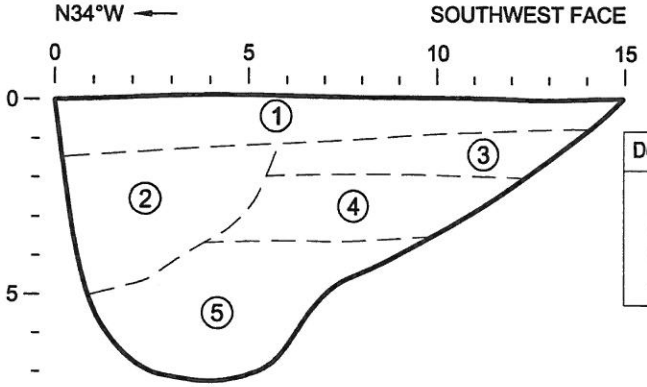
PROJECT NUMBER **43125-1** DATE **MAY 2004**

# LOG OF BORING K-6

**Windsor Mill II**

**Windsor, California**

## TP-1




| Depth | P.P. |
|---|---|
| 1 | 0.0 |
| 2 | 4.5+ |
| 3 | 2.5 |
| 5 | 4.5+ |

1. SANDY GRAVELLY SILT (ML) - light gray brown, dry, soft, firm, coarse grained sand, subrounded gavel to 2 inch diameter, abundant organics, "loose" (Fill).

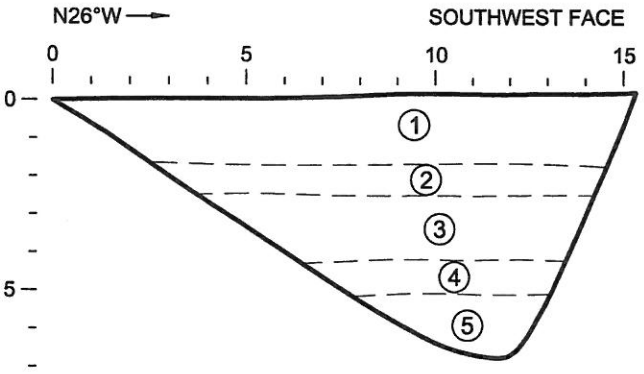
2. SANDY GRAVELY CLAY (CL) - dark brown, gray below, wet, soft, fine to coarse grained sand, subangular to subrounded gravels, 3 inch diameter, brick and concrete fragments to 2 feet diameter, abundant organic fill (Fill).

3. SILT (ML) - gray with orange mottles, dry, very stiff, porous.

4. CLAY WITH TRACE SAND (CL) - light blue, moist, stiff, fine grained sand, moderately plastic.

5. CLAY WITH SAND (CL/CH) - light brown with black, MnO mineralization, moist, hard, fine grained sand (QTge).

## TP-4




| Depth | P.P. |
|---|---|
| 2 | 2.0 |
| 4 | 2.0 |
| 6 | 4.5+ |

1. SANDY GRAVEL (GP) - Brown, gray brown, dry to damp, medium dense, fine to coarse grained sand, subrounded gravel 3 inch diameter (Fill).

2. CLAYEY SILT WITH SAND (ML) - gray brown to dark lean gray, moist to wet, soft, fine grained sand, highly porous.

3. CLAYEY SAND WITH SILT (SC) - dark brown, wet, soft, fine grained sand, porous.

4. CLAY WITH TRACE SAND (CL) - Light olive brown, moist, stiff, fine grained sand, moderately plastic.

5. CLAY WITH SAND (CL) - Light brown with black MnO mineralization,, damp, hard, fine grained sand (QTge).

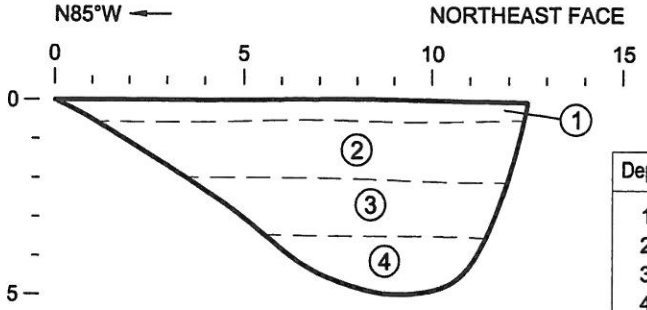
## EXPLANATION

- Drive Sample
- Bulk Sample
- PP Pocket Penetrometer (tsf)
- Seepage
- Groundwater Elevation

5 2.5 0

scale

## TP-7




| Depth | P.P. |
|---|---|
| 1 | 4.5+ |
| 2 | 2.0 |
| 3 | 2.5 |
| 4 | 4.5+ |

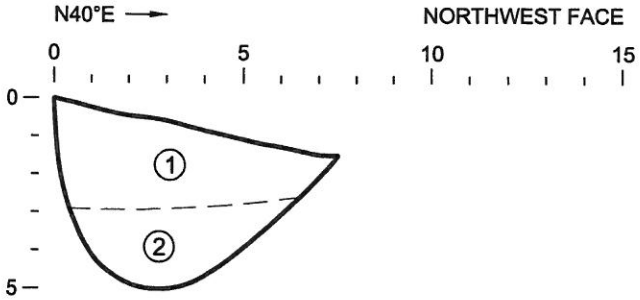
1. SANDY GRAVEL WITH SILT (GP) - Gray, dry, medium dense, angular gravel to 0.75 inch diameter, fine grained sand (Fill).

2. SANDY SILT (ML) - Dark brown, moist, stiff, porous, fine grained sand.

3. CLAY WITH SAND (CL) - Olive brown, moist, stiff, fine to medium grained sand, plastic.

4. CLAY WITH TRACE SAND - Olive brown, moist, hard, fine-grained sand, friable (QTge).

## TP-9

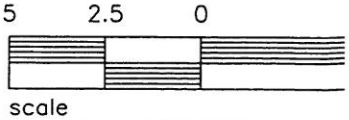



1. SANDY GRAVEL WITH SILT (GM) - Dark gray to gray brown, dry, loose, fine to coarse grained sand, subangular gravel to 1 inch diameter (Fill).

2. SANDY CLAY (CL) - Yellow brown with black MnO mineralization, moist, hard, fine grained sand (QTge).

## EXPLANATION

- Drive Sample
- Bulk Sample
- PP Pocket Penetrometer (tsf)
- Seepage
- Groundwater Elevation



**Windsor Mill II**
**Boring K-2 @ 2.0'**
**Dark Brown Clay with Sand**

## UBC EXPANSION INDEX

**Initial**

| | |
|---|---|
| Sample Height (in): | 1.0000 |
| Moisture Content (%): | 13.2 |
| Dry Density (pcf): | 98.7 |
| Void Ratio: | 0.707 |
| Saturation (%): | 50.6 |

**Final**

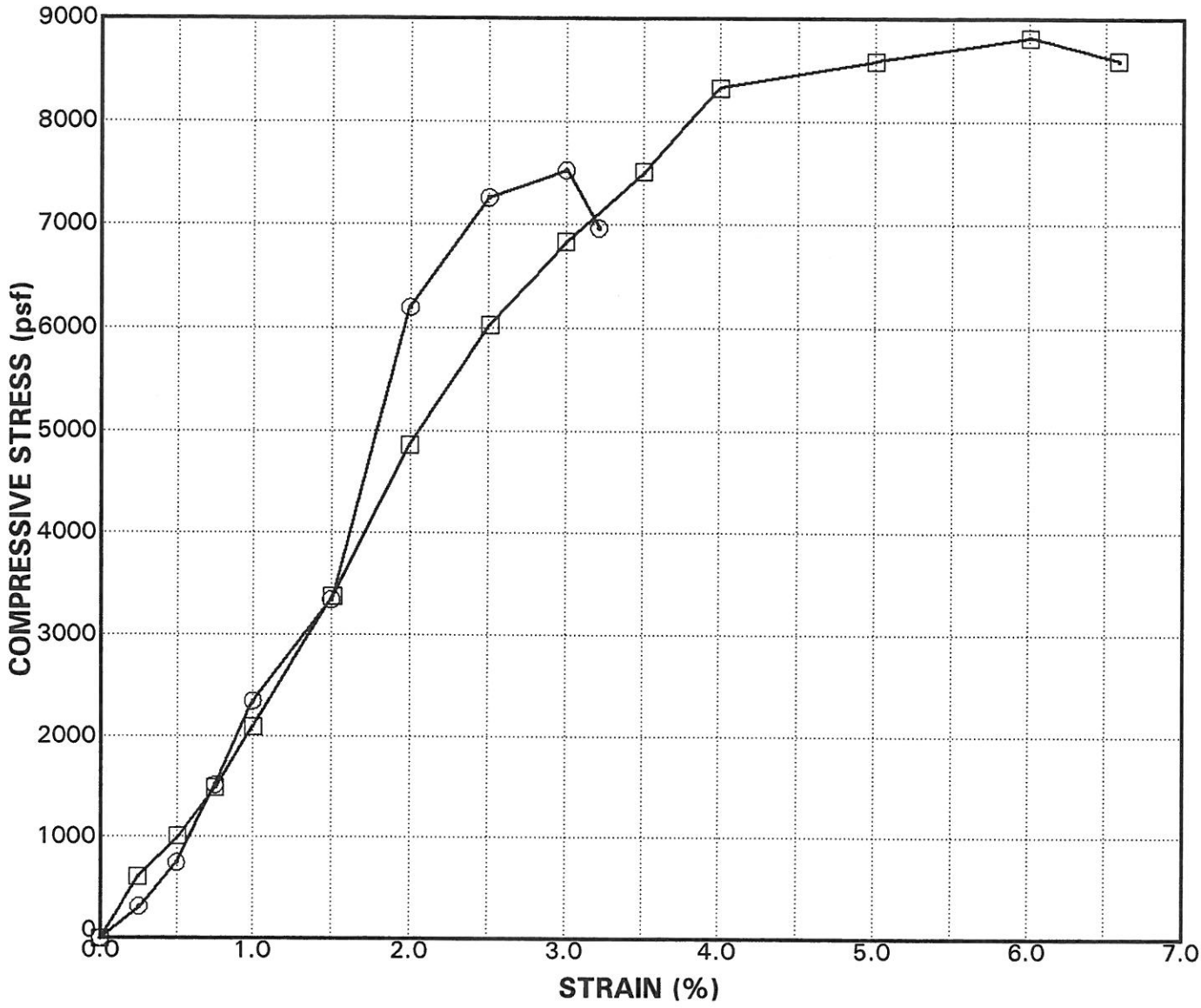
| | |
|---|---|
| Sample Height (in): | 1.0156 |
| Moisture Content (%): | 23.4 |
| Void Ratio: | 0.733 |
| Saturation (%): | 86.3 |

EXPANSION INDEX LEVELS:

- 0-20 = Very Low
- 21-50 = Low
- 51-90 = Medium
- 91-130 = High
- >130 = Very High

Expansion Index: 16

Expansion Index Level: VERY LOW




| Sample Source | Classification | Type of Test | Confinement Pressure (psf) | Ultimate Stress (psf) | Strain (%) | Dry Density (pcf) | Moisture Content (%) |
|---|---|---|---|---|---|---|---|
| ⊙ K-1 @ 4.0' | BROWN CLAYEY SAND | UC | --- | 7519 | 3 | 102 | 20.2 |
| ⊡ K-6 @ 5.0' | BROWN SANDY SILTY CLAY | UC | --- | 8806 | 6 | 112 | 18.3 |

UC = Unconfined Compression

TX/UU = Unconsolidated Undrained Triaxial

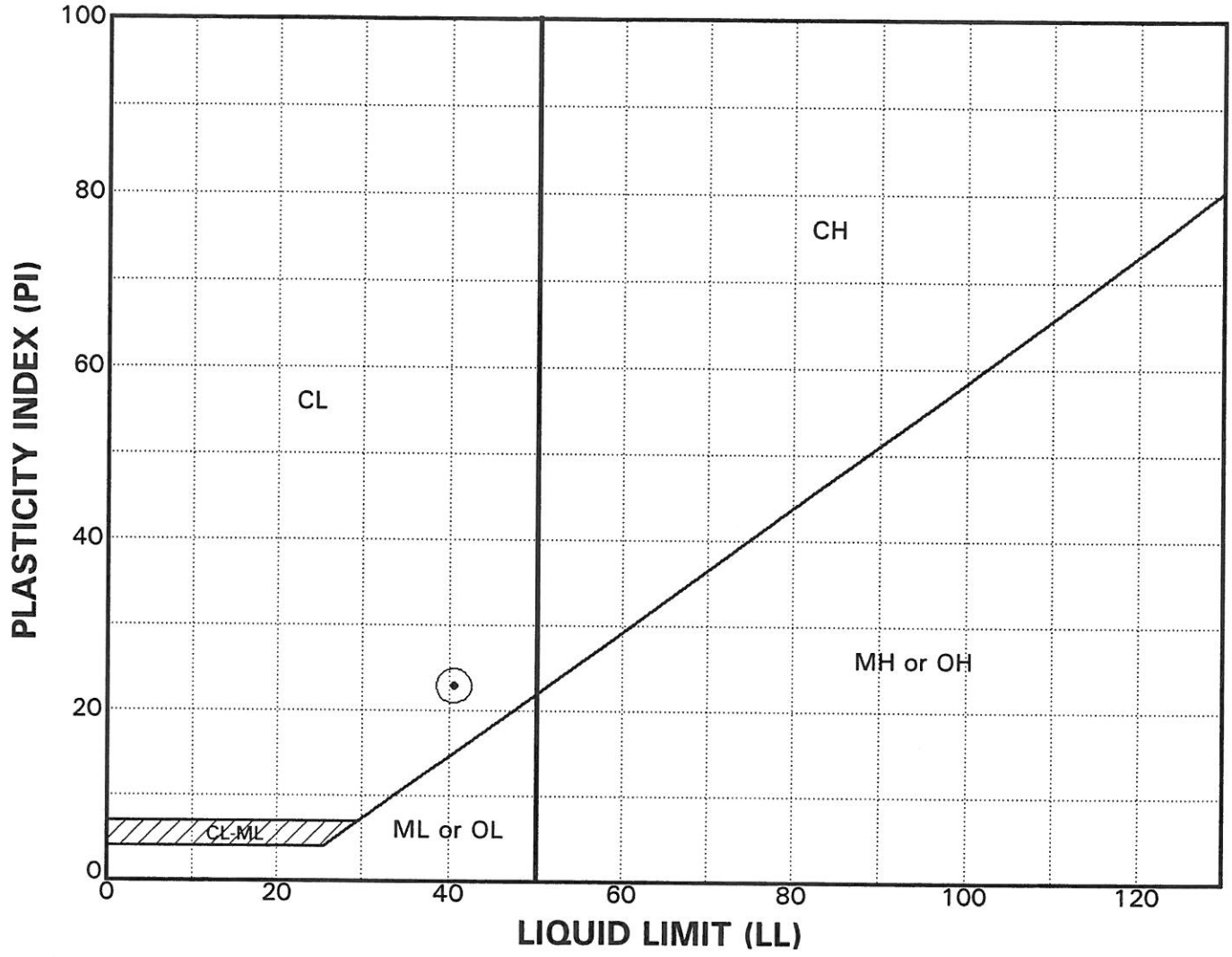
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PROJECT NUMBER **43125-1** DATE **MAY 2004**

# STRENGTH TEST DATA

**Windsor Mill II**

**Windsor, California**




| SAMPLE SOURCE | CLASSIFICATION | LIQUID LIMIT (%) | PLASTIC LIMIT (%) | PLASTICITY INDEX (%) | % PASSING #200 SIEVE |
|---|---|---|---|---|---|
| ⊙ K-5 @ 4.0 | BROWN SANDY LEAN CLAY (CL) | 41 | 18 | 23 | -- |
| | | | | | -- |

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PROJECT NUMBER 43125-1 DATE MAY 2004

**PLASTICITY CHART**

**Windsor Mill II**

**Windsor, California**

**13**

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