GEOTECHNICAL INVESTIGATION REPORT
PROPOSED DEL PASO PARK DEVELOPMENT
FULTON AVENUE
SACRAMENTO, CALIFORNIA

August 7, 2006

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August 7, 2006
File: 67337

Ms. Denise Malvetti
Economic Development Department
City of Sacramento
1030 Fifteenth Street, Second Floor
Sacramento, California 95814-4009

Subject: Geotechnical Investigation Report
Proposed Del Paso Park Development
Sacramento, California

Dear Ms. Malvetti:

Kleinfelder is pleased to present the attached geotechnical investigation report for the proposed Del Paso Park Development project to be located in Sacramento, California. The purpose of our investigation was to explore and evaluate the subsurface conditions at various locations on the site in order to develop geotechnical engineering recommendations for project design and construction.

Based on the results of our field investigation, laboratory testing, and engineering analyses, it is our professional opinion the site may be developed for the proposed Del Paso Park Development project using conventional grading, foundation, and an appropriate trenchless construction technique. Recommendations regarding the geotechnical aspects of project design and construction are presented in the following report.

Recommendations provided herein are contingent on the provisions outlined in the Additional Services and Limitations section of this report. The project Owner should become familiar with these provisions in order to assess further involvement by Kleinfelder and other potential impacts to the proposed project.

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We appreciate the opportunity of providing our services for this project. If you have questions regarding this report or if we may be of further assistance, please contact the undersigned.

Sincerely,

KLEINFELDER, INC.

Mark D. Fuhriman, P.E.
Project Engineer

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Senior Geotechnical Engineer

MDF:KGS:aak

cc: Client (3)

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GEOTECHNICAL INVESTIGATION REPORT
PROPOSED DEL PASO PARK DEVELOPMENT
SACRAMENTO, CALIFORNIA

1 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Del Paso Park Development project to be located at the former Sacramento Trap Shoot Club site at the northeast corner of Fulton Avenue and the Interstate 80 Business Route freeway in Sacramento, California. A Site Location Map is presented on Plate 1. We understand the results of our geotechnical investigation will be incorporated into the Environmental Impact Report (EIR) for the proposed project. The following sections of this report describe our understanding of the project and our scope of services.

1.1. PROJECT DESCRIPTION

Our understanding of the site remediation program and project development plans is based on information provided by Wood Rodgers, the project civil engineer, and Forrar Williams Architects, the project architect. We understand the proposed project will involve on-site soil remediation and redevelopment of the former trap shoot site into an automobile dealership facility. The project will also include the northerly extension of Fulton Avenue along the east side of the adjacent Haggin Oaks golf course parking lot, construction of a 3.4-acre-foot, approximately 8-foot deep storm water detention basin, construction of a new golf course trail extension, and installation of new pipelines for storm drain, sanitary sewer, and municipal water supply. A site plan provided by Wood Rodgers is shown on Plate 2.

The site of the proposed dealership complex is situated over two parcels known as Parcel A (10.8 acres) and Parcel B (6.7 acres). According to drawings provided by Wood Rodgers, some of the soils within and around these parcels are contaminated with lead shot. We understand that site remediation earthwork will include the following:

- Removal of approximately 14,000 cubic yards (cy) of clay pigeon debris and lead shot from Parcels A and B for off-site disposal,

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- Transfer of approximately 34,900 cy of contaminated soil from Parcel A into Parcel B, and
- Transfer of approximately 2,800 cy of contaminated soil from an area north of and immediately adjacent to Parcel A into Parcel B.

According to a drawing provided by Wood Rogers, the contaminated soil that is to be transferred into Parcel B will be remediated in-place at Parcel B, along with about 21,700 cy of contaminated soil already in Parcel B. Total vertical cuts in the west side of Parcel A will be up to about 5 feet, while up to about 9 to 10 feet of fill will be placed in other portions of Parcel A and in Parcel B. Final site grading will provide drainage to the north (see Site Plan, Plate 2). Final graded surface elevation at the south end of the site (adjacent to Interstate 80 Business Route freeway) will be about El. +68 feet. At the north end of the site (just south of and adjacent to the future extension of Fulton Avenue), the graded surface elevation will be about El. +59 feet.

We understand that the proposed automobile dealership will include 40,000 and 72,000-square-foot showroom buildings and an approximately 25,000-square-foot service center building. Asphalt-paved driveways and automobile parking areas will also be constructed around the dealership complex, as shown on Plate 2. Maximum column loads for the new buildings are presently unknown but expected to be typical of one- and two-story concrete masonry unit or concrete tilt up construction. Building floors will likely be of concrete slab-on-grade construction.

The pipelines will range in size from 8 inches to 30 inches. We anticipate the sewer and storm drain pipelines will be installed using open-cut trenching methods. However, one segment of 16-inch water pipeline that will cross under the Interstate 80 Business Route freeway to the proposed tie-in point at Auburn Boulevard will be installed using a trenchless method. Proposed pipeline materials include reinforced concrete for the sewers and storm drains, PVC for the on-site water, and ductile iron installed within a steel casing for the trenchless water pipeline installation under the freeway.

The information described above is our understanding of the project at this time. If the proposed project changes, we should be contacted to review our conclusions and recommendations for their applicability to the proposed construction.

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1.2. PURPOSE AND SCOPE OF SERVICES

The purposes of this study were to explore subsurface conditions at the site and to provide recommendations for geotechnical aspects of design and construction of the proposed project in support of the EIR. We accomplished these purposes by:

- Reviewing readily-accessible geologic and geotechnical information in the general site vicinity;
- Drilling 20 geotechnical soil borings to explore subsurface conditions and to obtain samples for laboratory testing (see Plate 3 for boring locations);
- Conducting geotechnical laboratory tests to assess pertinent soil engineering properties;
- Analyzing the field and laboratory data to develop conclusions and recommendations; and
- Preparing this report.

Environmental evaluations, assessments and analytical testing for soil contaminants is outside the scope of this report.

1.3. REPORT ORGANIZATION

The following sections of this report describe the geologic and seismic setting of the project, describe the subsurface conditions, and present our conclusions and geotechnical recommendations for design and construction. A description of the field exploration program and the exploratory boring logs are presented in Appendix A. A description of the geotechnical laboratory testing program and laboratory test results are presented in Appendix B.

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2 GEOLOGIC AND SEISMIC SETTING

2.1. REGIONAL GEOLOGY

The subject site is located in the southern portion of the Sacramento Valley. The Sacramento Valley represents the northward extension of California's Great Valley Geomorphic Province which is characterized by a thick accumulation of alluvial and floodplain deposits within an asymmetric trough, approximately 400 miles long and 40 miles wide. Erosion of the Sierra Nevada to the east and the Coast Ranges to the west has resulted in the accumulation of thousands of feet of granular and fine-grained alluvium in this valley. The deposits thin out, or overlap, onto older bedrock units representative of the mountain provinces along the boundaries of the basin.

2.2. LOCAL AND SITE GEOLOGY

Geologic mapping for the site vicinity has been performed at the greatest detail by Helley (1985) and is reproduced in part on Plate 4, Geologic Map. Helley (1985) indicates the southern half of the site is underlain by Quaternary alluvium (symbol Qa on Plate 4). This formation is described as Holocene age, unweathered and unconsolidated gravel, sand, and silt deposited by present-day streams and rivers. The Lower Member of the Riverbank Formation (symbol Qrl on Plate 4) is mapped underlying the northern half of the project. This formation is described as red semiconsolidated gravel, sand, and silt.

2.3. SEISMICITY AND REGIONAL FAULTING

2.3.1. Regional Faults

Tectonically, the Sacramento region is situated between several major, active fault systems including the San Andreas Fault System and Coast Range-Central Valley (CRCV) geomorphic block boundary to the west and the Sierra Nevada Fault System to the east. The San Andreas and Sierra Nevada systems are well known for Holocene rupture that has continued into historic time and represent the source of most of California's seismic history. The CRCV and Foothills Fault Systems are more

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controversial in that only short segments are suspected to have ruptured in the recent geologic past and the frequency of rupture of each of these fault systems is considerably lower than other, more active faults.

Geographic information system databases and Jennings' Fault Activity Map of California (Jennings, 1994) indicate the nearest faults to the site include the CRCV and Dunnigan Hills fault, the Foothills Fault System (FFS), and the Willows Fault.

The Dunnigan Hills fault, located about 26.5 miles west of the site, is mapped by Jennings as an active (i.e. Holocene) fault (Plate 5). However, some geologists and seismologist speculate that the magnitude 5.5 event that occurred near this fault may represent activity along the fault, studies performed by the California Geological Survey (CGS) to consider zoning as an Alquist-Priolo Earthquake Fault Zone (APEFZ) concluded that there was not sufficient evidence to consider this fault active and did not recommend zoning of this fault. The Dunnigan Hills fault is thought to be an expression of the active CRCV located several miles to the west. The nearest segments of the CRVC are included within an APEFZ.

The Foothills Fault System (FFS) is located about 26 miles east of the project site and is represented by the Prairie Creek-Spenceville-Deadman-Cleveland Hill fault system. Although there remains considerable controversy among geologists as to the activity of the FFS, historic seismicity (primarily low to moderate intensity events) aligns well with portions of this system and suggests that the system of faults is at least capable of generating small earthquakes at depth. Ground rupture occurred during the 1975 Oroville earthquake along the northern extent of this fault (Cleveland Hill fault 54± miles). This rupture was studied by the CGS and was placed within an APEFZ and is thus still considered capable of ground-surface rupture.

The nearest fault to the site is the Willows Fault mapped about 1½ miles southwest of the site. The Willows Fault was initially recognized by the linear step in groundwater elevations followed by later studies by the Department of Water Resources (DWR) in which significant elevation differences of older bedrock unit surfaces were identified at depth along this alignment. Although the fault is not considered capable of ground rupture or generation of large earthquakes based on its geologic history, it does infrequently generate small magnitude earthquakes and may be responsible for broad

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doming and folding of the Corning Dome and similar surface features north of Sutter Buttes.

2.3.2. Ground Rupture

The site is not located within or near a state designated Alquist-Priolo Earthquake Fault Zone (APEFZ). Therefore, the risk of ground rupture at the site due to fault displacement is considered to be negligible.

2.3.3. Seismic Ground Motions

Faults within in the region are capable of generating ground shaking at the site. According to the California Geological Survey Seismic Hazards Assessment website (http://www.consrv.ca.gov/cgs/rghm/pshamap/pshamain.html), the anticipated peak horizontal ground acceleration for alluvial site conditions is 0.18g for the Design Basis Earthquake (DBE) ground motion. The DBE is defined as the ground motion having a 10% probability of exceedance in a 50-year period (return period of 475 years).

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3 SITE CONDITIONS

Discussions of the field investigation and laboratory testing programs are presented in Appendices A and B, respectively, of this report. Detailed descriptions of the subsurface conditions encountered during our field investigation are presented on the Logs of Borings in Appendix A. Laboratory test results are provided on the boring logs and in Appendix B.

The following sections describe the general site conditions that we interpreted from the available topographic, subsurface and laboratory test data.

3.1. SITE DESCRIPTION

The site is part of an approximately rectangular-shaped parcel located northeast of the intersection of Fulton Avenue and the Interstate 80 Business Route freeway in Sacramento, California. The site is bounded by Haggin Oaks Golf course fairways to the north and east, the Interstate 80 Business Route freeway to the south, and Fulton Avenue to the west. Currently, the site is occupied by the Sacramento Trapshooting Club's trapshoot gun range, two buildings, and an existing paved parking lot at the southwest corner of the site. Existing site vegetation consists of seasonal weeds and grasses through out the majority of the site, and mature oak and ornamental trees along the site perimeter. Clay pigeon debris up to about 5 feet deep covers a portion of the southwest end of the site.

Site topography slopes gently downward toward the center of the site into a natural drainage basin that runs from the center of the site to a northerly direction. Another natural drainage ditch runs parallel along the eastern site perimeter. Existing maximum vertical relief across the site is approximately 5 to 10 feet.

3.2. SOIL STRATIGRAPHY

Borings B-4, B-9, B-10 and B-18 were drilled over asphalt-paved parking lots, and Borings B-17 and B-19 were drilled over an approximately 5-foot high mound of clay pigeon debris. Boring B-12, which was drilled near a golf course fairway at the north

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end of the site, encountered about 3 feet of highly-plastic clay, which we infer to be fill material for the golf course fairway.

Below the pavement and near-surface fills, subsurface soils consisted primarily of silts, silty and clayey sands, and lean clays, throughout the project area. To the maximum depths explored by our borings (about 16 to 35 feet), the silts and clays tend to be firm to hard, dry to moist, and of low plasticity. The sands tend to have a fine to coarse gradation. Many of the silt and sand samples that we observed exhibit varying degrees of cementation. The soil conditions observed at the site do not appear to be consistent with the description of "alluvium" mapped by Helley (1985) beneath the southern half of the site, but appear to be more representative of soils associated with the Riverbank Formation, as mapped and described by Helley at the north half of the project area.

3.3. GROUNDWATER

At the time of our field investigation, free groundwater was not encountered in our borings. A review of well hydrograph data published by the California Department of Water Resources (DWR) on the internet at: http://well.water.ca.gov/ identified two nearby wells that may represent groundwater conditions at this site. These wells are identified as 09N05E14B001M and 09N05E12L001M. The published hydrographs for these wells suggest that groundwater levels at the site may be near elevation -40 feet NGVD (about 105 to 115 feet below the ground surface).

3.4. VARIATIONS IN SUBSURFACE CONDITIONS

Our interpretations of soil and groundwater conditions, as described above, are based on data obtained from the borings drilled for this study and review of existing information. The conclusions and recommendations that follow are based on those interpretations. It is likely that undisclosed variations in subsurface conditions exist at the site and that seasonal variations in groundwater levels will occur. Although not observed at the site, it is common for seasonal seepage to develop at the interface between the surficial soils and the cemented soils at depth.

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4 CONCLUSIONS AND RECOMMENDATIONS

Based on our review of the field exploration and laboratory data collected to date, we believe that the project as currently proposed is feasible from a geotechnical standpoint, provided that the recommendations presented in this report are incorporated into plans for design and construction. The conclusions and recommendations that follow are based on design information provided by others, the results of our field and laboratory field investigations, our engineering analyses, and our professional judgment.

4.1.1. SEISMIC SHAKING, LIQUEAFACTION, AND DYNAMIC SETTLEMENT

4.1.2. Seismic Shaking

The principal effects of earthquakes on alluvial soil deposits in the site area are ground strong shaking and dynamic settlement. Based on the California Geological Survey (CGS) Seismic Hazards Assessment website, the project site lies in an area having an estimated peak horizontal ground acceleration of 0.18g.

4.1.3. Liquefaction Potential

Liquefaction describes a condition in which saturated soil loses shear strength and mobilizes as a result of increased pore pressure induced by strong ground shaking during an earthquake. The principal effects of liquefaction on structures are settlement (both total and differential), loss of foundation support, and for buried structures to rise buoyantly. Soils most susceptible to liquefaction are saturated, loose sandy soils and some granular silts. The potential for an earthquake with the intensity and duration characteristics capable of promoting liquefaction is a possibility during the design life of the project. However, given the relatively cohesive, dense/stiff, and or cemented nature of the soils underlying the site combined with the lack of groundwater in the upper 40 feet of soil, the potential for liquefaction is considered to be negligible.

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4.1.4. Dynamic Settlement

Loose, sandy soils or non-plastic silts above groundwater may consolidate or settle when subjected to cyclic application of loads such as ground shaking. Based on the soil conditions encountered and the methods recommended by Tokimatsu and Seed (1987), we estimate a 6.5 magnitude earthquake on a nearby fault (CRCV or FFS) generating a horizontal ground acceleration of 0.18g should induce negligible settlements in native soils, engineered fill, and trench backfill materials provided the recommendations contained in this report are followed.

4.2. EXPANSIVE SOILS

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 rainfall, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors and may cause unacceptable settlement or heave of structures, concrete slabs supported-on-grade, or pavements supported over these materials.

Based on the results of the field and laboratory investigations, near-surface soils encountered near the golf course fairways consist of highly-plastic clay that we consider to have high expansion potential. Near-surface soils from other portions of the site generally include lean clays and silts of low plasticity and are consequently judged to be of lower expansion potential. If potentially expansive soils are encountered in proposed building pad or pavement areas, mitigation alternatives could include excavation and removal of the expansive material and replacement with non-expansive soil or in-place stabilization through the addition of lime. Because we did not find evidence of wide-spread expansive soil deposits at the site, recommendations for expansive soil mitigation in this report are limited to removal and replacement with non-expansive materials. However, due to potential variability of the near-surface soil conditions, a Kleinfelder representative should be present during earthwork operations to look for potentially expansive soils and, if necessary, provide recommendations for mitigation.

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4.3. SOIL CORROSIVITY

Selected samples of the near-surface soils encountered at the site were subjected to chemical analysis for the purpose of corrosion assessment. The samples were tested for pH, resistivity, soluble sulfates, and soluble chlorides. The samples were tested in general accordance with California Test Methods 643, 422, and 417 for pH and resistivity, soluble chlorides, and soluble sulfates, respectively. The test results, as reported by Sunland Analytical, are provided in Appendix B on Plates B-5 through B-7. The test results are also summarized in Table 4.1, Summary of Corrosion Test Results.

Table 4.1
Summary of Corrosion Test Results

Boring No.	Sample Depth (feet)	Resistivity (Ohm-Cm)	pH	Water Soluble Sulfates (ppm)	Water Soluble Chlorides (ppm)
B-11	5.5	1,820	7.01	0.1	11.1
B-15	5.5	4,560	6.08	0.1	12.6
B-17	3.0	2,730	6.93	1.3	11.7

According to the 1997 Uniform Building Code, Table 19-A-4, a sulfate concentration below 0.10 percent by weight (1,000 ppm) is negligible. A water-soluble chloride content of less than 500 ppm is generally considered non-corrosive to reinforced concrete.

Minimum resistivity tests performed on soil samples indicated that the soils are considered to be moderately to severely corrosive to buried metal objects. A commonly accepted correlation between soil resistivity and corrosivity towards ferrous metals NACE (1984) is provided below:

Minimum Resistivity, ohm-cm	**Corrosion Potential**
0 to 1,000	- Severely Corrosive
1,000 to 2,000	- Corrosive
2,000 to 10,000	- Moderately Corrosive
Over 10,000	- Mildly Corrosive

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We have provided the above preliminary corrosion test results only as an indicator of potential soil corrosivity for the samples tested. Other soils found on the site may be more, less, or of a similar corrosive nature. In general, the corrosion potential for the soil samples tested are considered to be low to reinforced concrete, and corrosive to moderately corrosive to buried ferrous metals. Based on the above information, Type I/II portland cement should be adequate for concrete structures at the site.

Kleinfelder does not practice corrosion engineering. We recommend the project designer or corrosion engineer evaluate the corrosion potential of the site and provide specific corrosion mitigation methods appropriate for the project.

4.4. UBC SEISMIC DESIGN PARAMETERS

Structures should be designed for lateral force requirements as set forth in Section 1629.8 of the UBC (1997). Parameters for input to seismic modeling are provided on the basis of information contained in this report as follows:

Table 4.2
Seismic Design Parameters

Seismic Design Parameter	Reference	Symbol	Recommended Value
Seismic Source Type[1]	UBC Table 16-U	A-C	B
Near Source Factor[1]	UBC Table 16-S/T	Na, Nv	1.0, 1.0
Seismic Zone	UBC Figure 16-2	1-4	3
Seismic Factor	UBC Table 16-I	Z	0.30
Soil Profile type	UBC Table 16-J	S	S_C
Seismic Response Coefficient	UBC /Table 16-Q/R	Ca, Cv	0.33, 0.45

[1]Fault Activity Map of California and Adjacent Areas with Locations and Ages of Recent Volcanic Eruptions, California Division of Mines and Geology; Jennings, C.W. (1994).
[2]Probabalistic Seismic Hazard Map, Peak Horizontal Ground Acceleration, 10% Probability in 50 Years, Soft-Rock Site Condition, California Division of Mines and Geology (1996).

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4.5. SITE PREPARATION

4.5.1. Site Stripping and Grubbing

Site preparation should include the stripping and removal of existing vegetation, trees, clay pigeon mounds, debris, and other deleterious materials from the areas to be graded. We estimate the depth of stripping of surface vegetation in landscaped and/or undeveloped areas to be approximately 1 to 3 inches over a majority of the site. Deeper stripping or grubbing may be required where existing structures, concentrations of organic soils, or tree roots are encountered during site grading. Stripped topsoil (less any debris) may be stockpiled and reused for landscape purposes provided it is not contaminated with lead or other potentially hazardous material. Clay pigeon mounds may contain lead shot and are expected to be as deep as about 5 feet. We expect these materials will be removed from the site during hazardous material remediation operations. They should not be included in any engineered fill.

4.5.2. Existing Utilities, Wells, and/or Foundations

Abandoned utility lines, septic tanks, cesspools, wells, and/or foundations may exist on site. If present within the area of construction, these items should be removed and disposed of off-site. Existing wells should be abandoned in accordance with applicable regulatory requirements. Existing utility pipelines that extend beyond the limits of the proposed construction and that are to be abandoned in-place should be plugged with cement grout to prevent migration of soil and/or water. All excavations resulting from removal activities should be cleaned of loose or disturbed material (including <u>all</u> previously-placed backfill) and dish-shaped (with sides sloped 3(h): 1(v) or flatter) to permit access for compaction equipment.

4.5.3. Scarification and Compaction

Following site stripping and any required grubbing and/or over-excavation, we recommend all areas to receive engineered fill or to be used for the future support of structures or concrete slabs supported-on-grade be scarified to a depth of 8 inches, <u>uniformly</u> moisture-conditioned to between 2 and 5 percent above the optimum moisture content, and compacted to at least 90 percent of the maximum dry density as determined by ASTM (American Society for Testing and Materials) Test Method D

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1557[1]. The upper 12 inches of pavement subgrades should be scarified, moisture conditioned slightly above the optimum moisture content, and compacted to at least 95 percent relative compaction. Scarification and compaction may not be required within earthwork cut areas consisting of cemented soils if approved by the project Geotechnical Engineer during construction.

In-place scarification and compaction may not be adequate to densify all disturbed soil within areas grubbed or otherwise disturbed below a depth of about 8 inches. Therefore, over-excavation of disturbed soil, scarification and compaction of the exposed subgrade, and replacement with engineered fill may be required to sufficiently densify all disturbed soil.

4.6. WET WEATHER CONSTRUCTION/UNSTABLE SOIL CONDITIONS

Should grading be performed during or following extended periods of rainfall, the moisture content of the near-surface soils may be significantly above the optimum moisture content. Perched groundwater may also develop above dense, cemented on-site soils, saturating near-surface materials. This condition could seriously impede grading by causing an unstable subgrade condition. Typical remedial measures include the following:

- Dry – Drying the unstable subgrade would involve disking or ripping the wet subgrade to a depth of approximately 24 inches and allowing the exposed soil to dry. Multiple passes of the equipment (likely on a daily basis) will be needed because as the surface of the soil dries, a crust forms that reduces further evaporation. Frequent disking will help prevent the formation of a crust and will promote drying. This process could take several days to several weeks depending on the depth of ripping, the number of passes, and the weather.

- Remove & Replace plus Geotextile Fabric – The subgrade could be overexcavated 12 to 24 inches below existing grade and be replaced with aggregate base or coarse gravel underlain by geotextile fabric. The geotextile fabric should be Amoco 2006 (woven) or approved equivalent. The final depth of removal will depend upon field conditions revealed once the over-excavation

[1] *This test procedure should be used wherever relative compaction, maximum dry density, or optimum moisture content is referenced within this report.*

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begins. The geotextile fabric should be placed in accordance with the manufacturer's recommendations.

- Lime Treat – The unstable subgrade could be stabilized by mixing the upper 12 to 18 inches of the subgrade with lime. For estimating purposes, lime application rates of 3 to 4 percent for high calcium quick lime and 4 to 6 percent for dolomitic lime may be used. Final application rates should be determined in the field at the time of construction in consultation with the project geotechnical engineer. Lime treatment should be performed by a specialty contractor experienced in this work and should be performed in accordance with Caltrans Standard Specifications. Thus, since lime can be used with on-site soil, the expense of importing material can be avoided. Lime treated areas will have a high pH level (pH over 10) that will need to be removed from landscape areas.

Our firm should be consulted prior to implementing any remedial measure to observe the subgrade conditions and provide site-specific recommendations.

4.7. ENGINEERED FILL

4.7.1. Native Soils

The near-surface, on-site soils consisted predominantly of silts, lean clays, and clayey sands. These soils may be reused as engineered fill provided that strict moisture control is maintained during and following placement and compaction. Where potentially expansive fat clay soils are encountered during construction, they should not be placed within the building pad areas. If the on-site soils are used as engineered fill, these soils should be placed and compacted in accordance with the recommendations provided in Section 4.7.3, Compaction Requirements. Furthermore, oversized material (greater than 3 inches in maximum dimension) should not be included in any engineered fill that will support future structural loads.

4.7.2. Imported Non-Expansive Fill

Imported soils may be required to achieve the proposed site grades. All imported non-expansive engineered fill soils should be nearly-free of organic or other deleterious

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debris, essentially non-plastic, and less than 3 inches in maximum dimension. In general, well-graded mixtures of gravel, sand, non-plastic silt, and small quantities of cobbles, rock fragments, and/or clay are generally acceptable for use as engineered fill. Specific requirements for non-expansive engineered fill, as well as applicable test procedures to verify material suitability are provided in Table 4.3 below.

Table 4.3
Imported Non-Expansive Engineered Fill Requirements

Fill Requirement		Test Procedures	
		ASTM[1]	**Caltrans**[2]
Gradation			
Sieve Size	**Percent Passing**		
3 inch	100	D 422	202
¾ inch	70-100	D 422	202
No. 4	50-100	D 422	202
No. 200	20-70	D 422	202
Plasticity			
Liquid Limit	**Plasticity Index**		
<30	<12	D 4318	204
Organic Content			
Less than 3%		D 2974	---
Expansion Potential (UBC 18-2)			
Less than 20	---	---	---

[1] *American Society for Testing and Materials Standards (latest edition)*
[2] *State of California, Department of Transportation, Standard Test Methods (latest edition)*

All imported fill materials to be used for engineered fill should be sampled and tested by Kleinfelder prior to being transported to the site.

4.7.3. Compaction Requirements

4.7.3.1. On-Site Soils

On-site soils used for engineered fill should be uniformly moisture-conditioned to between 2 and 5 percent above the optimum moisture content, placed in horizontal lifts

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less than 8 inches in loose thickness, and compacted to at least 90 percent relative compaction as determined by ASTM (American Society for Testing and Materials) Test Method D 1557[1]. The upper twelve inches of pavement subgrades should be compacted to at least 95 percent relative compaction at a moisture content slightly above optimum.

4.7.3.2. Imported Non-expansive Engineered Fill

Imported non-expansive soils used for engineered fill should be uniformly moisture-conditioned to between 0 and 5 percent above the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent relative compaction as determined by ASTM (American Society for Testing and Materials) Test Method D 1557[1]. The upper twelve inches of pavement subgrades constructed in these materials should be compacted to at least 95 percent relative compaction at a moisture content slightly above optimum.

4.7.3.3. Deep Fills

All fills exceeding 5 feet in thickness should be compacted to at least 95 percent relative compaction to within 5 feet of finished subgrade. Disking and/or blending may be required to uniformly moisture-condition soils used for engineered fill.

4.8. TRENCH PREPARATION AND BACKFILL

4.8.1. General Considerations

The materials encountered in the borings at the elevations of the proposed trench bottoms generally consist of stiff to very hard silts and lean clays, and dense to very dense silty and clayey sands. In general, these materials appear suitable for pipeline support.

4.8.2. Subgrade Preparation

Prior to placement of bedding, the exposed subgrade at the bottoms of trench excavations should be examined to detect soft, loose, or unstable areas. Loose materials at trench bottoms resulting from excavation disturbance should be removed to

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firm material. If soft or unstable areas are encountered, these areas should be over-excavated to a depth of at least 2 feet or to a firm base and be replaced with additional bedding material. Where excavations cross existing trench backfill materials, the need for and extent of over-excavation or stabilization measures should be evaluated by the Geotechnical Engineer on an individual basis. Where clean crushed rock bedding materials are specified, the bedding material should be surrounded by a non-woven filter fabric (see Section 4.8.5, Filter Fabric Envelope) to prevent migration of fines into the bedding layer.

4.8.3. Pipe Bedding and Initial Backfill Materials

Pipe bedding and initial backfill should be appropriate for the types of pipelines to be installed. We anticipate clean washed sand and/or crushed rock will be used for pipe bedding and initial backfill. Pipe bedding and initial backfill requirements may be specified by the Owner or pipe manufacturers based on planned pipe types, bedding conditions, and other factors beyond the scope of this study. Accordingly, the project Civil Engineer should develop final project specifications and details.

If clean crushed rock is used for pipe bedding and initial backfill, we recommend it have a maximum particle size less than 1 inch and have less than 5 percent passing the No. 200 U.S. sieve. Where crushed rock is used, the material should be completely surrounded by a non-woven filter fabric such as Mirafi 140N, Propex 4547 (formerly Amoco) or equal. Where washed sand is used for pipe bedding and initial backfill, a filter fabric wrap is not necessary.

We recommend that care be taken to place the initial backfill in such a manner that it will be in intimate contact with the sides of the pipeline. The prepared pipe bedding may be pre-shaped by means of a template prior to placement of the pipeline. This preparation will help distribute the upward reaction on the pipe bottom over the width of the bedding contact. Techniques such as shovel slicing of the bedding under the pipe haunches can also help to provide more uniform support under the pipe.

4.8.4. Compaction of Bedding and Initial Backfill Materials

Where pipe bedding and initial backfill consists of clean crushed rock, compaction testing by conventional methods may not be practical. Crushed rock bedding and initial

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backfill materials should be placed in lifts (appropriate thickness for compaction equipment used) with mechanical compactive effort applied until no further consolidation of the material occurs. We recommend the lift thickness for compaction not exceed 1 foot. Clean washed sands should be placed in a manner to completely surround the pipe and minimize voids. They should also be mechanically compacted to at least 90 percent relative compaction. Conventional compaction testing may not be practical in a narrow trench. In these cases we recommend the adequacy of compaction of washed sand bedding and initial backfill materials be verified by observation of the moisture conditioning and compaction operations.

4.8.5. Filter Fabric Envelope

To reduce the potential for migration of the fine-grained native soils and intermediate backfill into crushed rock bedding and initial backfill, a filter fabric should be placed between native or fill soils and the crushed rock creating an envelope around the rock. Filter fabric should be laid-out and overlapped according to the manufacturer's recommendations. Recommended minimum filter fabric specifications are presented in Table 4.4 below.

Table 4.4
Recommended Filter Fabric Specifications

Property	Requirement	Test Method
Apparent Opening Size (AOS)	#100 U.S. Standard Sieve Size	ASTM D4751
Grab Tensile/Elongation	200 lbs./50%	ASTM D4632
Puncture Strength	120 lb. Minimum, Average Roll Value	ASTM D4833

Where washed sand, concrete slurry or Controlled Density Fill (CDF) material is used for bedding and initial backfill, the filter fabric wrap is not necessary.

4.8.6. Intermediate Trench Backfill Materials

Intermediate trench backfill (i.e., the material placed above the initial backfill) should consist of on-site soils or approved imported fill material that meets the requirements for engineered fill presented herein.

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4.8.7. Intermediate Trench Backfill Compaction Criteria

Intermediate trench backfill should be uniformly moisture-conditioned to between 2 and 5 percent above the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent relative compaction based on the ASTM D1557 test method. The upper twelve inches of pavement subgrades should be compacted to at least 95 percent relative compaction. Fills exceeding 5 feet in thickness should be compacted to at least 95 percent relative compaction for their full depth at a moisture content slightly above optimum. Additional fill lifts of fill should not be placed if the previous lift did not meet the required relative compaction or if soil conditions are not stable. Thorough mixing, aeration, watering and/or blending may be required to uniformly moisture-condition soils used for intermediate trench backfill. Ponding or jetting compaction methods should not be allowed. We do not recommend allowing the Contractor to place and compact materials in unshored trenches using remote equipment. Full access to testing personnel should be provided during backfilling.

4.9. TEMPORARY EXCAVATIONS

4.9.1. General

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.

4.9.2. Excavations and Slopes

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). Such regulations are

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strictly enforced and, if they are not followed, the Owner, Contractor, and/or earthwork and utility subcontractors could be liable for substantial penalties.

4.9.3. Construction Considerations

Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within ⅓ the slope height from the top of any excavation. Where the stability of adjoining buildings, walls, or other structures is endangered by excavation operations, support systems such as shoring, bracing, or underpinning may be required to provide structural stability and to protect personnel working within the excavation. Shoring, bracing, or underpinning required for the project (if any) should be designed by a professional engineer registered in the State of California.

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 the excavation(s) should be collected and disposed of outside the construction limits.

4.10. SHORING

4.10.1. General Recommendations

We anticipate shoring will be required for relatively deep pipeline trenches and bore pits for the trenchless crossing beneath the Interstate 80 Business Route freeway. Shoring system design and installation on this project should be the responsibility of the Contractor. Shoring systems should be designed by a California registered Civil Engineer based on the conditions exposed in the areas of excavation. Excavation of test pits along the proposed pipeline alignments is recommended to assist the shoring designer in evaluating the appropriate shoring systems for the project.

Trench shields, speed shoring, soldier piles, Slide Rail™, or other forms of shoring may be used where appropriate throughout the project provided Cal OSHA regulations are met. If soldier piles are to be used, continuous lagging should be provided to retain any potentially caving cohesionless materials. Sheet piles are not well suited to driving in areas underlain by very stiff to hard and/or cemented soils such as those encountered at the site. Where trenches are excavated in existing roadway areas or near existing

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structures, we recommend shoring systems be designed to provide positive restraint of trench walls. Where positive restraint of trench walls is not provided, lateral deformation of the trench walls may result in ground cracks, settlement and/or other ground movements that may affect adjacent underground utilities as well as surface improvements. If trench walls deflect laterally in pavement areas, parallel cracks may develop in the pavement and underlying soils that may require repair. The contractors should be made aware of this potential condition so that preventative measures can be implemented or repair measures provided for.

4.10.2. Lateral Deflection of Shoring Systems

Lateral deflection of the shored excavation will depend on the relative stiffness of the shoring system selected and mobilization of the active earth pressure. The limiting condition of maximum active earth pressure for stiff to hard silts and clays is generally reached when the shoring tilts or deflects laterally about 1 percent of the shoring wall height. The limiting condition of active earth pressure for dense sands and gravels is generally reached when the shoring tilts or deflects laterally about 0.1 percent of the shoring wall height. If the shoring tilts or deflects less than the limiting condition, the lateral earth pressure should be between the active and at-rest earth pressures. This soil movement has been observed extending horizontally as far back as 2H from the top of cantilever retaining structures with vertical movements approximately equal to the horizontal. The movement tends to be greatest close to the excavation and becomes less with increasing distance.

Shoring systems either incapable of deflection or which are fully constrained against deflection should be designed for at-rest earth pressure conditions.

4.10.3. Lateral Resistance

All soldier piles and other systems employing passive resistance should extend to a sufficient depth below the excavation bottom to provide the required lateral resistance. Embedment depths should be determined using methods for evaluating sheet pile walls and based on the principles of force and moment equilibrium. To account for three-dimensional effects, the passive pressure may be assumed to act on an area 2 times the width of the embedded portion of the soldier pile, provided adjacent piles are spaced at least 3 diameters, center-to-center. A minimum factor of safety of 1.2 should

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be applied to the calculated embedment depth and to determine the allowable passive pressure. The shoring professional should evaluate the final design conditions and shoring type to select the appropriate factor of safety for design.

Alternatively, lateral capacity of a soldier pile extending below the excavation bottom may be evaluated using the "Pole Formula" given in Section 1806.8, Chapter 18 of the Uniform Building Code, 1997 edition. The 100 percent increase allowed by the Code for isolated poles (which are not adversely affected by a 1/2-inch horizontal deflection at the ground surface due to short-term lateral loads) may be used.

The passive earth pressure, similar to active earth pressures, is mobilized when the shoring below the excavation bottom tilts or deflects laterally. For stiff to hard silts and clays, and dense sand conditions, the limiting condition of maximum passive earth pressure is generally reached when the shoring deflects laterally below the base of the excavation about 2 percent of the shoring wall height. If the shoring system is restrained against movement, the lateral resistance below the base of the excavation should be somewhere between the passive and at-rest earth pressure conditions. Accordingly, if lateral deflection at the base of the excavation is objectionable, the at-rest earth pressure should be used in design for lateral resistance.

4.10.4. Surcharge Pressures

Shoring systems should be designed to resist loads imposed by construction equipment, stockpiles, adjacent foundations, or roadways. Lateral forces due to areal surcharges (such as stockpiled soil, equipment, etc.) placed adjacent to the shoring may be assumed to act as a uniform horizontal pressure against the shoring.

4.10.5. Protection of Existing Facilities

The shoring designer should perform a deflection analysis for the proposed shoring system. A survey of existing utilities, pavements, and structures adjacent to those portions of the proposed excavation that will be shored should also be performed prior to construction. The purpose of the analysis and survey would be to evaluate the ability of existing structures, pipelines, or conduits to withstand anticipated horizontal and vertical movements associated with a shored excavation. If movements are greater than the tolerance of existing project features (utilities, pavements, structures, etc.),

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alternative shoring systems employing the at-rest earth pressure, tie-backs, dead-man anchors, or cross bracing may be needed to reduce deflections to acceptable levels. The Contractor should anticipate repairing cracks in pavements adjacent to shored excavations due to anticipated lateral displacements of the shoring system.

In areas where new excavations will encroach upon or pass under existing utilities, stabilization of these utilities and backfill materials may be necessary. Alternatives for stabilization include shoring or bracing systems, and various in-situ compaction or permeation grouting methods. Where pressure sensitive utility lines are present, pressure grouting methods may not be applicable. The stabilization method chosen for support of adjacent utilities (and backfilling) should be determined based on a thorough review of existing conditions and with the approval of the utility owner.

We recommend the proposed shoring system design be submitted (along with the appropriate design calculations) in advance for review by the design team. The purpose of the review would be to evaluate whether the proper soil parameters have been used and the anticipated shoring deflections are within the tolerance established by the owners of adjacent improvements that may be affected by nearby trench installations.

4.10.6. Monitoring

Horizontal and vertical movements of the shoring system should be monitored by establishing survey points, installation of inclinometers, or a combination of both prior to excavation. The results should be reviewed by a qualified Geotechnical Engineer on a daily basis for a period of at least one week during excavation and following construction of the shoring system. Measurements should be obtained on a weekly basis thereafter. Detailed recommendations for monitoring should be provided by a qualified Geotechnical Engineer after a review of the planned shoring system.

4.10.7. Construction Vibrations

The Contractor should use means and methods that will limit vibrations at the locations adjacent structures/facilities. Where construction operations such as sheet pile driving demolition, or similar activities induce significant ground vibrations near critical facilities we recommend vibration monitoring be performed. As a guide, peak particle velocities

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from construction vibrations within adjacent structures/facilities should be limited to less than about 1 inch/second when measured using an accelerometer. More stringent requirements may be needed adjacent to historic structures, buildings in poor condition, or buildings where vibration sensitive equipment is being operated. We suggest the need for vibration monitoring be evaluated on a case-by-case basis.

4.10.8. Shoring Removal

Shoring systems typically are removed as part of the trench backfill process. Depending on the shoring system used, the removal process may create voids along the sides of the trench excavation. If these voids are left in place and are significantly large, backfill may shift laterally into the voids resulting in settlement of the backfill and overlying pavements. Therefore, care should be taken to remove the shoring system and backfill the trench in such a way as to not create these voids. If the shoring system requires removal after backfill is in place, resulting voids should be filled with sand and cement slurry or other approved grout mix. If shoring cannot be removed without causing voids and/or disturbing pipes or structures, the shoring should be cut off above the pipe or structure and be left in place. Timber lagging to be left in place should be pressure treated.

4.11. PIPELINE DESIGN CRITERIA

The following paragraphs present geotechnical design recommendations for evaluation of soil loads on buried rigid and flexible pipes. Recommendations for design and construction of pipelines, thrust blocks, and appurtenances installed by open cut excavation and trenchless methods are presented in the following sections of this report.

4.11.1. Soil Loads on Buried Flexible Pipes

The pipe loading pressure for flexible pipes such as PVC, HDPE, or welded steel may be determined by calculating the soil overburden and live load pressures and multiplying by the pressure transfer coefficient C_p. The coefficient C_p typically varies from 0.65 to 2.0 depending on the type and degree of compaction of the bedding and initial backfill materials. The value of C_p may be determined from the pipe manufacturers or may be conservatively estimated as $C_p = 2.0$. For crushed rock or

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clean washed sand bedding and initial backfill materials compacted as recommended in this report, a C_p value of 0.80 is recommended for design.

4.11.2. Design Values for Buried Flexible Pipes

Flexible pipes typically derive part of their resistance to ring deflection from the initial backfill and trench wall soils. Evaluation of ring deflection of buried pipes under soil and live loads may be determined using the Iowa Formula. The elastic modulus of the soils surrounding the pipe, or E', may be evaluated by knowing the trench width, the pipe diameter, the elastic modulus of the initial backfill (E'_b), and the elastic modulus of the native trench wall soils (E'_n - also termed Constrained Modulus). Recommendations for pipe design using the Iowa Formula are presented in Table 4.5.

Table 4.5 presents recommended E'_b values for use in the Iowa Formula for proposed initial backfill materials placed and compacted in accordance with our recommendations. The value of E'_b is a lateral modulus of subgrade reaction for the initial backfill material. For E'_b values at depths between the intervals presented below, the E'_b value between data points may be determined by linear interpolation.

The recommended E'_b values presented in Table 4.5 apply to clean crushed rock or washed sand bedding and initial backfill material along the sides of the pipe at the recommended level of compaction. These values are applicable for pipe design where the initial backfill width is at least 2 times the pipe diameter (D) on each side of the pipe (trench width of 5D).

Table 4.5
E'b Values for Design of Buried Flexible Pipes

Soil Type	Depth to Springline of Pipe (ft)	Recommended E'_b (psi)
Pipe Bedding and Initial Backfill (clean crushed rock or washed sand)	5 10 15	1000 1500 1600

Notes:
1. The above design values are based on "Evaluation of the Modulus of Soil Reaction, E', and its Variation With Depth," by Hartley & Duncan, dated June 1982.
2. Based on providing at least 2 pipe diameters of backfill on each side of pipes.

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Where the zone of backfill beside the pipe is less than 2D, the E'_b values presented above may not be applicable and the constrained soil modulus E'_n will affect flexible pipe design. The actual lateral soil modulus at the pipe depth will lie somewhere between E'_b and E'_n depending on the trench width.

Based on the field and laboratory data obtained along the pipeline alignments, we recommend an E'_n value of 3,000 psi (Howard, 1996) be used for design of flexible pipes. This value is applicable to the undisturbed native soils encountered at the site.

For trench widths less than 5D, the design E' may be calculated by multiplying E'_b by the Soil Support Combining Factors (S_c) presented in Table 4.6, where B_d is the trench width at pipe springline and D is the diameter of the pipe.

$$\text{Design } E' = S_c(E'_b)$$

Table 4.6
Sc Values For Design of Buried Flexible Pipes
(Soil Support Combining Factor)

E'_n/E'_b	B_d /D 1.5	B_d /D 2.0	B_d /D 2.5	B_d /D 3.0	B_d /D 4.0	B_d /D 5.0
0.1	0.15	0.30	0.60	0.80	0.90	1.00
0.2	0.30	0.45	0.70	0.85	0.92	1.00
0.4	0.50	0.60	0.80	0.90	0.95	1.00
0.6	0.70	0.80	0.90	0.95	1.00	1.00
0.8	0.85	0.90	0.95	0.98	1.00	1.00
1.0	1.00	1.00	1.00	1.00	1.00	1.00
1.5	1.30	1.15	1.10	1.05	1.00	1.00
2.0	1.50	1.30	1.15	1.10	1.05	1.00
3.0	1.75	1.45	1.30	1.20	1.08	1.00
5.0	2.00	1.60	1.40	1.25	1.10	1.00

Source: "Pipeline Installation," A. Howard, 1996.

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4.11.3. Flexible Pipe Trench Width Recommendations

According to ASTM D 2321, ”Standard Practice for Underground Installation of Thermoplastic Pipes for Sewers and other Gravity-Flow Applications”, the minimum trench width for flexible pipes should be the greater of 16 inches greater than the pipe diameter or 1.25 times the pipe diameter plus 12 inches. For flexible pipes, the trench width should be kept to a minimum to reduce the soil loading on the pipes. Wider trenches will generally impart higher soil loads on buried flexible pipes. Where granular pipe zone backfill is used, the trench should be wide enough to accommodate compaction equipment and shoring along the sides of the pipe. Care should be taken during installation of the pipe zone backfill around the haunches of the pipe (i.e., from the bottom of the pipe to springline) such that voids are eliminated and the backfill material is firm and unyielding. Lateral restraint against ring deflection for the pipes will be provided by the stiffness of the pipe zone backfill material and/or the trench wall soils.

4.11.4. Flexible Pipe Construction Considerations

Flexible pipes require uniform support from bedding materials especially in haunch areas to prevent overloading. The pipeline designers should evaluate the proximity of adjacent pipelines, excavations, and their related effects on the proposed construction. If proper trench wall support cannot be provided in a portion of the pipe trench, we recommend consideration be given to the use of lean concrete or Controlled Low Strength Material (CLSM) initial backfill around the pipes. In general we recommend lean concrete or CLSM materials have a 28-day compressive strength between 50 and 200 psi.

4.11.5. Soil Loads on Buried Rigid Pipes

The soil dead load (W_D) on a rigid conduit in a trench may be evaluated using Marston’s Formula as the following expression $W_c = C_d \gamma B_d^2$ where:

C_d = Load Coefficient based on Kμ’
γ = Moist unit weight of backfill material (pcf)
B_d = Width of trench at top of pipe (ft)

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Design of the pipelines should include evaluation of the dead loads imposed by the initial and intermediate trench backfill. The parameter C_d will depend on 1) the backfill type, which in turn will depend on the pipe manufacturer's recommendations; 2) the trench width; and 3) the installation depth. For a trench installation with the ratio of the backfill depth to trench width at the top of pipe (H/B_d) of at least 1 and for a trench width at top of pipe no greater than 3 times the pipe diameter, the value of C_d may be calculated using the following equation (American Concrete Pipe Association [ACPA], 2000).

$$C_d = \frac{1 - e^{-2K\mu' \frac{H}{B_d}}}{2K\mu'}$$

where K = Active earth pressure coefficient

μ' = friction coefficient between fill material and sides of trench and

H = backfill height above the pipe crown.

The product Kμ' is dependent on the backfill type, degree of compaction, and moisture content. Where backfill materials are compacted as recommended in this report, the estimated Kμ' values shown in Table 4.7 are applicable for the various soil types encountered during our investigation and those anticipated to be used for trench backfill.

Table 4.7
Estimated Kμ' Values for Pipe Design

Soil Type	Ku'
Clay (CL, CH)	0.12
Silt (ML)	0.13
Silty Sands (SM)	0.15
Well and Poorly Graded "Clean" Sands (SW, SP)	0.165
Sandy Gravels and Cobbles (GW, GP, GM, GC)	0.18
Reference: ASCE (1982)	

Based on the materials encountered in the borings, we recommend using a Kμ' value of 0.130 for design. For analysis of trench backfill loads using Marston's formula, we

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recommend using a wet soil unit weight, γ, equal to 125 pcf for initial and intermediate backfill.

4.11.6. Rigid Pipe Trench Width Recommendations

Trench widths for buried rigid pipelines should be determined based on the pipe manufacturer's recommendations and on the ability to properly place pipe zone backfill materials such that the potential for voids within the backfill is minimized. In general, we recommend trench widths be selected to allow the space necessary for compaction equipment and shoring on either side of the pipe. Wider spaces should be considered where shovel slicing of bedding materials is necessary in pipe haunch areas and where shoring interferes with needed working space along side the pipes. Trenches that are over widened during construction will likely cause soil loadings on the pipeline to increase. The increased load can cause damage to the pipeline. The designers and Contractor should be aware of this condition and select an appropriate trench width for construction.

4.11.7. Live Loads On Buried Conduits

Live loads on buried conduits due to vehicular loadings may be determined from Plate 6.

4.11.8. Design Values for Steel Pipes Installed by Trenchless Methods

For a steel pipe installed by a trenchless method (e.g., horizontal auger boring or pipe jacking), the "dead load" vertical soil pressure may be evaluated using the procedure outlined by the American Lifelines Alliance, Guidelines for the Design of Buried Steel Pipe (2001). This procedure is presented below:

$$P_{vu} = P_v - 2c(C/D)$$

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where P_{vu} = Vertical earth load pressure for undisturbed placement conditions

P_v = Earth dead load pressure on the conduit (γC)

γ = Unit weight of soil

c = Soil cohesion

C = Height of fill above pipe

D = Pipe outer diameter

For a steel pipe installed under the freeway between Borings B-1 and B-2 at a depth between 5 and 15 feet below existing grade, we recommend a cohesion factor of c = 800 psf and a total unit weight of γ = 125 pcf. The cohesion factor includes a safety factor of about 2 with respect to the estimated ultimate value.

4.12. PIPELINE THRUST BLOCKS

Thrust blocks for the buried pipelines may be designed using an allowable lateral bearing pressure of 1,200 psf at a minimum depth of 3 feet into firm native soil or engineered fill. This value may be increased by 400 psf for each additional foot of depth beyond the recommended minimum, up to a maximum value of 4,000 psf. For evaluation during temporary test cases, these values may be increased by a factor of 1.5.

4.13. TRENCHLESS PIPELINE CROSSING

The following sections present our recommendations for the trenchless pipeline crossing, including suitable installation methods and shoring of the bore pits.

4.13.1. Installation Methods

Selection of an appropriate trenchless installation technique will depend on a number of factors including pipe material type and size, length and depth of run, subsurface conditions, space constraints in the site vicinity, and environmental factors. We understand the water pipeline under the freeway will be a 16-inch ductile iron pipe installed in a steel casing that will be at least 5 feet deep. The proposed length of the

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trenchless crossing is approximately 250 feet. The subsurface conditions for the trenchless crossing, as revealed by Borings B-1 and B-2, generally consist of about 5 feet of lean clay over hard, dry to moist, and moderately cemented silt. Groundwater was not encountered in Borings B-1 and B-2.

4.13.1.1. Horizontal Auger Boring

Based on the expected subsurface conditions, the proposed crossing length and depth, and the proposed pipe material type, horizontal auger boring methods could be considered as an appropriate trenchless installation technique for the cemented silt and silty sand soils encountered at this site. Other trenchless methods, such as microtunneling or pipe jacking, require larger, heavier equipment and generally are better suited for larger pipe sizes than are contemplated for this project.

Horizontal auger boring methods are non-steerable and require the excavation of entrance and exit pits at each end of the crossing. The pits are excavated to the proposed pipe elevation. From the entrance pit, the pipe segments are jacked horizontally into place while continuous-flight augers simultaneously remove the drill cuttings. The auger is driven by an external power source in the entrance pit, which in turn powers the cutting head. As the auger advances, pipe segments are added until the auger reaches the exit pit. A typical entrance pit is rectangular with a length of 20 to 40 feet. The exit pit can be shorter. Pit size varies depending on the contractor's equipment manufacturer and space constraints.

Depending on the skill and experience of the contractor, horizontal auger borings can usually be installed to within allowable tolerances and without supplemental guidance systems to about 100 to 200-foot lengths. Beyond these lengths, unguided horizontal borings tend to wander off-course. Changes in the stiffness of the soils along the bore path are common in cemented soil deposits such as those encountered at the site. This may also lead to vertical or horizontal misalignment of the bore. If unguided horizontal auger boring is used, provisions should be made to accommodate potential variations in the exit point. Consideration should be given to equipment torque requirements since SPT N values at the site are generally near or above 100 blows per foot.

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Specialized methods are available that utilize a guided horizontal pilot hole prior to auger boring to facilitate borehole alignment. Horizontal directional drilling (HDD) equipment can be used to drill a pilot hole between entry and exit pits prior to auger boring. The process involves placing special HDD drilling equipment within the entry pit and drilling horizontally. Drilling fluid would need be recovered from the exit pit upon completion of the bore using a vacuum truck or other appropriate means. If this method is to be considered, please contact our office for additional information.

4.13.1.2. Pilot Tube or Guided Auger Boring

The precision of a horizontal auger boring installation may be improved through use of a guided boring machine (GBM). The GBM employs a laser-guided steering/cutting head, which is used to install small-diameter pilot-pipe joints from the entrance pit to the exit pit. Next, the pilot hole is reamed with larger cutter, and the product pipeline is jacked into place. The GBM method typically is applicable for pipe sizes up to about 24 inches. GBM methods generally do not work well in cemented soils such as those encountered at the site due to pilot tube penetration and equipment torque limitations. The equipment has the ability to be retracted from the bore, which is desirable where obstructions or drill refusal is likely. Recent GBM projects using 24-inch diameter equipment in cemented soils have encountered refusal at bore lengths between about 70 and 120 feet. It is often possible to drill from each pit (two headings) to connect the bore.

4.13.1.3. Microtunneling

Microtunnel boring machines (MTBM's) equipped to excavate cemented materials may be applicable for the proposed crossing. Microtunneling machines are steerable and can better hold line and grade tolerances that unguided equipment. However, the bore diameter may need to be larger as a result of power and torque requirements required to excavate cemented soils.

Similar to horizontal auger boring, microtunneling requires the excavation of entrance and exit pits at each end of the crossing. The pits are excavated to the proposed pipe elevation. From the entrance pit, the pipe segments are added and hydraulically jacked horizontally into place behind the MTBM to advance the machine. The excavated materials are conveyed through the MTBM and jacked pipes and are removed at the

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entry pit. The MTBM cutting head is driven by an external power source typically located outside of the entrance pit. Spoil removal methods may employ auger, conveyor, or slurry systems. Slurry systems typically require more space as a slurry/spoil separation system is needed. A typical entrance pit is rectangular with a length of about 20 to 40 feet. The exit pit can often be shorter. Pit size varies depending on the contractor's equipment manufacturer and space constraints.

4.13.1.4. Horizontal Directional Drilling

Conventional Horizontal Directional Drilling (HDD) is generally be performed without entry or exit pits. The process uses guided mud rotary drilling methods to drill a hole and then pull a pipe string back through the hole. The hole typically starts at a slight downward angle and then bends horizontally and to a slight upward angle toward the exit point. A laydown area of sufficient length to lay out the pipe section to be pulled in is required on the exit pit side of the bore. Conventional HDD does not appear practical due to the diameter and relatively short length of the bore as well as space constraints along Auburn Boulevard. We estimate the minimum bore length required for a 24- to 30-inch diameter bore would be greater than 1,000 feet unless an exit pit and pipe pushing system were employed. Such pipe pushing systems are highly specialized and may or may not be appropriate for this project. An experienced HDD contractor should be contacted to evaluate the feasibility of such a method.

4.13.2. Bore Pit Shoring

Where horizontal auger boring or microtunneling methods are used, relatively deep excavations will be required to construct the entry and exit pits. We expect shoring systems will be required to construct the pit along Auburn Boulevard. Due to the relatively hard and cemented soil conditions present at this location, drilled and/or internally braced shoring systems appear most suitable. Driven shoring systems such as sheet piles may not be feasible and are not recommended. The side walls of the pit along the north side of Interstate 80 could be sloped.

Although not encountered in our borings, it is common for perched groundwater to occur in the area and require removal from excavations due to seasonal infiltration and landscape watering. Provisions should be made to collect and remove groundwater seepage that may accumulate in the excavations.

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Recommendations for shoring are provided in Section 4.10, Shoring, of this report. We recommend that Kleinfelder be given the opportunity to review proposed excavation and shoring designs for applicability and consistency within the information contained in this report.

4.14. STORM WATER DETENTION BASIN

4.14.1. Soil Conditions

Borings B-6, B-11, and B-15 were drilled in the vicinity of the storm water detention basin. Within the upper 8 feet of soil the borings encountered cemented silts overlying silty to poorly graded sands and clayey sands overlying cemented silts.

4.14.2. Infiltration

Although infiltration tests were not performed as part of this study, based on our experience we expect the clayey sand and cemented silt soils will have low to very low permeability. The silty sands encountered are also expected to have low permeability due to their natural cementation. Although cemented, the poorly graded sands with silt are expected to be slightly more permeable in the horizontal direction however, in the vertical direction they are underlain by cemented silts with very low permeability. We, therefore, conclude the detention basin infiltration rates should be very slow.

4.14.3. Slopes

In general, we recommend slopes for the storm water detention basin be graded no steeper than 3(h):1(v). However, slopes within the cemented silt soils should be stable at gradients as steep as about 2(h):1(v). Erosion protection should be provided for the basin side slopes.

4.15. FOUNDATION DESIGN RECOMMENDATIONS

Foundations should satisfy two independent criteria with respect to foundation soils. First, the foundation should have an adequate safety factor against bearing failure with respect to shear strength of the foundation soils. Second, the vertical movements of the foundation due to settlement of the foundation soils should be within tolerable limits.

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According to preliminary grading plans provided by Wood Rodgers, the automobile dealership buildings will be supported on fill and/or native materials that will vary across the building footprints. To reduce the potential for adverse foundation performance under these conditions, the subgrade preparation and engineered fill material selection and compaction recommendations presented in this report (Section 4.5, Site Preparation, and Section 4.7, Engineered Fill) should be followed. The following paragraphs present recommendations for shallow spread and strip footings. Recommendations for design and construction of concrete floor slabs are discussed in Section 4.10, Concrete Slab-on-grade Floors.

4.15.1. Foundation Type and Depth

We recommend the proposed structures be supported on shallow foundation systems such as spread and strip footings, with the floors supported at-grade by reinforced concrete slabs. We recommend shallow spread and strip footings be installed using a minimum embedment of 18 inches below lowest adjacent building pad. The thickness of the floor slab and any underlayment should not be considered in design.

Stresses imposed by footings on buried utility lines may cause excessive cracking, collapse and/or a loss of serviceability. To reduce these risks, footings should extend below a 2(h):1(v) plane projected upward from the closest bottom edge of the adjacent utility trench or foundations.

4.15.2. Allowable Net Bearing Capacity

For shallow spread and strip footings installed at the recommended depth on firm, unyielding native soils or engineered fill, we recommend an allowable net bearing capacity of 3,000 pounds per square foot (psf). A one-third increase may be applied to this value when evaluating the effects of transient loads such as wind or seismic. The net allowable bearing capacity includes a safety factor of at least 3 with respect to shear failure of the foundation soils. Continuous strip footings should be at least 18 inches wide, and spread footings should have a least lateral dimension no smaller than 24 inches.

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4.15.3. Lateral Load Resistance

Lateral loads may be resisted by a combination of friction at the foundation base and by passive resistance acting against the vertical faces of foundation elements. A friction coefficient of 0.35 between the foundation and the underlying soil may be used. For passive resistance, an equivalent fluid pressure based on a fluid weight of 400 pounds per cubic foot (pcf) acting against the footing may be used. Passive resistance in the upper 12 inches should be neglected unless the area adjacent to the footing is covered by concrete or pavement. The friction and passive resistance may be used concurrently provided the passive resistance is reduced by 50 percent. These values may be increased by one-third when considering temporary wind or seismic forces.

4.15.4. Uplift

Uplift load resistance may be based on the total weight of the concrete footing and the total weight of soil above the footing, if it is buried. For uplift resistance computation purposes, we recommend use of a total unit weight of concrete of 150 pcf and a total unit weight of soil of 125 pcf.

4.15.5. Settlement

For foundations designed and constructed in accordance with the recommendations in this report and under static loading conditions, total post-construction foundation settlement is expected to be less than about ¾-inch. Post-construction differential settlement is expected to be about half of the total settlement.

4.16. CONCRETE SLAB-ON-GRADE FLOORS

4.16.1. Subgrade Preparation

Floor slabs-on-grade should be structurally independent of the rest of the foundation system. Slab thickness and reinforcing should be evaluated by the designer based on the anticipated use and loadings. Prior to constructing interior concrete slabs supported-on-grade, the subgrade soils should be prepared as recommended in Section 4.5, Site Preparation, and Section 4.7, Engineered Fill. Scarification and compaction may not be required if floor slabs are to be placed directly on undisturbed

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engineered fill, or native soil compacted during site preparation, or within earthwork cut areas consisting of cemented soils and if approved by the project Geotechnical Engineer during construction.

4.16.2. Rock Capillary Break or Base Course

We recommend the compacted subgrade be overlain with a minimum 4-inch thickness of compacted crushed rock to serve as a capillary break. The material should have less than 5 percent by weight passing the No. 4 sieve size. A capillary break may reduce the potential for soil moisture migrating upwards toward the slab. In general, Caltrans Class 2 aggregate base or similar materials do not meet the above recommendations and should not be used to underlay interior concrete slabs supported-on-grade where moisture sensitive floor coverings or storage is anticipated.

A capillary break may not be required for some types of construction (such as warehouses, equipment buildings, garages, and other non-habitable structures). For these types of structures, the gravel capillary break recommended above may be omitted and the slab placed directly on a minimum 6-inch thick layer of compacted Class 2 aggregate base material. The material should be compacted to at least 95 percent relative compaction at a moisture content slightly above optimum.

4.16.3. Vapor Retarder Membrane

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 introduced moisture (such as landscape irrigation or plumbing leaks) the current industry standard is to place a vapor retarder membrane on the compacted crushed rock layer (described above). This membrane typically consists of visquene or polyvinyl plastic sheeting at least 10 mils in thickness. It should be noted that although capillary break and vapor barrier systems are currently the industry standard, this system may not be completely effective in preventing floor slab moisture problems. These systems will not "moisture proof" the floor slab nor will it assure floor slab moisture transmission rates will meet floor-covering manufacturer standards. The design and construction of such systems are 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-

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grade floor design. Building design and construction may have a greater role in perceived moisture problems since sealed buildings/rooms or inadequate ventilation may result in excessive moisture in a building and affect indoor air quality.

4.16.4. Floor Slab Considerations

It should be noted that although capillary break and vapor barrier systems are currently the industry standard in the Sacramento area, this system may not be completely effective in preventing floor slab moisture problems. Various factors such as surface grades, adjacent planters, the quality of slab concrete, and the permeability of the on-site soils affect slab moisture control performance. In many cases, perceived floor moisture problems are the result of improper curing of flooring adhesives, not excessive slab moisture transmission. 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 and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking, or curling in 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.

It is emphasized that we are not concrete slab-on-grade floor moisture proofing experts. We make no guarantee nor provide any assurance that use of the capillary break/vapor retarder 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.

In general, the elevation of exterior grades should not be higher than the elevation of the gravel capillary break beneath the slabs. To reduce the potential for underslab moisture problems, we recommend that floor slabs be established 6 inches or more above surrounding final grades. Drainage should be provided for planters adjacent to buildings such that water does not accumulate against foundations.

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4.17. MOISTURE PROTECTION

Foundation and slab performance depends greatly on how well runoff waters drain from the site. This drainage should be maintained both during construction and over the entire life of the project. The ground surface around structures should be graded so that water flows rapidly away from structures without ponding. The surface gradient needed to do this depends on the landscaping type. In general, pavement and lawns within five feet of buildings should slope away at gradients of at least two percent. Densely vegetated areas should have minimum gradients of 5 percent away from buildings in the first five feet if it is practical to do so.

Planters should be built so that water exiting from them will not seep into the foundation areas or beneath slabs and pavement. In general, the elevation of exterior grades should not be higher than the elevation of the subgrade beneath the slab to help prevent water intrusion beneath slabs. In any event, maintenance personnel should be instructed to limit irrigation to the minimum actually necessary to properly sustain landscaping plants. Should excessive irrigation, waterline breaks, or unusually high rainfall occur, saturated zones and "perched" groundwater may develop. Consequently, the site should be graded so that water drains away readily without saturating the foundation or landscaped areas. Potential sources of water, such as water pipes, drains, and the like, should be frequently examined for signs of leakage or damage. Any such leakage or damage should be promptly repaired.

All utility trenches that pass beneath perimeter foundations should be backfilled with compacted non-pervious fill material or a lean concrete trench plug to reduce the potential for external water to migrate beneath the building through the utility trenches. Special care should be taken during installation of sub-floor water and sewer lines to reduce the possibility of leaks.

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4.18. EXTERIOR CONCRETE SLABS SUPPORTED-ON-GRADE

4.18.1. Subgrade Preparation

Prior to constructing exterior concrete slabs supported-on-grade[2] surficial soils should be scarified to a minimum depth of 8 inches, uniformly moisture-conditioned to between 2 and 5 percent above the optimum moisture content, and compacted to at least 90 percent relative compaction. Scarification and compaction may not be required if exterior slabs are to be placed directly on undisturbed engineered fill, or native soil compacted during site preparation, or within earthwork cut areas consisting of cemented soils if approved by the project Geotechnical Engineer during construction.

Once the slab subgrade soil has been moisture conditioned and compacted, the soil should not be allowed to dry prior to concrete placement. If the subgrade soil is too dry, the moisture content of the soil should be restored to the recommended value prior to placement of concrete. The geotechnical engineer should check the moisture content of the subgrade soil prior to construction of the slabs.

Proper moisture conditioning and compaction of subgrade soils is important. Even with proper site preparation, we anticipate that over time there will be some effects of soil moisture change on concrete flatwork. Exterior flatwork will be subjected to edge effects due to the drying out or wetting of subgrade soils where adjacent to landscaped or non-paved areas. To help reduce edge effects, lateral cutoffs such as a thickened edge are suggested. Control joints should be used to reduce the potential for panel cracks as a result of soil displacement. Steel reinforcement will aid in keeping the control joints and other cracks closed.

Exterior concrete slabs-on-grade should be cast free from adjacent footings or other non-heaving edge restraints. This may be accomplished by using a strip of 1/2-inch asphalt-impregnated felt divider material between the slab edges and the adjacent structure. Frequent construction or control joints should be provided in all concrete slabs where cracking is objectionable. Dowels at the construction and control joints will also aid in reducing uneven slab movements.

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[2] *Within this report exterior concrete slab supported-on-grade refers to walkways, driveways, patios, etc. and specifically excludes roadway pavements.*

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4.19. RETAINING WALLS

4.19.1. Lateral Earth Pressures

Retaining walls should be designed to resist the earth pressure exerted by the retained, compacted backfill plus any additional lateral force that will be applied due to surface loads placed at or near the wall or below-grade structure. Recommended design criteria for retaining walls and subsurface structures are presented in Table 4.8 below.

Table 4.8
Lateral Earth Pressures
(Ultimate Values)

		Equivalent Fluid Density (pcf)	
Backfill Configuration	**Earth Pressure**	**Drained**	**Submerged or Undrained**
Level	Active At Rest	45 60	85 95
Level	Passive	400	220
Surcharge factor, Ka = 0.35 x surcharge pressure for active case Surcharge factor, Ko = 0.50 x surcharge pressure for at-rest case			

Passive resistance should be neglected within the upper 1 foot of soil unless the passive pressure area in front of the footing is protected by concrete or pavement and is not disturbed by excavation or other means. A friction factor of 0.35 may be used to estimate sliding resistance between the bottoms of wall footings and the underlying bedrock. Passive resistance and sliding friction may be combined provided the sliding friction component is reduced by 50 percent.

An additional lateral earthquake-induced force of $8H^2$ should be used for design of unrestrained walls, where H is the retained wall height. This resultant of the earthquake-induced earth pressure may be assumed to act at a point that is 0.6H above the base of the wall. The pressure distribution is an inverted triangle with the maximum pressure of 16H at the top of the wall.

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Lateral resistance parameters provided above are ultimate values. Therefore, a suitable factor of safety should be applied to these values for design purposes. The appropriate factor of safety will depend on the design condition and should be determined by the project Structural Engineer. Depending on the application, typical factors of safety could range from 1.0 to 2.0.

4.19.2. Wall Drainage

The above-recommended lateral earth pressure values do not include lateral pressures due to hydrostatic forces. Therefore, wall backfill should be free draining and provisions should be made to collect and dispose of excess water that may accumulate behind earth retaining structures.

Wall drainage may be provided by free-draining gravel surrounded by synthetic filter fabric or by prefabricated, synthetic drain panels. In either case, drainage should be collected by perforated pipes and directed to a sump, storm drain, weep hole(s), or other suitable location for disposal. We recommend drain rock consist of durable stone having 100 percent passing the 1-inch sieve and less than 5 percent passing the No. 4 sieve. Synthetic filter fabric should have an equivalent opening size (EOS), U.S. Standard Sieve, of between 40 and 70, a minimum flow rate of 110 gallons per minute per square foot of fabric, and a minimum puncture strength of 110 pounds.

4.19.3. Backfill

To achieve the earth pressure and drainage conditions described above, the backfill adjacent to the walls should consist primarily of granular, low-plasticity soils. Clays with moderate or high expansion potential (highly-plastic) should not be used for retaining wall backfill. Over-compaction of wall backfill should be avoided because increased compaction effort can result in lateral pressures significantly higher than those recommended in this report. Backfill placed within 3 feet of the walls should be compacted with light or hand-operated equipment. All backfill should be placed and compacted in accordance with recommendations provided in Section 4.7, Engineered Fill.

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4.19.4. Construction Considerations

Properly compacted retaining wall backfill may experience some settlement or deflection after construction. This is a result of normal deflection of the wall and settling of relatively deep-engineered fills. This post-construction "settling in period" will vary with wall type, size, and construction and should be taken into account in overall site design.

If the building walls are to also act as retaining walls, we recommend the walls be waterproofed prior to placement of the wall backfill. Waterproofing should extend from the base of the foundation to the elevation of the exterior finish grade. We recommend contacting a waterproofing consultant/manufacturer for specific recommendations for placement, sealing and protection.

4.20. PAVEMENTS

Pavements are expected to include the Fulton Avenue extension as well as parking lots and driveways for the automobile dealership complex. A laboratory R-value of 5 was obtained on a sample of the near-surface clayey sand soils obtained from Boring B-15 (northwest area of the site). We anticipate the majority of the near surface clayey soils will have similar R-values. The following sections present our recommendations for asphalt concrete and portland cement concrete pavement sections based on an R-value of 5. The anticipated Traffic Indices and corresponding alternative pavement sections presented below should be reviewed by the project civil engineer in consultation with the owner during development of the final plans.

4.20.1. Asphalt Concrete Pavement Sections

Recommended pavement sections based on Caltrans design criteria are presented below in Table 4.9 for a range of Traffic Indices (TI) from 4.5 to 10 and a subgrade R-value of 5.

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Table 4.9
Asphalt Concrete Pavement Section Alternatives

Traffic Index (TI)	Alternative 1		Alternative 2		
	AC (Inches)	AB (Inches)	AC (Inches)	AB (Inches)	ASB (Inches)
4.5	2-1/2	9	2-1/2	4	6
5.0	2-1/2	11	2-1/2	5	6-1/2
6.0	3	13-1/2	3	6	8-1/2
7.0	3-1/2	16-1/2	3-1/2	7	10-1/2
8.0	4-1/2	18-1/2	4-1/2	7-1/2	12
9.0	5	21-1/2	5	9	13-1/2
10.0	6	23-1/2	6	9-1/2	15-1/2

Notes:
AC = Asphalt Concrete
AB = Class 2 Aggregate Base (R-value = 78)
ASB = Class 2 Aggregate Subbase (R-value = 50)

Asphalt concrete should meet the requirements for 1/2 or 3/4-inch maximum, medium Type B asphalt concrete per Section 39, Caltrans Standard Specifications. Aggregate base material should conform to Section 25 of the Caltrans Standard Specifications.

4.20.2. Golf Course Trail Extension

If paved golf cart paths are proposed, we recommend they consist of at least 2 inches of asphalt concrete over 6 inches of compacted Caltrans Class 2 aggregate base material. Concrete pavements should also be underlain by the recommended base course. The subgrade and base course should be prepared as recommended for pavement sections provided herein.

4.20.3. Portland Cement Concrete Pavement Sections

Portland cement concrete pavements are typically better able to resist the intense stresses induced in pavements by the turning motions of vehicles - particularly delivery and garbage trucks. Concrete pavements should be used in areas frequented by such vehicles as well as in driveway and entry aprons. Concrete pavement sections presented in Table 4.10 below are based on current Portland Cement Association (PCA) design procedures and the assumptions listed below. These assumptions should be reviewed by the project owner and civil engineer to evaluate their suitability

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for this project. Changes in the assumptions will affect the corresponding pavement section.

- Modulus of subgrade reaction = 175 psi/in
- Modulus of rupture of concrete = 600 psi
- Aggregate Interlock Joints
- No concrete shoulders
- 30-year design life
- Load Safety Factor = 1.0

Table 4.10
Recommended Portland Cement Concrete Pavement Sections

Proposed Use	Average Daily Truck Traffic (ADTT)	Portland Cement Concrete		Aggregate Base	
		(feet)	(inches)	(feet)	(inches)
Main Drive Aisles/Light Truck Traffic	20	0.50	5.5	0.50	6.0
Truck Lanes and Access ways	20	0.50	6.0	0.50	6.0

Portland cement concrete pavement sections provided above are contingent on the following recommendations being implemented during construction.

- All pavement subgrades should be prepared as recommended in Section 4.5, Site Preparation, and Section 4.7, Engineered Fill of this report.

- Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils are not allowed to become wet.

- Concrete pavement should have a minimum 28-day compressive strength of 4,000 psi. Concrete slumps should be from 3 to 4 inches. The concrete should be properly cured in accordance with PCA recommended procedures and vehicular traffic should not be allowed for 3 days (automobile traffic) or 7 days (truck traffic).

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- To help offset plastic shrinkage, concrete pavement may be reinforced with at least No. 3 bars, 24 inches on-center, each way or 6x6-W2.9xW2.9 wire mesh (located ⅓ of the slab thickness from the top of the slab).
- Construction and/or control joint spacing should not exceed 12 feet.
- Thickened edges should be used along outside edges of concrete pavements. Edge thickness should be at least 2 inches greater than the concrete pavement thickness and taper to the actual concrete pavement thickness 36 inches inward from the edge. Integral curbs may be used in lieu of thickened edges.
- Overfinishing of concrete pavements should be avoided. Typically, a broom or burlap drag finish should be used.

The above pavement recommendations should be incorporated into project plans and specifications by the project architect and/or engineer. These recommendations are not intended to be used as a specification for construction.

4.20.4. Pavement Subgrade Preparation

Subgrade preparation should be conducted in accordance with the recommendations provided in Section 4.5, Site Preparation, and Section 4.7, Engineered fill. Compacted pavement subgrade should be non-yielding.

4.20.5. Unstable Subgrade

In the event unstable (pumping) subgrades are encountered within planned pavement areas, we recommend a heavy, rubber-tired vehicle (typically a loaded water truck) be used to test the load/deflection characteristics of the finished subgrade materials. We recommend this vehicle have a minimum rear axle load (at the time of testing) of 16,000 pounds with tires inflated to at least 65 pounds per square inch pressure. If the tested surface shows a visible deflection extending more than 6 inches from the wheel track at the time of loading, or a visible crack remains after loading, corrective measures should be implemented. Such measures could include disking to aerate, chemical treatment, replacement with drier material, or other methods. We recommend Kleinfelder be retained to assist in developing which method (or methods) would be applicable for this project.

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4.20.6. Variations in Subgrade Materials

Pavement sections provided above are based on the soil conditions encountered during our field investigation, our assumptions regarding final site grades, and limited laboratory testing. In the event actual pavement subgrade materials are significantly different than those tested for this study, we recommend representative subgrade samples be obtained and additional R-value tests performed. Should the results of these tests indicate a significant difference, the design pavement section(s) provided above may need to be revised.

4.20.7. Pavement Subgrade Drainage

Proper and sufficient drainage should be provided to allow flow into drainage inlets or lateral ditches. Water should not be allowed to pond on or adjacent to the pavement and cause the subgrade to become saturated. Periodic inspections and repair of cracks should be conducted as part of regular facility maintenance.

We recommend that concrete curbs separating pavement and landscaped areas extend into the subgrade and below the bottom of adjacent aggregate base materials.

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5 ADDITIONAL SERVICES AND LIMITATIONS

5.1. ADDITIONAL SERIVICES

Variations in soil types and conditions are possible and may be encountered during construction. To permit correlation between the soil data obtained during this investigation and the actual soil conditions encountered during construction, we recommend that Kleinfelder be retained to provide observation and testing services during site earthwork and foundation construction. This will allow us the opportunity to compare actual conditions exposed during construction with those encountered in our investigation and to provide supplemental recommendations if warranted by the exposed conditions. Earthwork should be performed in accordance with the recommendations presented in this report, or as recommended by Kleinfelder during construction.

Kleinfelder should review geotechnical portions of the final plans and specifications to evaluate how the recommendations presented in this report were implemented into the designs. If Kleinfelder is not retained for these services, the client will assume our responsibility for any potential claims that may arise. We should also provide consultation during construction to assist in the observation of how key parts of the design are implemented, answering questions from the designers or contractors, and looking for subsurface conditions that might differ from the design assumptions or that might require modification of the design. This review provides an opportunity to detect misinterpretation or misunderstandings prior to the start of construction.

5.2. LIMITATIONS

We have prepared this report for the City of Sacramento and their consultants for use in the design of the proposed Del Paso Park project near the intersection of Fulton Avenue and the Interstate 80 Business Route freeway in Sacramento, California. We prepared this report in substantial accordance with the generally accepted geotechnical engineering practice, as it exists in the project area at the time of our study. No warranty, express or implied, is made or intended. It is likely that soil conditions vary between or beyond the locations that we have explored to date. Land use, site

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conditions (both on-site and off-site) or other factors may change over time, and additional geotechnical exploration may be required if the construction plan or schedule changes.

This report may be used only by the City of Sacramento and their project consultants, only for the purposes stated, and within a reasonable time from its issuance. Any party other than the City of Sacramento who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be conducted 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.

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APPENDIX A
FIELD INVESTIGATION

General

The subsurface conditions at the site were explored between June 19 through 21, and on July 14, 2006 by drilling 20 borings to depths ranging from about 16 to 36½ feet below existing grade. Borings were drilled using CME 45 and Mobile B-61 truck-mounted drill rigs equipped with 4-inch-diameter solid flight augers and 6-inch-diameter hollow stem augers. The locations of borings drilled for this investigation are shown on Plate 3 of this report.

Borings were located in the field by visual sighting and/or pacing from existing site features. Therefore, the locations of borings shown on Plate 3 should be considered approximate and may vary slightly from that indicated on the plate.

Logs of Borings and Descriptions

Our representative maintained logs of the borings, visually classified soils encountered according to the Unified Soil Classification System (see Plate A-1), and obtained relatively undisturbed and bulk samples of the subsurface materials. Soil classifications were made in the field from samples and auger cuttings in accordance with ASTM D 2487 (Visual-Manual Procedure). Sample classifications, blow counts recorded during sampling, and other related information were recorded on the boring logs. Following laboratory testing, the visual soil classifications made in the field were reviewed and re-classified in accordance with ASTM D 2488. A key to the Logs of Borings is presented on Plate A-2 of this appendix. Logs of Borings are presented on Plates A-3 through A-22.

Where cemented soils were encountered the degree of cementation was evaluated using the criteria presented in Table A.1 below.

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Table A.1
Cemented Soil Descriptions

Description	Characteristics
“Weakly cemented”	Crumbles or breaks with handling or little finger pressure.
Moderately cemented”	Crumbles or breaks with considerable finger pressure.
Strongly cemented”	Does not crumble or break with finger pressure.

Sampling Procedures

During the drilling operations, penetration tests were performed in accordance with ASTM D-1586 at regular intervals using either a California or Standard Penetration Sampler to evaluate the relative density/consistency of the soils encountered and to retain soil samples for laboratory testing. The penetration tests were performed by initially driving the sampler 6 inches into the bottom of the bore hole using a 140 pound automatic trip-hammer falling 30 inches. The sampler was driven the first 6 inches to penetrate loose soil cuttings and “seat” the sampler. Thereafter, the sampler was progressively driven an additional 12 inches with the results recorded as the corresponding number of blows required to advance the sampler 12 inches, or any part thereof. Soil samples obtained from the borings were packaged and sealed in the field to reduce moisture loss and disturbance and brought to our Sacramento laboratory for testing. After borings were completed, they were backfilled with the drill cuttings. Samples will be retained for a period of six months from the date of this report.

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APPENDIX B
LABORATORY TESTING

General

Laboratory tests were conducted on selected samples to aid in soil classification and to evaluate physical properties of the soils that may affect the geotechnical aspects of project design and construction. A description of the laboratory testing program is presented below. A summary of all laboratory tests performed is presented on the Summary of Laboratory Tests, Plate B-1. Most of the laboratory test results are also included on the boring logs.

Moisture Content and Dry Unit Weight

Moisture content and dry unit weight tests were performed to evaluate moisture-conditioning requirements during site preparation and earthwork grading, soil overburden, and active and passive earth pressures, and relative soil strength and compressibility. Moisture content was evaluated in general accordance with ASTM Test Method D 2216. Dry unit weight was evaluated using procedures similar to ASTM Test Method D 2937. Results of these tests are presented on the logs and are summarized on the Summary of Laboratory Tests.

Sieve Analysis

Sieve analyses were performed to evaluate the gradational characteristics of the material and to aid in soil classification. Tests were performed in general accordance with ASTM Test Method D 422. Results of these tests are presented on Plate B-3, and are summarized on the boring logs and the Summary of Laboratory Tests.

Atterberg Limits

Atterberg Limits tests were performed to aid in soil classification and to evaluate the plasticity characteristics of the material. Additionally, test results were correlated to published data to evaluate the shrink/swell potential of near-surface site soils. Tests were performed in general accordance with ASTM Test Method D 4318. Results of

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these tests are presented on the logs, Plate B-2 and are summarized on the Summary of Laboratory Tests.

Unconfined Compression

An unconfined compression test was performed on a selected, undisturbed sample to evaluate the undrained shear strength of the sample collected. Test procedures were in general accordance with ASTM Test Method D 2166. Results of this test are presented on the Summary of Laboratory Tests.

R-Value

A resistance value (R-value) test was performed on a bulk soil sample obtained from Boring B-15 to evaluate pavement support characteristics of the near-surface site soils. Test procedures were in general accordance with California Test 301. Results of this test are presented on Plate B-4.

Corrosion

Selected samples of the near-surface soils encountered at the site were subjected to chemical analysis for the purpose of corrosion assessment by a subcontract laboratory, Sunland Analytical of Rancho Cordova, California. The samples were tested for pH, resistivity, soluble sulfates, and soluble chlorides. The samples were tested in general accordance with California Test Methods 643, 422, and 417 for pH and resistivity, soluble chlorides, and soluble sulfates, respectively. The results of the tests reported by Sunland Analytical are provided on Plates B-5 through B-7.

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