GEOTECHNICAL ENGINEERING INVESTIGATION REPORT
for
Oroville Riverfront Road Development
Grand Avenue and West Side of Highway 70
APN 031-100-001, -005, -006, -008, -011 and
APN 031-164-100
Oroville, California, 95965

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Project No. 70,165-01
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Reference: ***Oroville Riverfront Road Development***
APN 031-100-001, -005, -006, -008, -011, and 031-164-100
Oroville, California, 94609

Subject: ***Geotechnical Engineering Investigation Report***

Dear Mr. Bloom,

Holdrege & Kull (H&K) is pleased to have had this opportunity to provide geotechnical engineering services for development of the Oroville Riverfront Road Development located at APN 031-100-001, -005, -006, -008, -011, and 031-164-100, along Grand Avenue and the west side of State Highway 70, Oroville, California. Our geotechnical engineering investigation of the site was performed consistent with the scope of services presented in our November 7, 2006 proposal PC06.030.

The findings, conclusions, and recommendations presented in this report are based on the following relevant information collected and evaluated by H&K: literature review, surface observations, subsurface exploration, laboratory test results, and our experience with similar conditions in the area. It is our opinion that the site is suitable for the proposed construction provided the geotechnical engineering design recommendations presented in this report are incorporated into the earthwork and structural improvements. This report should not be relied upon without review by H&K if a period of 24 months elapses between the issuance report date shown above and the date when construction commences.

Our experience and that of the civil engineering profession clearly indicates that during the construction phase of a project the risks of costly design, construction and maintenance problems can be significantly reduced by retaining the design geotechnical engineering firm to review the project plans and specifications and to provide geotechnical engineering construction quality assurance (CQA) observation and testing services. Upon your request we will prepare a CQA geotechnical engineering services proposal that will present a work scope, tentative schedule and fee estimate for your consideration and authorization. If H&K is not retained to provide geotechnical engineering CQA services during the construction phase of the project, then H&K will not be responsible for geotechnical engineering CQA services provided by others nor any aspect of the project that fails to meet your or a third party's expectations in the future.

H&K appreciates the opportunity to provide geotechnical engineering services for this important project. If you have questions or need additional information, please do not hesitate to contact the undersigned below at 530-894-2487.

Sincerely,

Holdrege & Kull

Prepared by:	Reviewed by:
Erik W. James, E.I.T. Staff Engineer/Geologist	Domald M. Olsen, R.C.E. 49,514 Principal Engineer/Geologist

Copies To: Addressee (4 copies)

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1 INTRODUCTION

Holdrege & Kull (H&K) performed a geotechnical engineering investigation of the Oroville Riverfront Road Development located at APN 031-100-001, -005, -006, -008, -011, and 031-164-100 in Oroville, California consistent with the scope of services presented in our Geotechnical Engineering Investigation Proposal (PC06.030), dated November 7, 2006. A copy of the proposal, excluding the fee and contract sections, is included in Appendix A. Our findings, conclusions, and recommendations are presented herein. For your review, Appendix B presents a document prepared by ASFE entitled *"Important Information About Your Geotechnical Engineering Report."* This document summarizes project specific factors, limitations, content interpretation, responsibilities, and other pertinent information. Please read this document carefully. The information presented in this report is organized into the following sections: introduction, site investigation, laboratory testing, conclusions, recommendations, limitations, figures, and appendices. This introduction report section presents the following information: site location description, investigation purpose, and scope-of-services.

1.1 SITE LOCATION AND DESCRIPTION

The project site is located west of State Highway 70 (SR 70) and south of Grand Avenue in Oroville, encompassing about 65-acres. Figure 1 shows the site location and near vicinity. At the time of our site investigation the following site condition were observed:

- The site is surrounded by homes to the west and north, state route highway 70 (SR 70) to the east, undeveloped land along the eastern side of SR 70, and the Feather River to the south.
- The site consists of an upper terrace area at about elevation 240-feet above mean sea level (MSL) which encompasses the northern third of the property, and a lower terrace area at about elevations 150- to 170-feet MSL that encompasses the remainder of the property. The, Feather River abuts the toe of the lower terrace bluff to the south at an elevation of about 135-feet MSL.
- There is a gated access road to the upper bluff terrace area from the west on 5th Street. There is an un-gated access road from the east off of Oroville Drive near 2nd Street, which pass beneath SR 70 to reach the lower bluff terrace area.
- The upper terrace area is nearly flat and is underlain by of native soil consisting of a cemented, silty, gravelly, cobbley, conglomerate. There is a steep upper bluff slope (1.5 horizontal to 1.0 vertical) that separates the upper and lower terrace areas. The lower terrace area is generally flat, but there are several deep dredge pond depressions (up to 30- to 40-feet) that are partially filled with water. The lower terrace area appears to be underlain by cobbley dredged mine tailings.

There is a steep lower bluff slope that separates the lower terrace area and the Feather River to the south.

1.2 PROPOSED IMPROVEMENTS

H&K understands that the proposed site construction will include the following earthwork grading and structural improvements:

- Earthwork grading at the site will include: site clearing and grubbing, construction of cuts and engineered fills for building pads and streets, and installation of underground utilities.
- Structural improvements will include: one-story and two-story commercial office buildings constructed with wood-framing and cinder-block walls, concrete slab-on-grade first floors, loading dock areas, patios and sidewalks, and asphalt concrete (AC) paved streets and parking lots.

1.3 INVESTIGATION PURPOSE

The purpose of our investigation is to obtain sufficient on-site information about the soil, rock and ground water conditions at the site to allow us to prepare geotechnical engineering design recommendations for construction of the proposed earthwork and structural improvements described in the preceding. H&K did not evaluate the site for the presence of hazardous waste, mold, asbestos, and radon gas. Therefore, the presence and removal of these materials are not discussed in this report.

1.4 SCOPE-OF-SERVICES

H&K performed a specific scope-of-services to develop geotechnical engineering design recommendations for the proposed earthwork and structural improvements. A brief description of each work scope task is presented below. A detailed description of each work scope task is presented in Section 2 (Site Investigation) of this report.

- **Task 1, Site Investigation:** H&K performed a site investigation to characterize the existing surface and subsurface soil, rock, and ground water conditions encountered to the maximum depth excavated. H&K's field engineer/geologist made observations, took representative soil samples, and performed field tests at a limited number of exploratory trench locations. H&K performed laboratory tests on selected soil samples to evaluate their geotechnical engineering material properties.
- **Task 2, Data Analysis and Engineering Design:** H&K evaluated the field and laboratory site data, proposed site improvements, and used this information to develop geotechnical engineering design recommendations for earthwork and structural improvements. Engineering judgment was used to extrapolate our observations and conclusions regarding the field and laboratory data to other

areas located between and beyond the locations of our subsurface exploratory excavations.

- **Task 3, Report Preparation:** H&K prepared this report to present our findings, conclusions, and recommendations.

2 SITE INVESTIGATION

H&K performed a site investigation to characterize the existing site conditions and to develop geotechnical engineering design recommendations for earthwork and structural improvements. Each component of our site investigation is presented below.

2.1 LITERATURE REVIEW

H&K performed a limited review of available literature that was pertinent to the project site. The following summarizes our findings.

2.1.1 Site Improvement Plan Review

H&K reviewed the preliminary site improvement plans prepared by Cranmer Engineering Inc. which are shown in Figure 2. Figure 2 only shows the proposed building and street locations. We understand that there are two proposed locations for the western access road alignment. One proposed location is along the eastern side of the upper terrace bluff and the other road location is along the western side of the upper terrace bluff. We understand the final decision on the location of the western access road alignment will be made at a later date, based in part on additional site work that will be performed by H&K. The additional H&K site work was not included in the work scope performed for this report. The additional H&K work scope will be defined at a later date and will be performed for an additional fee..

2.2 FIELD INVESTIGATION

H&K performed a field investigation of the site on November 28 and November 29, 2006. H&K's Field Engineer/Geologist described the surface and subsurface soil, rock, and ground water conditions observed at the site using the procedures cited in the American Society for Testing and Materials (ASTM), Volume 04.08,*"Soil and Rock; Dimension Stone; and Geosynthetics"* as general guidelines for our field and laboratory procedures. The Field Engineer/Geologist described the soil color using the general guideline procedures presented in the Munsel Soil Color Chart. Engineering judgment was used to extrapolate the observed surface and subsurface soil, rock, and ground water conditions to areas located between and beyond our subsurface exploratory locations. The surface, subsurface, and ground water conditions observed during our field investigation are summarized below.

2.2.1 Surface Conditions

H&K observed the following surface conditions during our field investigation of the property. Figure 3 shows the site topography which generally consists of two relatively flat lying terrace areas that are separated by a relatively steep upper terrace bluff slope with inclinations ranging from about 1.5H:1V (horizontal to vertical slope ratios) to near vertical. The average elevations of the upper and lower terrace areas are about 240-feet and 160-feet MSL, respectively. The southern edge of the lower terrace is bordered by an equally steep lower bluff slope. The toe of the lower bluff slope abuts the Feather River at an elevation of about 135-feet MSL. The surface of the upper terrace and upper bluff areas are underlain by native soil consisting of weakly cemented, silty, cobbley, conglomerate materials. The surface of the lower terrace and lower bluff areas are predominantly underlain by relatively clean, cobbley dredged mine tailings with several scattered dredge ponds that are partially filled with dredged clay and silt deposits. We understand that the lower terrace and lower bluff areas were mined with dredge equipment for their gold content during the early 1900s.

Presently, the upper terrace area is predominantly covered with well developed growth of meadow grasses and weeds. Native pine and oak trees, and brush surround the meadow and cover the upper bluff surfaces. The lower terrace area is predominantly covered poorly developed growths of grasses and weeds, with scattered trees growing around the dredged pond areas. The lower bluff slope is predominantly covered with grasses, weeds and scattered trees.

2.2.2 Subsurface Conditions

The subsurface soil, rock and groundwater conditions were investigated by excavating exploratory trenches at the site. The subsurface information obtained from this investigation method is described herein.

2.2.2.1 Exploratory Excavation Information

H&K provided engineering oversight for the excavation of 9 exploratory soil trenches at the project site with a Caterpillar 420D backhoe equipped with a 24-inch wide bucket. Figure 3 shows the approximate locations of our subsurface exploratory excavations. The trenches were excavated to depths ranging from 3 to 12 feet below the existing ground surface. Bedrock refusal or refusal within cemented, hard-consolidated soil did not occur in any of the subsurface exploratory excavations. The soil, rock and ground water conditions below these depths are unknown. Engineering judgment was used to extrapolate the observed soil, rock, and ground water conditions to areas located between and beyond our subsurface exploratory excavations.

H&K's Field Engineer/Geologist logged each exploratory trench using the Unified Soils Classification System (USCS) as a guideline. Representative relatively undisturbed soil samples were not collected due to the rocky soil material onsite. Representative disturbed bulk soil samples were also collected from T06-1. The bulk soil samples were placed in plastic sack containers, labeled and transport to our soil laboratory facility. Selected soil samples were tested in our laboratory to determine their engineering material properties which may include: particle size gradation and plasticity. The gravelly and cobbley nature of the native soil and dredged soil materials that were encountered in our exploratory trenches made it impossible for us to obtain relatively undisturbed soil samples that were suitable for laboratory shear strength testing. Therefore, we used the limited soil laboratory tests results for the on-site materials and our engineering judgment to develop geotechnical engineering recommendations for: foundation, retaining wall, concrete slab-on-grade floors, ground water drainage systems, and both cut slope and fill slope grading designs.

Detailed descriptions of the soil, rock and ground water conditions that were encountered at each subsurface exploratory location are presented on the exploratory trench logs included in Appendix C. The percentages listed are based on visual field estimates of each material's dry weight. The units encountered during our subsurface investigation of the site are described below.

1. **Lower Terrace And Bluff Areas**

- **Unit A, GW, Well Graded Gravel Mine Dredged Soil:** This soil unit consists of the following field estimated particle size percentages 5% low-plastic fines, 5% fine sand, 5% medium sand, 5% coarse sand, 20% fine gravel, 20% coarse gravel, 35% cobble, 5% boulder. This soil is predominantly Grayish Brown with a Munsel Color Chart designation of (10YR 5/2). This soil was loose to dense and damp to moist at the time of our subsurface investigation. The material appears to be dredge mine tailings.

2. **Upper Terrace And Bluff Areas**

- **Unit B, SM, Silty Sand Native Soil:** This soil unit consists of the following field estimated particle size percentages 40% low-plastic fines, 45% fine sand, 5% medium sand, 5% coarse sand, 3% fine gravel, 2% coarse gravel. This soil is predominantly dark reddish brown with a Munsel Color Chart designation of (5YR 3/3). This soil was loose and damp at the time of our subsurface investigation.
- **Unit C, GM, Silty Gravel Native Soil:** This soil unit consists of the following field estimated particle size percentages 15% low-plastic fines, 5% fine sand, 5% medium sand, 5% coarse sand, 30% fine gravel, 20% coarse gravel, 20% cobble. This soil is predominantly dark reddish brown with a Munsel Color Chart designation of (5YR 3/3). This soil was dense to very dense and damp at the time of our subsurface investigation. This unit was also weakly cemeted.

2.2.2.2 *Lower Terrace Exploratory Trench Side-Wall Instability*

The trench side-walls of the exploratory trenches that were excavated into the gravely-cobbley dredge mine tailings were extremely unstable. Generally, the trench excavation width at the surface grew proportionately with the increase in excavation depth, because the side-walls continued to collapse as the trench was deepened. In other words, the trench could not be deepened below a depth of about 10-feet below the existing land surface because the trench side-walls would collapse and fill the excavation bottom. This unstable condition will make it very difficult for underground utilities to be installed within the dredged mine tailings that underlie the lower terrace area.

2.2.2.3 Ground Water Conditions

H&K did not encountered perched ground water nor the ground water table in any of the exploratory trenches excavated at the site; however, ground water was observed in the dredged mine ponds during our site investigation.. Seasonal fluctuations in the local groundwater table at the project site and vicinity are unknown at this time; however it is generally understood that the ground water table elevation is highest at the end of the winter rainy season and lowest at the end of the summer dry season.

3 LABORATORY TESTING

H&K performed laboratory tests on selected soil samples taken from the subsurface exploratory excavations to determine their engineering material properties. The gravelly and cobbley nature of the native soil and dredged mine soil materials that were encountered in our exploratory trenches made it impossible for us to obtain relatively undisturbed soil samples that were suitable for laboratory shear strength testing. Therefore, we used the limited soil laboratory tests results for the on-site materials and our engineering judgment to develop geotechnical engineering design recommendations for earthwork and structural improvements. The following laboratory tests were performed using the cited American Society for Testing and Materials (ASTM), Caltrans Test Method (CTM), or Uniform Building Code (UBC) guideline procedures:

- ASTM D422, Particle Size Gradation (Sieve Only)

Table 3.1 presents a summary of the laboratory test results. Appendix D presents the laboratory test data sheets.

Table 3.1, Laboratory Test Results

Trench No.	Sample		Test Results											
	No	Depth	D2487 D2488	D2216	D2937	D422		D4318		D4829	D2844 CTM301	D1140	D3080	
			USCS	Moisture Content	Dry Density	Passing No.4	Passing No.200	Plasticity Index	Liquid Limit	Expansion Index (Uncor-rected/Saturated.)	R-Value	Passing 200 sieve (%)	Cohesion	Friction Angle
		(ft)	(sym)	(%)	(pcf)	(%)	(%)	(%)	(%)		(dim)		(psf)	(degree)
T06-1	BS-1 (1)	2.5				36.3	4.3							

Notes: (1) Large cobbles and boulders (greater then about 4- to 5-inch diameter were not included in the sample)

4 LIMIT EQUILIBRIUM SLOPE STABILITY ANALYSIS

H&K performed slope stability analyses of the existing upper and lower bluff slopes. The slope stability analyses were performed using SLIDE 5.0™ a two-dimensional limit equilibrium slope stability computer program developed by Rocscience. SLIDE 5.0™ computes a safety factor against failure of the slope by comparing the ratio of the resisting forces that act to make the slope stable to the driving forces that act to make the slope unstable. The generally accepted minimum safety factors for short-term and long-term slope loading conditions are 1.1 and 1.5, respectively. The short-term loading condition includes evaluation of the final slope configurations subjected to earthquake shaking forces (seismic). The long-term loading conditions include evaluation of the final slope configuration subject to static forces. The slope stability analysis methodologies for both static and seismic (earthquake) loading conditions, material properties and slope configurations are described in the following:

4.1 ENGINEERING MATERIAL PROPERTIES

The engineering material properties for both the upper bluff and lower bluff slope materials were conservatively estimated based on our experience with similar earth materials because the rocky nature of the soil made it impossible to obtain samples suitable for laboratory shear strength testing. H&K was not able to determine shear strength parameters by either laboratory testing or in situ testing for the soil due to the presence of over-size particles (cobble and boulder size materials from 3-inch to 2-foot diameters). Therefore, we assumed conservative shear strength properties for the upper bluff slope native cobbley soil and lower bluff slope cobbley mine tailings materials. We also assumed values for alluvial deposits at the southwest toe of the upper bluff slope and for the fluvial deposits that that underlie the Feather River channel at the toe of the lower bluff slope. We also made assumptions about the depth of the dredged mine tailings within the lower bluff because we did not encounter the bottom of these tailings at the maximum depth excavated in any of our exploratory trenches. Based on our literature review and experience we estimated engineering material properties for long-term loading and short-term seismic loading conditions for the fill soil, native soil, and bedrock. Table 5.1.1 summarizes the estimated material properties used in the slope stability analyses.

Table 5.1.1, Estimated Material Properties

Material Type	Unit Weight	Saturated Unit Weight	Cohesion (psf)	Friction Angle (degree)
Tailings Conglomerate (Lower Bluff)	145	155	100	35
Bluff Conglomerate (Upper Bluff)	145	155	2300	50
Alluvium	126	138	500	35
Fluvial Deposits	115	140	100	35

4.2 SEISMICITY ANALYSES

We performed a probabilistic seismicity analyses to develop estimates of the peak ground accelerations or maximum horizontal acceleration (MHA) that could occur at the site resulting from an earthquake occurring on known faults located within a 100 kilometer radial distance from the site. Typically the earthquake induced MHA (ground shaking) that is expected to occur at a site is defined as the mean peak ground accelerations (50^{th} percentile) and for an upper bound the mean plus one standard deviation (84^{th} percentile) peak acceleration. The mean peak acceleration estimate has a 10 percent probability of exceedance during a 50 year period and the mean plus one standard deviation peak acceleration has a 10 percent probability of exceedance in a 100 year period. We performed the pseudo-static analysis consistent with the guideline procedures presented in the following publications:

- California Geological Survey (CGS) Special Publication No. 117, "Guidelines For Evaluating And Mitigating Seismic Hazards In California," March 1997.
- American Society Of Civil Engineers (ASCE), "Recommended Procedures For Implementation Of DMG Special Publication 117 Guidelines for Analyzing And Mitigating Landslide Hazards In California," June 2002.

These analyses and their results are described below.

4.2.1 Deterministic Peak Ground Acceleration Analysis:

A deterministic seismicity analysis computes estimated ground motions expected at a project site using applicable attenuation relationships, earthquake magnitudes occurring on specific faults, and the shortest radial distance to the closest portion of a specific fault to the project site. A separate deterministic analysis must be performed on each fault that is considered capable of impacting the site. Ground accelerations are generally expressed as a percentage of gravitation acceleration (g) which is 9.81 meters per second squared (32.2 feet per second squared).

We used EQFAULT a computer program developed by Thomas F. Blake to perform a deterministic seismicity analysis of the know faults generally located within typically a 100-kilometer radial distance to the Oroville Riverfront Road Development site and the Idriss (1994) acceleration attenuation relationship for rock and stiff soil. EQFAULT uses the California Geological Survey (CG) 2002 fault model and data base. Appendix E presents the EQFAULT computer program output files. The EQFAULT results are summarized below.

Deterministic Analysis Seismic Site Parameters			
Fault Name	**Approx. Distance (km)**	**Maximum Earthquake Mag. (Mw)**	**Peak Site Acceleration (g)**
Foothills Fault System	2.0	6.5	0.629
Great Valley 1	62.0	6.7	0.065
Great Valley 2	66.2	6.4	0.046
Great Valley 3	75.2	6.8	0.055
Mohawk – Honey Lake Zone	92.4	7.3	0.053
Rate for NE California	98.8	7.3	0.049
Bartlett Springs	102.3	7.1	0.039
Hunting Creek - Berrysessa	102.7	6.9	0.032
Great Valley 4	104.6	6.6	0.028
Battle Creek	106.6	6.5	0.025
Collayomi	124.3	6.5	0.016
Hat Creek – MacArthur-Mayfield	124.4	7.0	0.033
Round Valley	127.0	6.8	0.022
Honey Lake	129.4	6.9	0.023
Concord – Green Valley	129.6	6.9	0.023
Great Valley 5	137.8	6.5	0.016
West Napa	140.4	6.5	0.013
Maacama (South)	140.9	6.9	0.021
Maacama (Central)	142.1	7.1	0.025
Maacama (North)	148.9	7.1	0.024
Rodgers Creek	149.3	7.0	0.021
Genoa	154.9	6.9	0.022

The deterministic seismic analysis indicates that the peak horizontal ground acceleration expected at the site will be about 0.629-g

4.2.2 Probabilistic Peak Ground Acceleration Analysis:

A probabilistic seismicity analysis evaluates the different levels of ground motions (accelerations) that are expected to occur at a site in terms of an aggregate peak horizontal acceleration or maximum horizontal acceleration (MHA) with a probability of exceedance in a specified period of time. Typically the MHA is determined for a mean acceleration with a 10 percent probability of exceedance in a 50 year period and an upper bound of the mean plus one standard deviation MHA with a 10 percent probability of exceedance in a 100 year period.

The MHA expected at the site (computed with selected attenuation relationship) does not correspond to a unique earthquake magnitude and unique radial distance between a fault and the site, because a probabilistic analysis sums the contributions of all possible earthquakes occurring on all of the faults that are considered to be capable of impacting the site. In order to identify the most strongly influencing earthquake magnitude (modal magnitude, M) and radial distance (modal distance, R) that is associated with the expected MHA and hazard level (i.e., 10 percent probability of exceedance in a 50 year period) it is necessary to de-aggregate the hazards analysis results. It should be understood that de-aggregation is sensitive to the ground motion parameter for which the hazard analyses are performed (i.e., different values of M and R can be interpreted from the de-aggregation data for a single MHA).

We used FRISKSP a computer program developed by Thomas F. Blake to perform a probabilistic seismicity analysis of the known faults located within a 100-kilometer radial distance to the Oroville Riverfront Road Development site and the Idriss (1994) acceleration attenuation relationship for sites underlain by rock and stiff soil. FRISKSP also uses the California Geological Survey (CG) 2002 fault model and data base. Appendix E presents the FRISKSP computer program output files. The FRISKSP results are summarized below.

Probabilistic Analysis Seismic Site Parameters		
Probability of Exceedance (%)	**Period (Years)**	**Maximum Horizontal Acceleration (g)**
10	50 = mean	0.10
10	100 = mean +1 SD	0.10

Seismicity Analysis Results: We believe that the probabilistic analyses results represent a more realistic seismic impact to the site than do the deterministic analyses results. Therefore, the probabilistic analyses results should be used in the pseudo-static slope stability analyses.

4.3 SLOPE STABILITY METHODOLOGIES

We used Bishop's Modified Method (A.W. Bishop, 1955, "The Use of the Slip Circle in the Stability Analysis of Slopes," Geotechnique, Vol. 5, No. 1) to perform long-term static, short-term seismic and short-term static reservoir rapid drawdown slope stability analyses of the proposed cut and fill slope configurations. The results of the slope stability analyses are presented herein.

4.3.1 Long-Term Static Slope Stability Methodology

We estimated the ground water table elevations from the water levels observed during our site investigation in the dredge pond depressions of the lower terrace area, and the elevation of the water surface of the Feather River. Therefore we assumed the presence of a single water table surface that extends continuously from the Feather River water level to the level in the ponds within the dredge tailings of the lower terrace, and extrapolated that surface toward and beneath the upper terrace. We used SLIDE 5.0™ a computer program that performs an iterative search for the failure surface with the lowest static safety factor (F_{static}) using a grid-circle-center approach for each circular failure surface. The critical slope mass is identified as the slope mass with the lowest static safety factor.

4.3.2 Short-Term Seismic Slope Stability Methodology

The seismic stability analysis was performed by using a pseudo-static methodology that imposes a constant horizontal seismic acceleration in the down-slope direction of the critical slope mass that was identified by the static slope stability analysis. The seismic acceleration acting on the inscribed critical slope mass serves as an additional force that acts to make the slope unstable. The seismic acceleration causes the static safety factor to be lowered. This lowered safety factor is commonly referred to as the pseudo-static safety factor or seismic safety factor ($F_{seismic}$). The shear strength engineering material properties for short-term seismic loading conditions are usually assumed to be represented by consolidated drained (CD) conditions. We used SLIDE 5.0™ to compute seismic safety factors for a peak horizontal ground acceleration of 0.10g that has a 10 percent probability of exceedance in a 50 year period.

4.4 UPPER BLUFF STABILITY ANALYSES

The following presents the results of our fill slope stability analyses for the existing natural slopes.

4.4.1 Upper Bluff Slope Stability Analyses – Southwestern Slope

H&K performed both static and seismic stability analyses of the upper bluff slope configuration shown on Figure 3 as Section A-A'. The topographic base map was provided by Cranmer Engineering. We used the assumed engineering material properties of the upper bluff material described in Section 4.1 of this report.

The results of the upper bluff western slope stability analyses are shown on Figures F1, F2 and F3 of Appendix F and are summarized below.

- Minimum static safety factor, $F_{static} = 4.47 \geq 1.50$ = indicates that a stable slope condition exists for a long-term static loading conditions.
- Probabilistic peak horizontal ground acceleration = 0.10-g = (10 percent probability of exceedance in 50 years).
- Minimum seismic safety factor, $F_{seismic} = 3.74 \geq 1.10$ = indicates that a stable slope condition exists for a short-term seismic loading condition.

4.4.2 Upper Bluff Slope Stability Analyses – Southeastern Slope

H&K performed both static and seismic stability analyses of the upper bluff slope configuration shown on Figure 3 as Section B-B′. The topographic base map was provided by Cranmer Engineering. We used the assumed engineering material properties of the upper bluff material described in Section 4.1 of this report.

The results of the upper bluff southeastern slope stability analyses are shown on Figures F4, F5 and F6 of Appendix F and are summarized below.

- Minimum static safety factor, $F_{static} = 4.33 \geq 1.50$ = indicates that a stable slope condition exists for a long-term static loading conditions.
- Probabilistic peak horizontal ground acceleration = 0.10g (10 percent probability of exceedance in 50 years).
- Minimum seismic safety factor, $F_{seismic} = 3.60 \geq 1.10$ = indicates that a stable slope condition exists for a short-term seismic loading condition.

4.5 LOWER BLUFF STABILITY ANALYSES

The following presents the results of our fill slope stability analyses for the existing dredged material fill-slopes.

4.5.1 Lower Bluff Slope Stability Analyses – Southwestern Slope

H&K performed both static and seismic stability analyses of the lower bluff slope configuration shown on Figure 3 as Section C-C′. The topographic base map was provided by Cranmer Engineering. We used the assumed engineering material properties of the upper bluff material described in Section 4.1 of this report.

The results of the lower bluff southwestern slope stability analyses are shown on Figures F7, F8 and F9 of Appendix F and are summarized below.

- Minimum static safety factor, $F_{static} = 2.07 \geq 1.50$ = indicates that a stable slope condition exists for a long-term static loading conditions.

- Probabilistic peak horizontal ground acceleration = 0.10-g = (10 percent probability of exceedance in 50 years).
- Minimum seismic safety factor, $F_{seismic}$ = 1.64 ≥1.10 = indicates that a stable slope condition exists for a short-term seismic loading condition.

4.5.2 Lower Bluff Slope Stability Analyses – Southeastern Slope

H&K performed both static and seismic stability analyses of the lower bluff slope configuration shown on Figure 3 as Section D-D'. The topographic base map was provided by Cranmer Engineering. We used the assumed engineering material properties of the upper bluff material described in Section 4.1 of this report.

The results of the lower bluff southeastern slope stability analyses are shown on Figures F10, F12 and F13 of Appendix F and are summarized below.

- Minimum static safety factor, F_{static} = 2.24 ≥ 1.50 = indicates that a stable slope condition exists for a long-term static loading conditions.
- Probabilistic peak horizontal ground acceleration = 0.10-g (10 percent probability of exceedance in 50 years).
- Minimum seismic safety factor, $F_{seismic}$ = 1.75 ≥1.10 = indicates that a stable slope condition exists for a short-term seismic loading condition.

5 CONCLUSIONS

The conclusions presented below are based on information developed from our field and laboratory investigations.

1. It is our opinion that the site is suitable for the proposed construction improvements provided that the geotechnical engineering design recommendations presented in this report are incorporated into the earthwork and structural improvement project plans.
2. Prior to construction, H&K should be allowed to review the proposed earthwork grading plan and structural improvement plans to determine if our geotechnical engineering recommendations are applicable or need modifications.
3. At the time of our investigation the site consisted of an upper terrace and upper bluff slope areas and a lower terrace and lower bluff slope areas that are generally situated in the northern one-third and southern two-thirds of the property, respectively. . The upper terrace and bluff areas are generally underlain by native weakly cemented silty, gravelly, cobbley conglomerate materials and the lower terrace and bluff areas are predominantly underlain by cobbley dredge mining tailings and silt and clay filled dredge ponds. The upper terrace area is covered with grass and surrounded by oak trees on the bluff slopes. The lower terrace area is relatively un-vegetated, except for sparse grasses and occasional trees along the toe of the lower bluff slopes, and around the dredge pond depressions.
4. The soil conditions observed to a maximum depth of 12 feet below the existing ground surface in our subsurface exploratory excavations generally consisted of (described relative to the existing ground surface) upper terrace area: 0- to 1-foot of Silty SAND (SM) and 1- to 10-feet of Sandy GRAVEL (GM), and lower terrace area: unknown thickness of Cobbly GRAVEL (GW) dredge tailings.
5. At the time of our subsurface site investigation on November 28 and November 29, H&K observed ground water in the dredge pond depression on the lower terrace. However, the depth to the ground water table may vary at these and other on-site locations as a result of both seasonal fluctuations and local soil conditions.
6. During our site investigations the exploratory trenches that were excavated into the lower terrace cobbley dredged mine tailings, collapsed to form large bowl-shaped excavations. Generally, the trench excavation width at the surface grew proportionately to the increase in excavation depth, because the side-walls continued to collapse as the trench was deepened. In other words, the trenches could not be deepened below a depth of about 10-feet below the existing land surface because the trench side-walls would collapse and fill the excavation bottom. This unstable condition will make it very difficult for underground utilities to be installed within the dredged mine tailings that underlie the lower terrace

area. Therefore the underground utility contractor should anticipate similar trench instabilities during construction and should plan to use appropriate shoring methods.

7. Our slope stability analysis generally indicates that both the upper terrace and lower terrace bluff slopes will be stable for both long-term static and short-term seismic loading conditions. However our analysis also indicates that the lower terrace bluff slopes along the Feather River which are underlain by the predominantly cohesionless (relatively clean), loose to dense, gravely-cobbley, dredge tailing materials may be susceptible to both shallow slumping, lateral spreading and deep seated failures during seismic (earthquake) loading conditions. Therefore, we recommend that a minimum 100-foot set-back distance from the top of the lower terrace bluff be established for construction of permanent buildings.

6 RECOMMENDATIONS

H&K developed geotechnical engineering design recommendations for earthwork and structural improvements from our field and laboratory investigation data. Our recommendations are presented hereafter.

6.1 EARTHWORK GRADING

Our earthwork grading recommendations include: clearing and grubbing, native soil preparation, fill construction, cut-fill transitions, fill slope grading, cut slope grading, erosion controls, underground utility trenches, construction de-dewatering, soil corrosion potential, retaining wall back-fill, subsurface drainage, surface water drainage, review of construction plans, and construction quality assurance/quality control (QA/QC) monitoring. Our earthwork grading recommendations are presented below.

6.1.1 Grading Difficulties on the Lower Terrace Dredge Tailings

The cobbley gravel (GM) dredged mine materials that were encountered in our exploratory trenches were very unstable. This unstable condition caused the trench side-walls to collapse forming large bowl shaped trenches. Excavation below a depth of about 10-feet was not possible the side-walls collapsed into the bottom of the trench. Excavating utility trenches and grading cut slopes with inclinations greater than 3H:1V and deeper than 5-feet will be difficult. The underground utility contractor should anticipate that the need for use of appropriate shoring methods.

6.1.2 Lower Terrace Dredge Tailings Pond Removal

The dredge tailings pond areas should be completely excavated to remove all of the clay and silt deposits that were placed during the mining operations. The silt and clay dredge tailings materials should not be used for constructing engineered fills; however, these materials can be stockpiled on-site for use in constructing landscaped fill areas and as top soil materials. The excavated areas should be observed by the project geotechnical engineering consultant prior to backfilling. Backfilling of the excavated pond areas should be performed consistent with the grading recommendations presented in Sections 6.1.4 and 6.1.5 of this report.

6.1.3 Stripping and Grubbing

The site should be stripped and grubbed of vegetation and other deleterious materials as described below.

1. Strip and remove the top 2 to 4 inches of soil containing shallow vegetation roots and other deleterious materials. This highly organic topsoil can be stockpiled on-site and used for surface landscaping, but should not be used for constructing compacted engineered fills. Grub the underlying 6 to 8 inches of soil to remove any large vegetation roots or other deleterious material while leaving the soil in

place. The project geotechnical engineer or his/her representative should approve the use of any soil materials generated from the clearing and grubbing activities.

2. Remove all large shrub and tree roots and tree stumps. Excavate the remaining cavities or holes to a sufficient width so that an approved backfill soil can be placed and compacted in the cavity or holes. Sufficient backfill soil should be placed and compacted in order to match the surrounding elevations and grades. The project geotechnical engineer or his/her representative should observe and approve the preparation of the cavities and holes prior to placing and compacting engineered fill soil in the cavities and holes.
3. Remove all rocks greater than 3 inches in greatest dimension from the top 12 inches of the soil. Rocks with a greatest dimension larger than 3 inches will be referred to in this report as "over sized" rock materials. Over sized rock materials can be stockpiled on-site and used to construct engineered fills; however they must be placed at or near the bottom of deep fills but not shallower than 3 feet from the finished subgrade surface. The oversized rock should be placed with enough space between them to avoid clustering and the creation of void space. The project engineer or his/her representative should approve the use and placement of all over sized rock materials prior to constructing compacted engineered fills.
4. Excessively large amounts of vegetation, other deleterious materials, and over sized rock materials should be removed from the site.

6.1.4 Native Soil Preparation For Engineered Fill Placement

After completing site clearing and grubbing activities, the exposed native soil should be prepared for placement and compaction of engineered fills as described below.

1. The native soil should be scarified to a minimum depth of 6 inches below the existing land surface or cleared and grubbed surface and then uniformly moisture conditioned. If the soil is classified as a coarse-grained soil by the USCS (i.e., GP, GW, GC, GM, SP, SW, SC or SM) then it should be moisture conditioned to within $\pm$ 3 percentage points of the ASTM D1557 optimum moisture content. If the soil is classified as a fine grained soil by the USCS (i.e., CL, CH, ML, MH) then it should be moisture conditioned between 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content.
2. The native soil should then be compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry unit weight (density). The moisture content, density, and relative percent compaction should be tested by the project engineer or the project engineer's field representative to evaluate whether the compacted soil meets or exceeds this minimum percent compaction and moisture content requirements. The earthwork contractor shall assist the project engineer or the project engineer's field representative by excavating test pads with the on-site earth moving equipment. Native soil preparation beneath

concrete slab-on-grade structures (i.e., floors, sidewalks, patios, etc.), asphalt concrete (AC) pavement should be prepared as specified in Section 5.2 (Structural Improvements).

3. The prepared native soil surface should be proof rolled with a fully loaded 4,000 gallon capacity water truck with the rear of the truck supported on a double-axel, tandem-wheel, undercarriage or approved equivalent. The minimum tire pressure should be 65 pounds per square inch (psi). The proof rolled surface should be visually observed by the project engineer or the project engineer's field representative to be firm, competent and relatively unyielding. The project engineer or the project engineer's field representative may also evaluate the surface material by hand probing with a ¼-inch-diameter steel probe; however, this evaluation method should not be performed in place of proof rolling as described in the preceding.
4. Construction quality assurance tests should be performed using the minimum testing frequencies presented in Table 6.1.4 or as modified by the project engineer to better suit the site conditions.

Table 6.1.4, Minimum Testing Frequencies

ASTM No.	Test Description	Minimum Test Frequency(1)
D1557	Modified Proctor Compaction Curve	1 per 40,000 SF (2) Or Material Change (3)
D2922	Nuclear Moisture Content	1 per 10,000 SF
D3017	Nuclear Density	1 per 10,000 SF

Notes:
(1) These are minimum testing frequencies that may be increased or decreased at the project engineer's discretion on the basis of the site conditions encountered during grading.
(2) SF = square feet
(3) Which ever criteria provide the greatest number of tests.

5. The native soil surface should be graded to minimize ponding of water and to drain surface water away from the building foundations and associated structures. Where possible surface water should be collected, conveyed, and discharged into natural drainage courses, storm sewer inlet structures, permanent engineered storm water runoff percolation/evaporation basins, or engineered infiltration subdrain systems.

6.1.5 Engineered Fill Construction With Testable Earth Materials

Engineered fills are constructed to support structural improvements. Engineered fills should be constructed using non-expansive soil as described in Section 5.1.3.1. If possible, the use of expansive soil for constructing engineered fills should be avoided. If the use of expansive soil cannot be avoided then engineered fills should be constructed as described in Section 5.1.3.2 or as modified by the project engineer. If soil is to be imported to the site for constructing engineered fills, then H&K should be allowed to evaluate the suitability of the borrow soil source by taking representative soil samples for laboratory testing. Testable earth materials are generally considered to be soils with gravel and larger particle sizes retained on the No. 4 mesh sieve that make up less than 30 percent by dry weight of the total mass.

The relative percent compaction of testable earth materials can readily be determined by the following ASTM test procedures: laboratory compaction curve (D1557), field density (D2922) and field moisture content (D3017). Construction of engineered fills with non-expansive and expansive testable earth materials are described below.

6.1.5.1 Engineered Fill Construction With Non-Expansive Soil

Construction of engineered fills with non-expansive soil should be performed as described below.

1. Non-expansive soil used to construct engineered fills should consist predominantly of materials less than 1/2 inch in greatest dimension and should not contain rocks greater than 1 inches in greatest dimension (over sized material). Non-expansive soil should have a plasticity index (PI) of less than or equal to PI ≤ 15 as determined by ASTM D4318 Atterberg Indices test. Over sized materials can be placed at or near the bottom of deep fills, but not within 3.0 feet of the finished subgrade surface or within 2.0 foot of the foundation bottom. Deep fills are defined as fills that are greater than 10 feet in vertical thickness. Over sized materials should be spread apart to prevent clustering so that void spaces are not created. The project engineer or project engineer's field representative should approve the use of over sized materials for constructing engineered fills.
2. Non-expansive soil used to construct engineered fills should be uniformly moisture conditioned. If the soil is classified by the USCS as coarse grained (i.e., GP, GW, GC, GM, SP, SW, SC or SM), then it should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content. If the soil is classified by the USCS as fine grained (i.e., CL, CH, ML or MH), then it should be moisture conditioned to between 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content.
3. Engineered fills should be constructed by placing uniformly moisture-conditioned soil in maximum 8-inch-thick loose lifts (layers) prior to compacting.
4. The soil should then be compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.
5. The earthwork contractor should compact each loose soil lift with a tamping foot compactor such as a Caterpillar (CAT) 815 Compactor or equivalent as approved by our project engineer or the project engineer's field representative. A smooth steel drum roller compactor should not be used to compact loose soil lifts for construction of engineered fills.
6. The field and laboratory CQA tests should be performed consistent with the testing frequencies presented in Table 6.1.5.1 or as modified by the project engineer to better suit the site conditions.

Table 6.1.5.1, Minimum Testing Frequencies For Non-Expansive Soil

ASTM No.	Test Description	Minimum Test Frequency(1)
D1557	Modified Proctor Compaction Curve	1 per 3,000 CY (2) Or Material Change (3)
D2922	Nuclear Moisture Content	1 per 500 CY
D3017	Nuclear Density	1 per 500 CY

Notes: (1) These are minimum testing frequencies that may be increased or decreased at the project engineer's discretion on the basis of the site conditions encountered during grading.
(2) CY = cubic yards
(3) .Whichever criteria provide the greatest number of tests.

7. The moisture content, density, and relative percent compaction of all engineered fills should be tested by the project engineer's field representative during construction to evaluate whether the compacted soil meets or exceeds the minimum compaction and moisture content requirements. The earthwork contractor shall assist the project engineer's field representative by excavating test pads with the on-site earth moving equipment.
8. The prepared finished grade or finished subgrade soil surface should be proof rolled with a fully loaded 4,000 gallon capacity water truck with the rear of the truck supported on a double-axel, tandem-wheel, undercarriage or approved equivalent. The minimum tire pressure should be 65 pounds per square inch (psi). The proof rolled surface should be visually observed by the project engineer or the project engineer's field representative to be firm, competent and relatively unyielding. The project engineer or the project engineer's field representative may also evaluate the surface material by hand probing with a ¼-inch-diameter steel probe; however, this evaluation method should not be performed in place of proof rolling as described in the preceding.

6.1.5.2 Engineered Fill Construction With Expansive Soil

H&K did not observe any expansive soil at the site during our subsurface investigation. If expansive soils are encountered during grading of the site and if the property owner desires to use these expansive soil to construct engineered fills, then following three options should be considered. Each option has inherent risks and associated costs relative to future problems associated with expansive soil including settlement (downward movement) and/or heave (upward movement) of foundations and concrete slab-on-grade floors. The options are presented in the general order of decreasing cost but increasing risk with regards to future problems related to soil shrink-swell behavior. Prior to implementing any of these options, H&K should be notified so that we can develop appropriate recommendations or modify our recommendations, if necessary.

Option 1, Remove And Replace With Non-Expansive Soil (H&K Preferred Option-Lowest Shrink-Swell Behavior Risk):

This mitigation option has the lowest inherent risk of incurring future problems regarding settlement and/or swell (heave) of foundations and concrete slab-on-grade floors. This option consists of removing the expansive soil to a depth to be determined by the project geotechnical engineer. We estimate that expansive soil, if encountered at the site, should be removed to a minimum depth of 2 feet below the bottom of the building foundations and concrete slab-on-grade floors, which ever creates the greater depth below the adjacent finished grade surface. The actual removal depth or depths should be evaluated by the project geotechnical engineer or his/her field representative during grading and may be either increased or decreased depending upon the site conditions observed.

Non-expansive soil should then be placed, moisture conditioned and compacted to achieve the finished grades as described in Section 5.1.3.1 of this report. This option, when compared to the other two options, generally incurs the greatest upfront costs to the project but has the lowest risks for future problems arising from the high shrink-swell behavior of the soil. Repair of future problems due to soil shrink-swell behavior is generally from 10 to 100 times more costly then the cost of removing and replacing with non-expansive soil during initial grading.

Option 2, Expansive Soil Treatment With High Calcium Lime And Fly-Ash (Moderate Shrink-Swell Behavior Risk):

This mitigation option has an intermediate (moderate) inherent risk of incurring future problems regarding settlement and/or swell (heave) of foundations and concrete slab-on-grade floors. This option consists of mixing non-hydrated high calcium lime (commonly referred to as quick-lime) and fly-ash with the on-site expansive soil to reduce the expansive shrink-swell behavior of the soil. Treatment of native soil with lime and/or fly-ash should not be performed if the ambient air temperature will be below 40 degrees Fahrenheit during any portion of the mixing, compacting and curing periods. The curing period is typically three to five days after mixing and compacting. This option, when compared to the other two options, generally incurs an intermediate upfront cost to the project with an intermediate risk for future problems arising from the high shrink-swell behavior of the soil. H&K did not evaluate as part of our geotechnical engineering investigation work scope the percents of high calcium lime and fly-ash to be mixed with on-site expansive soil. For cost planning purposes only, the contractor may assume that each 1-foot thick layer of compacted fill consisting of the on-site high plasticity clay soils will require mixing with about 6 percent (by dry weight) non-hydrated high calcium lime. If the owner selects this option to mitigate the on-site expansive soils, then H&K should be notified so that H&K can prepare a treatment evaluation work scope that will include: preparation of grading construction specifications and performing laboratory tests to determine the required treatment percentage of lime and fly-ash for the on-site expansive soils. **H&K will perform the lime and fly-ash treatment evaluation only after receiving the owner's authorization to proceed and an approved budget amendment.**

Option 3, Reworking Expansive Soil (Highest Shrink-Swell Behavior Risk):

This mitigation option has the highest inherent risk of incurring future problems with settlement and/or heave of foundations and concrete slab-on-grade floors. This option consists of reworking the existing on-site expansive soil to reduce its expansive shrink-swell behavior. This option, when compared to the other two options, generally incurs the lowest upfront cost to the project with the highest risk for future problems arising from the high shrink-swell behavior of the soil. Construction of engineered fills with expansive soil should be performed as described below; however, these recommendations may need to be revised by the project geotechnical engineer or his/her representative during grading depending upon the actual site conditions encountered

1. Expansive soil used to construct engineered fills should consist predominantly of materials less than 1/2 inch in greatest dimension and should not contain rocks greater than 3 inches in greatest dimension (over sized material). Expansive soil will have a plasticity index (PI) greater than PI > 15 as determined by ASTM D4318 Atterberg Indices test. Over sized materials can be placed at or near the bottom of deep fills, but not within 3.0 feet of the finished subgrade surface or within 2.0 foot of the foundation bottom. Deep fills are defined as fills that are greater than 10 feet in vertical thickness. Over sized materials should be spread apart to prevent clustering so that void spaces are not created. The project engineer or project engineer's field representative should approve the use of over sized materials for constructing engineered fills.
2. Expansive soil used to construct engineered fills should be uniformly moisture conditioned to within 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content. The actual moisture content should be reviewed by the project engineer to determine if this preliminary moisture content range is appropriate or should be modified.
3. Engineered fills should be constructed by placing uniformly moisture-conditioned expansive soil in maximum 8-inch-thick loose lifts (layers) prior to compacting.
4. The expansive soil should then be compacted to achieve a minimum relative compaction of 88 percent and a maximum relative compaction of 92 percent of the ASTM D1557 maximum dry density. The actual percent relative compaction should be reviewed by the project engineer to determine if this preliminary relative percent compaction range is appropriate or should be modified.
5. Field and laboratory CQA tests should be performed consistent with the testing frequencies presented in Table 6.1.5.2 or as modified by the project engineer to better suit the site conditions.

Table 6.1.5.2, Minimum Testing Frequencies For Expansive Soil

ASTM No.	Test Description	Minimum Test Frequency(1)
D1557	Modified Proctor Compaction Curve	1 per 2,000 CY (2) Or Material Change (3)
D2922	Nuclear Moisture Content	1 per 500 CY
D3017	Nuclear Density	1 per 500 CY

Notes: (1) These are minimum testing frequencies that may be increased or decreased at the project engineer's discretion on the basis of the site conditions encountered during grading.
(2) CY = cubic yards.
(3) Whichever criteria provide the greatest number of tests.

6. The earthwork contractor should compact each loose soil lift with a tamping foot compactor such as a Caterpillar (CAT) 815 Compactor or equivalent as approved by the project engineer. A smooth steel drum roller compactor should not be used to compact loose soil lifts for construction of engineered fills with expansive soil.
7. The moisture content, density, and relative percent compaction of all engineered fills constructed with expansive soil should be tested by project engineer's field representative during construction to evaluate whether the compacted soil meets or exceeds the minimum compaction and moisture content requirements. The earthwork contractor shall assist the project engineer's field representative by excavating test pads with the on-site earth moving equipment.
8. The prepared finished grade or finished subgrade soil surface constructed with expansive soil should be proof rolled with a fully loaded 4,000 gallon capacity water truck with the rear of the truck supported on a double-axel, tandem-wheel, undercarriage or approved equivalent. The minimum tire pressure should be 65 pounds per square inch (psi). The proof rolled surface should be visually observed by the project engineer or the project engineer's field representative to be firm, competent and relatively unyielding. The project engineer or the project engineer's field representative may also evaluate the surface material by hand probing with a ¼-inch-diameter steel probe; however, this evaluation method should not be performed in place of proof rolling as described in the preceding.

6.1.6 Engineered Fill Construction With Non-Testable Earth Materials

If non-testable earth materials are encountered at the site during grading, and if these materials are used to construct engineered aerial fills and/or engineered utility trench backfills, then a performance (procedural) based construction quality assurance (CQA) method shall be used to evaluate the compaction work performed by the earthwork contractor. Non-testable earth materials generally consist of mixtures of gravels and/or cobbles with a matrix of sand, silty and/or clay materials. The gravel and larger particle size material content (materials retained above the No. 4 mesh sieve) generally is greater than 30 percent by dry weight of the total mass of the material. Use of non-testable earth materials for constructing engineered aerial fills and engineered utility trench backfills should be approved by the project geotechnical engineering consultant on a case-by-case basis. The performance based compaction method and criteria to be used during large scale grading should be determined by constructing a small test fill area for engineered aerial fills and a small test trench section for engineered utility trench backfills. The compaction method and CQA criteria should include the following site specific criteria:

1. **Specified Compaction Equipment:** We recommend that the contractor use a CAT815 Compactor equipped with kneading-foot wheels or an approved equivalent for construction of engineered aerial fills and a CAT245D Excavator equipped with a kneading-foot compactor wheel or an approved equivalent for construction of utility trench backfills. A smooth steel drum roller compactor should not be used to compact loose soil lifts for construction of engineered fills with non-testable earth materials.
2. **Maximum Loose Lift:** We recommend that the maximum loose lift (layer) thickness prior to compaction for both aerial fills and utility trench backfills should not exceed 12-inches.
3. **Moisture Content Range:** We recommend that the fill material be moisture conditioned such that the moisture content range of the matrix soil materials is between 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content for a compaction curve performed only on the matrix soil material.
4. **Minimum Number of Compactor Passes:** We recommend that the minimum number of specified compactor equipment passes for each loose lift coverage of earth materials used for construction of aerial fills and utility trench backfills be 8 and 20 passes, respectively. The actual number of compactor passes should be approved by the project geotechnical engineer or his/her representative from the results of constructing aerial test fills and/or utility trench test fills.
5. **Compaction Test Trenches:** At the direction of the CQA field technician the earthwork contractor shall periodically use his on-site equipment to excavate test trenches into the compacted non-testable engineered fill materials. The CQA field technician will evaluate the competency and stability of the compacted engineered fill material by making the following observations:
   - The relative difficulty of the contractor's equipment to excavate the compacted engineered fill materials. In other words the relative competency of the compacted engineered fill materials to resist excavation by the contractor's equipment.
   - The presence or lack of presence and quality of imprints left in the matrix soil materials by the gravels, cobbles and/or rocks that were removed by the contractor's equipment during excavation of the test trench.
   - The presence or lack of presence of newly broken gravel, cobble and/or rock materials that were sheared by the contractor's equipment during excavation of the test trench.

- The moisture content of the matrix soil materials exposed in the test trench is relatively uniform and is between 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content.

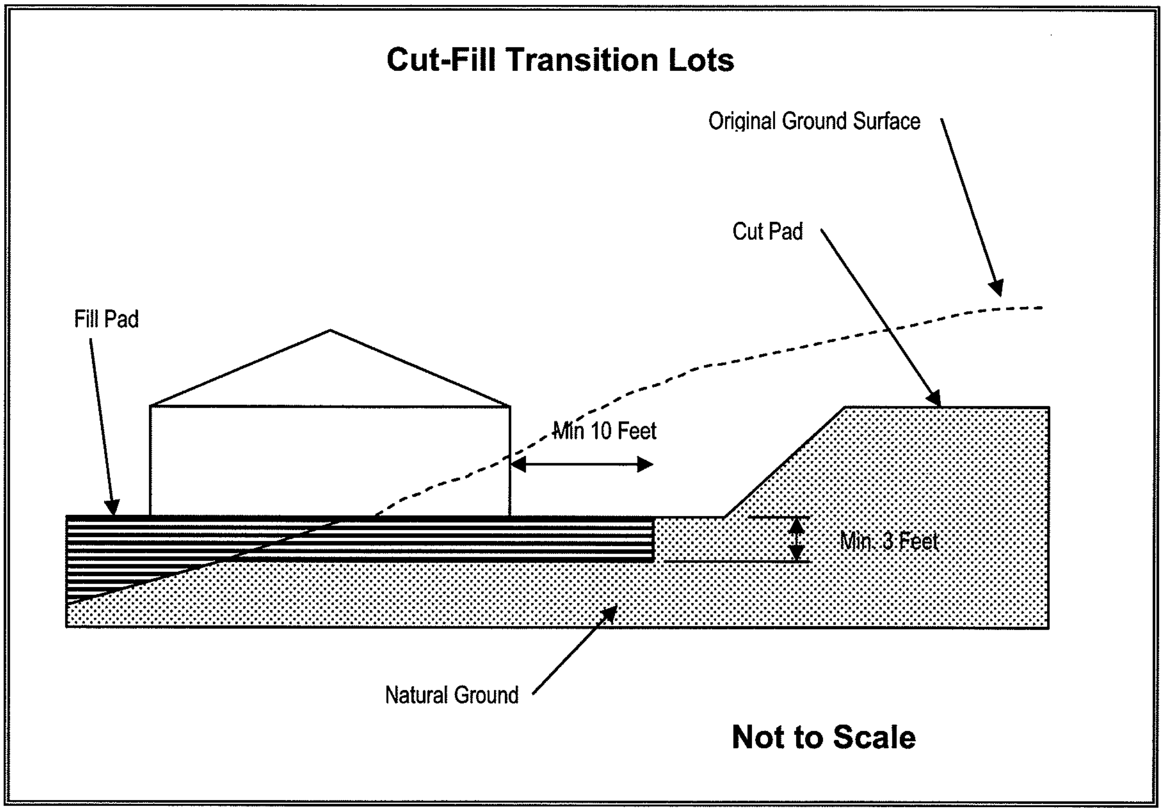
6. Proof Rolling: The prepared finished grade or finished subgrade soil surface constructed with non-testable earth materials should be proof rolled with a fully loaded 4,000 gallon capacity water truck with the rear of the truck supported on a double-axel, tandem-wheel, undercarriage or approved equivalent. The minimum tire pressure should be 65 pounds per square inch (psi). The proof rolled surface should be visually observed by the project engineer or the project engineer's field representative to be firm, competent and relatively unyielding. The project engineer or the project engineer's field representative may also evaluate the surface material by hand probing with a ¼-inch-diameter steel probe; however, this evaluation method should not be performed in place of proof rolling as described in the preceding.

6.1.7 Cut-Fill Transition Building Pads

Cut-fill transition building pads or lots should be graded as described below.

1. Building pads should be graded such that surface cut-fill transition lines do not occur on the surface directly beneath any structures. If a cut-fill transition line will occur on the surface and crosses directly beneath a building footprint area, then the building pad areas should be over excavated and replaced with compacted engineered fill soil to eliminate the surface exposure of the cut-fill transition contact. This construction method will eliminate the occurrence of what is commonly referred to as a "hard-point or hard-line" beneath the building footprint. The hard-line demarks an abrupt change in both the elastic and consolidation settlement behaviors of the engineered fill and native soil and/or rock materials exposed on opposite sides of the hard-line on the building pad surface. This abrupt change in material behaviors can result in development of cracks in the building foundations and/or concrete slab-on-grade floors that are typically coincident with the location and orientation of the underlying hare-line. Over excavation of the entire building pad footprint area and then replacing with engineered fill to the finished design subgrade elevations will significantly diminish the adverse impacts of the hard-line or cut/fill transition line beneath the building footprint area.

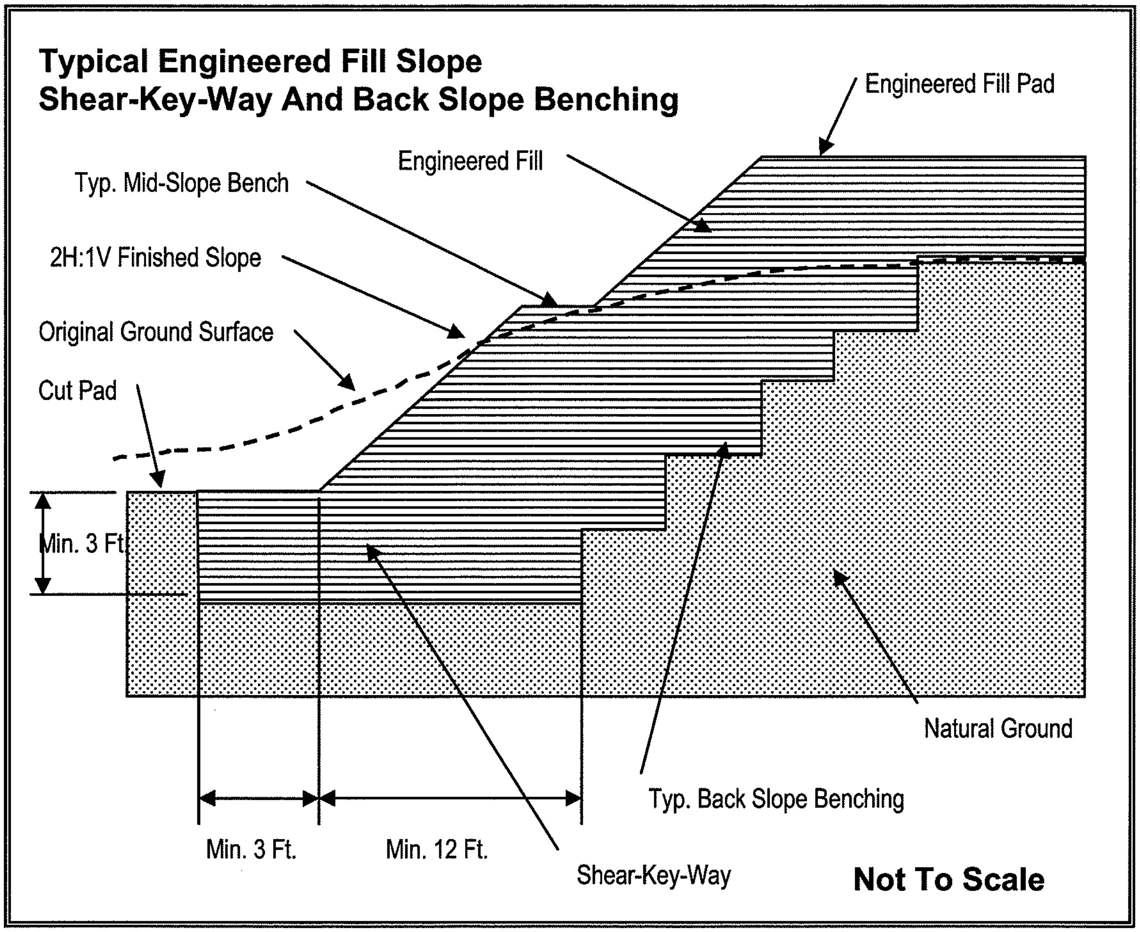
2. The depth of over excavation will depend on the foundation depth on the cut-side of the transition line and the total fill thickness on the fill-side of the transition line. The entire building and to a minimum distance of 10 feet beyond the building's foundation should be underlain by a minimum of 3 feet of non-expansive engineered fill or to provide a minimum of 2 feet of non-expansive engineered fill below the bottom of the foundations, which ever condition provides the greatest depth from the surface as shown on the figure below.



6.1.8 Fill Slope Grading

Fill slopes should be graded as described below.

1. Fill slopes should be graded with a maximum slope gradient of 2H:1V (horizontal to vertical slope ratio) and with a maximum vertical height of 30 feet. If fill slopes are to be graded steeper than 2H:1V and/or with a vertical height greater than 30 feet, then H&K should be notified so that slope stability analysis of the proposed slope configuration can be performed and provide revised recommendations.
2. A shear-key-way should be graded at the base of the fill slope prior to constructing the fill slope when fills are to be constructed on natural slopes with slope ratios greater than 5H:1V. The shear-key-way should be a minimum of 15-feet-wide and extend to a minimum depth of 3-feet below the finished subgrade surface or deeper as determined by the project geotechnical engineer during grading. The shear-key-way base should be graded with a minimum slope gradient of 2 percent towards the inside fill slope surface.



3. Fill slopes should be graded in horizontal lifts to the lines and grades shown on the grading plans. The design finished grade of a fill slope should be achieved by over building the slope face and then cutting it back to the design finished grade. Fill slopes should not be graded (extended horizontally) by compacting moisture conditioned, loose soil lifts on the slope face as thin veneer layers; in other words, do not construct engineered fill slopes by placing and compacting successive thin layers (veneers) of soil over the fill slope face at an inclination that is roughly coincident with the final fill slope horizontal to vertical slope ratio. The in-slope edge of each horizontal lift should be benched into the firm, competent, and relatively unyielding soil of the natural ground slope.
4. If ground water seepage from the slope and/or shear-key-way areas are encountered during grading or if the site conditions indicate that ground water seepage does occur during the wet winter season, then Holdrege & Kull should be notified so that we can assess the conditions and provide a design for installation of permanent dewatering subdrains.
5. Surface benches should be graded into the finished fill slope with a minimum width of 12.5 feet and with maximum vertical intervals of 15 feet between benches

or at the mid slope height if the total vertical slope height is greater than 15 feet but less than 30 feet.

6. Benches should be graded with a minimum slope gradient of 2 percent towards the inside fill slope surface; in other words, the bench slope gradient should cause surface water to drain towards the fill slope side of the bench (not over and down the fill slope face).
7. Fill soils used to construct slopes should be uniformly moisture conditioned, placed in loose lifts, and compacted as described in Section 5.1.3.

6.1.9 Cut Slope Grading

Cut slopes should be graded as described below.

1. Cut slopes should be graded with a maximum slope gradient of 2H:1V (horizontal to vertical slope ratio) and with a maximum vertical height of 30 feet. If cut slopes are to be graded steeper than 2H:1V and/or with a vertical height greater than 30 feet, then H&K should be notified so that we can perform a slope stability analysis of the proposed slope configurations and provide revised recommendations, if necessary.
2. Surface benches should be graded into the finished fill slope with a minimum width of 12.5 feet and with maximum vertical intervals of 15 feet between benches or at the mid slope height if the total vertical slope height is greater than 15 feet but less than 30 feet.
3. The benches should be graded with a minimum slope gradient of 2 percent towards the cut; in other words, the bench slope gradient should cause surface water to drain towards the cut slope side of the bench (not over and down the cut slope face).

6.1.10 Access Road Cut Slope Analyses And Grading

We understand that cut slopes will need to be graded along the final access road alignment that will extend form the upper terrace area to the lower terrace area; however, the final alignment of this access road has not yet been determined. When the final access road alignment is defined we will need to perform an additional stability analyses of the associated cut slopes. The access road alignment cut slope stability analysis was not a part of this geotechnical engineering investigation work scope and fee. We anticipate that cut slopes may need to be graded with maximum slope inclinations of 1H:1V.

6.1.11 Erosion Controls

Erosion controls should be installed as described below.

1. Erosion controls should be installed on all cut and fill slopes to minimize erosion caused by surface water run off.

2. Install on all slopes either an appropriate hydroseed mixture compatible with the soil and climate conditions of the site as determined by the local U.S. Soil Conservation District or apply an appropriate manufactured erosion control mat.
3. Install surface water drainage ditches at the top of all cut and fill slopes to collect and convey both sheet flow and concentrated flow away from the slope face.
4. Install surface water drainage ditches on the inside of all cut and fill slope benches to collect and convey both sheet flow and concentrated flow away from the slopes and to designed over side drain structures.
5. The drainage swales and over side drain structures should be lined with a minimum 2-inch-thick gunite concrete surface, erosion mats, or other suitable materials. Over side drains can also be designed with corrugated metal pipe that are anchored to the slope. If over side drains are deemed necessary, then H&K should be allowed to perform both hydraulic and structural analyses for design of these surface water drainage control structures.
6. The intercepted surface water should be discharged into natural drainage course or into other collection and disposal structures.

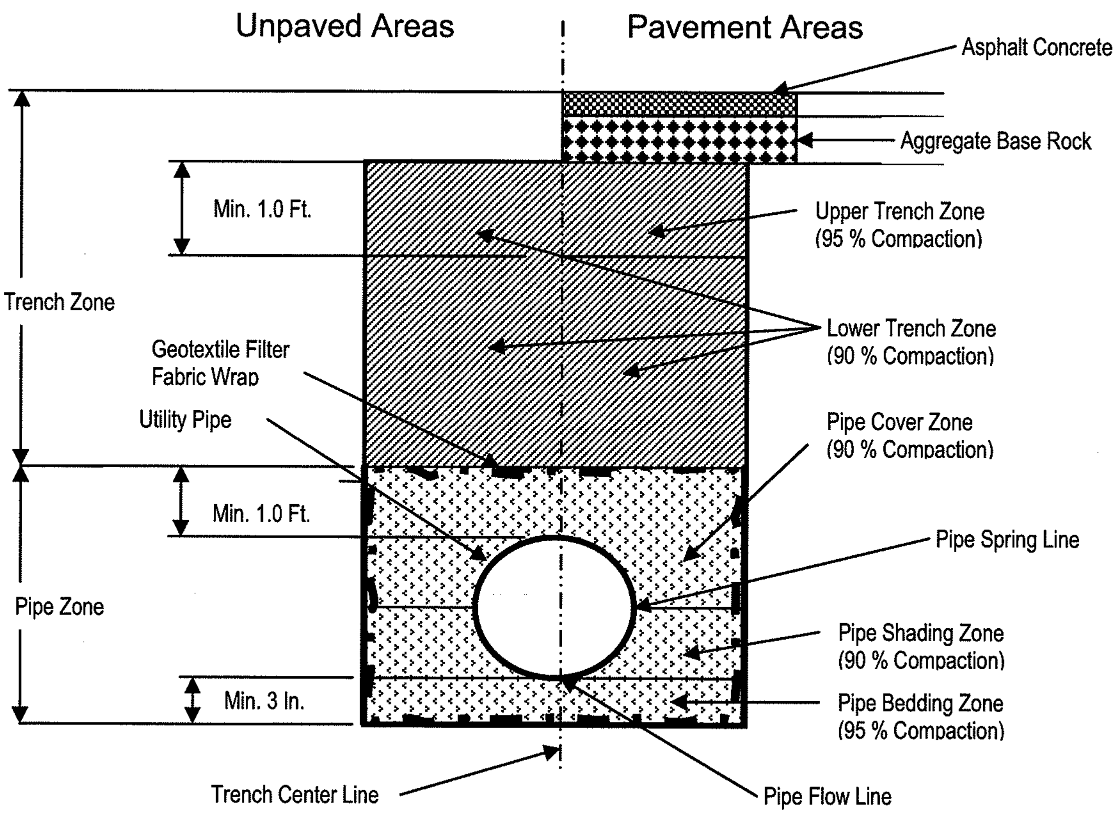
6.1.12 Underground Utility Trenches

Underground utility trenches should be excavated and backfilled as described below for each trench zone as shown in the figure below.

1. **Trench Excavation in Lower Terrace Mine Tailings:** The contractor should anticipate the need for using appropriate trench shoring equipment when excavating utility trenches in the lower terrace gravely, cobbley dredge mine tailing materials, because of a high potential for trench side-wall instability. Based on field observation of our exploratory trenches, deep excavations formed large bowl-shaped pits instead of narrow trenches with vertical side-walls.
2. **Trench Excavation Equipment:** H&K anticipate that the contractor will be able to excavate all underground utility trenches with a Case 580 Backhoe or equivalent.
3. **Trench Shoring:** All utility trenches that are excavated deeper than 4 feet below the surrounding ground surface are required by the California Occupational Safety and Health Administration (OSHA) to be shored with bracing equipment prior to being entered by any individuals, whether or not they are associated with the project.
4. **Trench Dewatering:** H&K does not anticipate that the proposed underground utility trenches will encounter shallow ground water in the upper terrace area, but will probably encounter shallow ground water in the lower terrace area. However, if the utility trenches are excavated during the winter rainy season, then shallow or perched ground water may be encountered in both the upper and lower terrace areas. The earthwork contractor may need to employ de-watering methods as discussed in Section 6.1.13 in order to excavate, place and compact the trench backfill materials.

5. **Pipe Zone Backfill Type And Compaction Requirements:** The backfill material type and compaction requirements for the pipe zone which includes the bedding zone, shading zone and cover zone as shown in the Figure 6.1.12 are described below.

FIGURE 6.1.12. TYPICAL TRENCH BACKFILL ZONES



Not To Scale

- **Pipe Zone Backfill Material Type:** Trench backfill used within the pipe zone which includes the bedding zone, shading zone and cover zone should consist of ¾-inch minus, washed, crushed rock. The crushed rock particle size gradation should meet the following requirements (percents are expressed as dry weights using ASTM D422 test method): 100 percent passing the 3/4 inch sieve, 80 to 100 percent passing the ½ inch sieve, 60 to 100 percent passing the 3/8 inch sieve, 0 to 30 percent passing the No. 4 sieve, 0 to 10 percent passing the No. 8 sieve, and 0 to 3 percent passing the No. 200 sieve. If ground water is encountered within the trench during construction or if it is expected to rise during the rainy season to a elevation that will infiltrate the pipe zone within the trench, then the pipe zone material should be wrapped with a minimum 6 ounce per square yard, non-woven, geotextile filter fabric such as Amoco 4506 manufactured by Amoco Fabrics and Fibers Company or equivalent should be used.. The geotextile seam should be located along the trench centerline and have a minimum 1-foot overlap. If the utility pipes are coated with a corrosion protection material, then the pipes should be wrapped with a minimum 6 ounce per square yard, non-woven, geotextile cushion fabric such as Amoco 4506 manufactured by Amoco Fabrics and Fibers Company or equivalent should be used. The geotextile cushion fabric should have a minimum 6 inch seam overlap. The geotextile cushion fabric will protect the pipe from being scratched by the crushed rock backfill material.

- **Pipe Bedding Zone Compaction:** Trench backfill soil placed in the pipe bedding zone (beneath the utilities) should be a minimum 3-inches thick, moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content and compacted to achieve a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density.
- **Pipe Shading Zone Compaction:** Trench backfill soil placed within the pipe-shading zone (above the bedding zone and to a height of one pipe radius length above the pipe spring line) should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. The pipe shading zone backfill material should be **shovel sliced** to remove voids and to promote compaction.
- **Pipe Cover Zone Compaction:** Trench backfill soil placed within the pipe cover zone (above the pipe shading zone to one foot over the pipe top surface) should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.

6. **Trench Zone Backfill And Compaction Requirements:** The trench zone backfill materials consists of both lower and upper zones as discussed below.

   - **Trench Zone Backfill Material Type:** Soil used as trench backfill within the lower and upper intermediate zones as shown on the preceding figure should consist of non-expansive soil with a plasticity index (PI) of less than or equal to PI ≤ 15 (based on ASTM D4318) and should not contain rocks greater than 3 inches in greatest dimension.
   - **Lower Trench Zone Compaction:** Soil used to construct the lower trench zone backfills should be uniformly moisture conditioned to within 0 to 4 percentage points of the ASTM D1557 optimum moisture content, placed in maximum 12-inch-thick loose lifts (layers) prior to compacting and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.

- **Upper Trench Zone Compaction (Road And Parking Lot Areas):** Soil used to construct the upper trench zone backfills should be uniformly moisture conditioned to within 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content, placed in maximum 8-inch-thick loose lifts (layers) prior to compacting and compacted to achieve a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density.
- **Upper Trench Zone Compaction (Non-Road And Non Parking Lot Areas):** Soil used to construct the upper trench zone backfills should be uniformly moisture conditioned to within 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content, placed in maximum 8-inch-thick loose lifts (layers) prior to compacting and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.

7. **CQA Testing And Observation Engineering Services:** The moisture content, dry density, and relative percent compaction of all engineered utility trench backfills should be tested by project engineer's field representative during construction to evaluate whether the compacted trench backfill material meet or exceed the minimum compaction and moisture content requirements presented in this report. The earthwork contractor shall assist the project engineer's field representative by excavating test pads with the on-site earth moving equipment.

- **Compaction Testing Frequencies:** The field and laboratory CQA tests should be performed consistent with the testing frequencies presented in Table 6.1.12 or as modified by the project engineer to better suit the site conditions.

Table 6.1.12, Minimum Testing Frequencies For Utility Trench Backfill

ASTM No.	Test Description	Minimum Test Frequency(1)
D1557	Modified Proctor Compaction Curve	1 per 500 CY (1) Or Material Change (2)
D2922	Nuclear Moisture Content	1 per 100 LF per 24-Inch-Thick Compacted Backfill Layer (3) The maximum loose lift thickness shall not exceed 12-inches prior to compacting.
D3017	Nuclear Density	1 per 100 LF per 24-Inch-Thick Compacted Backfill Layer (3) The maximum loose lift thickness shall not exceed 12-inches prior to compacting.

Notes: (1) These are minimum testing frequencies that may be increased or decreased at the project engineer's discretion on the basis of the site conditions encountered during grading.
(2) CY = cubic yards.
(3) Whichever criteria provide the greatest number of tests

- **Final Proof Rolling:** The prepared finished grade aggregate base (AB) rock surface and/or finished subgrade soil surface of utility trench backfills should be proof rolled with a fully loaded minimum 4,000 gallon capacity water truck with the rear of the truck supported on a double-axel, tandem-wheel, undercarriage or approved equivalent. The minimum tire pressure should be

65 pounds per square inch (psi). The proof rolled surface should be visually observed by the project engineer or the project engineer's field representative to be firm, competent and relatively unyielding. The project engineer or the project engineer's field representative may also evaluate the surface material by hand probing with a ¼-inch-diameter steel probe; however, this evaluation method should not be performed as a substitute for proof rolling as described in the preceding.

6.1.13 Construction De-watering

H&K does not anticipate the need to perform de-watering of the site during earthwork grading. However, the earthwork contractor should be prepared to de-water the utility trench excavations and any other excavations if perched water or the ground water table are encountered during grading. The following recommendations are preliminary and are not based on performing a ground water flow analysis. A detailed de-watering analysis was not a part of our proposed work scope. It should be understood that it is the earthwork contractor's sole responsibility to select and employ a satisfactory de-watering method for each excavation.

1. H&K anticipates that de-watering of utility trenches can be performed by constructing sumps to depths below the trench bottom and removing the water with sump pumps.
2. Additional sump excavations and pumps should be added as necessary to keep the excavation bottom free of standing water and relatively dry when placing and compacting the trench backfill materials.
3. If ground water enters the trench faster than it can be removed by the de-watering system thereby allowing the underlying compacted soil to become unstable while compacting successive soil lifts, then it may be necessary to remove the unstable soil and replace it with free draining, granular drain rock. Native backfill soil can again be used after placing the granular rock to an elevation that is higher than the ground water table.
4. If granular rock is used it should be wrapped in a non-woven geotextile fabric such as Amoco 4506 manufactured by Amoco Fabrics and Fibers Company or equivalent should be used. The geotextile filter fabric should have minimum 1-foot overlap seems. The granular rock should meet or exceed the following gradation specifications (all percents are expressed as dry weights using ASTM D422 test method): 100 percent passing the 3/4 inch sieve, 80 to 100 percent passing the ½ inch sieve, 60 to 100 percent passing the 3/8 inch sieve, 0 to 30 percent passing the No. 4 sieve, 0 to 10 percent passing the No. 8 sieve, and 0 to 3 percent passing the No. 200 sieve.
5. H&K recommends that the utility trench excavations be performed as late in the summer months as possible to allow the ground water table to reach its' lowest seasonal elevation.

6.1.14 Soil Corrosion Potential

The selected materials used for constructing underground utilities should be evaluated by a corrosion engineer for compatibility with the on site soil and ground water conditions. H&K did not perform a corrosion potential evaluation of the on site soil and ground water as part of our scope-of-services.

6.1.15 Subsurface Ground Water Drainage

H&K does anticipate encountering perched ground water or the local ground water table during the wet weather seasons. If groundwater is encountered during grading, then H&K should be allowed to observe the conditions and provide site-specific de-watering recommendations.

6.1.16 Surface Water Drainage

H&K recommends the following surface water drainage mitigation measures:

1. Grade all slopes drain away from building areas with a minimum 2 percent slope for a distance of not less than 10 feet from the building foundations.
2. Grade all landscape areas near and adjacent to buildings to prevent ponding of water.
3. Direct all building downspouts to solid (non-perforated) pipe collectors which discharge to natural drainage courses, storm sewers, catchment basins, infiltration subdrains, or other drainage facilities.

6.1.17 Grading Plan Review And Construction Monitoring

Construction quality assurance includes review of plans and specifications and performing construction monitoring as described below.

1. H&K should be allowed to review the final earthwork grading improvement plans prior to commencement of construction to determine whether our recommendations have been implemented, and if necessary, to provide additional and/or modified recommendations.
2. H&K should be allowed to perform construction quality assurance (CQA) monitoring of all earthwork grading performed by the contractor to determine whether our recommendations have been implemented, and if necessary, to provide additional and/or modified recommendations.
3. Our experience and that of our profession clearly indicates that during the construction phase of a project the risks of costly design, construction and maintenance problems can be significantly reduced by retaining the design geotechnical engineering firm to review the project plans and specifications and to provide geotechnical engineering construction quality assurance (CQA) observation and testing services. Upon your request we will prepare a CQA geotechnical engineering services proposal that will present a work scope,

tentative schedule and fee estimate for your consideration and authorization. If H&K is not retained to provide geotechnical engineering CQA services during the construction phase of the project, then H&K will not be responsible for geotechnical engineering CQA services provided by others nor any aspect of the project that fails to meet your or a third party's expectations in the future.

6.2 STRUCTURAL IMPROVEMENTS

Our structural improvement design criteria recommendations include: seismic design parameters, shallow continuous strip and isolated foundations, retaining walls, concrete slab-on-grade floors, patios and sidewalks, and pavement designs. These recommendations are presented here after.

6.2.1 Permanent Building Minimum Set-Backs From Bluffs

We recommend that a minimum 100-foot set-back distance from the top and bottom of the bluff slopes be established for construction of all permanent buildings. This minimum set-back distance will minimize the potential negative impacts on permanent buildings resulting from both shallow slumps and deep seated failures of the bluff slopes. We recommend that improvements within these set-back distances from the top and bottom of the bluff slopes be limited to parking areas and non-critical, temporary buildings.

6.2.2 Seismic Design Parameters

The site is not located within an Alquist-Priolo special study zone dealing with known fault ruptures as defined by “Preliminary Fault Activity Map of California,” California Department of Conservation, Division of Mines and Geology, 1992.

We used UBCSEIS a computer program developed by Thomas F. Blake to estimate the seismic coefficients for the site based on the 1997 Uniform Building Code. UBCSEIS performs fault searches using a modified version of the California fault-data file that was compiled by the California Division of Mines and Geology (CDMG). UBCSEIS estimates the following for each fault identified in the database relative to the project site location:

- Closest distance to fault.
- Closest Type A, B and C faults.
- Coefficients Na, Nv, Ca, and Cv for the Closest Type A, B and C faults.
- Coefficients Ts and To from the largest of the Na, Nv, Ca and Cv coefficients.
- Response spectrum from the selected coefficients.

The UBCSEIS computer program output file is included in Appendix E and is summarized in Table 6.2.2.

Table 6.2.2, UBC Seismic Design Parameters

Design Parameter	1997 UCB Volume 2 Source Table	Assigned Value
Seismic Map Zone	Figure 16-2, Page 2-37	Zone = 3
Seismic Zone Z Factor	Table 16-1, Page 2-30	Z = 0.30
Soil Profile Type	Table 16-J, Page 2-30	Sc = Very Dense Soil And Soft Rock
Occupancy Category	Table 16-K, Page 2-30	Standard Occupancy Structures
Seismic Coefficient C_a	Table 16-Q, Page 2-38	C_a = 0.33
Seismic Coefficient C_v	Table 16-R, Page 2-35	C_v =0.45
Seismic Source and Distance Bartlett Spings = 102.3 Km Hunting Creek, Berryessa = 102.7 Km	Table 16-U, Page 2-35	 Source Type = A Source Type = B
Near Source Factor N_a	Table 16-S, Page 2-35	N_a = 1.0
Near Source Factor N_v	Table 16-T, Page 2-35	N_v = 1.0

6.2.3 Shallow Continuous Strip And Stepped Foundations

Shallow continuous strip and stepped foundations for load bearing walls should be designed as follows:

1. The base of all shallow foundations should bear on firm competent non-expansive native soil, or either non-expansive engineered fill or expansive engineered fill compacted consistent with the earthwork recommendations of Section 5.1.
2. Continuous strip foundations should be constructed with the following dimensions:
    a. Minimum Width = 12 Inches
    b. Minimum Embedment Depth below the lowest adjacent exterior surface grade as shown in Table 6.2.3.
3. The allowable bearing capacities to be used for structural design of all shallow foundations founded in either non-expansive native soil or non-expansive engineered fill are presented in Table 6.2.3.

Table 6.2.3, Continuous Strip Foundation Maximum Bearing Pressures

Minimum Foundation Embedment Depth (inches)	Maximum Bearing Pressures For Live + Dead Loads (psf)	Maximum Bearing Pressures For Live + Dead + Wind or Seismic Loads (psf)
12	1,500	1,995
18	2,000	2,660
24	2,500	3,325
30	3,000	3,990
36	3,500	4,655

4. Foundation lateral resistance may be computed from passive pressure along the side of the foundation and sliding friction resistance along the foundation base; however the larger of the two resistance forces should be reduced by 50 percent

when combining these two forces. The passive pressure can be assumed to be equal to an equivalent fluid pressure per foot of depth. The passive pressure force and sliding friction coefficient for computing lateral resistance are as follows:

a. Passive pressure = 275 (H), where H = foundation depth below lowest adjacent soil surface.

b. Foundation bottom sliding friction coefficient = 0.40 (dimensionless).

5. Minimum steel reinforcement for continuous strip foundations should consist of two No. 4 bars (ASTM A615 Grade 60 billet steel) with one bar placed near the top and one bar placed near the bottom of each foundation or as designated by a California licensed structural engineer.
6. Concrete coverage over steel reinforcements should be a minimum of 3 inches as recommended by the American Concrete Institute (ACI).
7. Prior to placing concrete in any foundation excavations the contractor shall remove all loose soil, rock, wood, and debris, or other deleterious materials from the foundation excavations.
8. Foundation excavations should be saturated prior to placing concrete to aid the concrete curing process; however, concrete should not be placed in standing water.
9. Total settlement of individual foundations will vary depending on the plan dimensions of the foundation and actual structural loading. Based on the anticipated foundation dimensions and loads, we estimated that the total post-construction settlement of foundations designed and constructed in accordance with our recommendations will be on the order of 1/2 inch. Differential settlement between similarly loaded, adjacent foundations is expected to be about 1/4 inch, provided the foundations are founded into similar materials (e.g., all on competent and firm engineered fill, native soil or rock). Differential settlement between adjacent foundations founded on dissimilar materials (e.g., one foundation on soil and one on rock) may approach the maximum estimated total settlement of 1/2 inch. Settlement of all foundations are expected to occur rapidly and should be essentially complete shortly after the total design load has been applied.
10. Prior to placing concrete in any foundation excavation the project geotechnical engineer or his/her field representative should observe the excavations to document that the following requirements have been achieved: minimum foundation dimensions, minimum reinforcement steel placement, and dimensions, removal of all loose soil, rock, wood, and debris, or other deleterious materials, and that firm and competent native soil or engineered fill soil is exposed along the entire foundation excavation bottom. Strict adherence to these requirements is paramount to the satisfactory behavior of a building foundation. Minor deviations of these requirements can cause the foundations to undergo minor to severe

amounts of settlement which can result in cracks developing in the foundation and adjacent structural members such as concrete slab-on-grade floors.

6.2.4 Shallow Isolated Spread Foundations

Shallow isolated spread foundations (i.e., square, rectangular and circular) for column loads should be designed as follows:

1. The base of all shallow foundations should bear on firm competent non-expansive native soil or either non-expansive engineered fill or expansive engineered fill compacted consistent with the earthwork recommendations presented in Section 6.2.
2. Shallow isolated square, rectangular and circular spread foundations should be designed by a California licensed civil engineer with the following dimensions:
   a. Minimum Width and Length or Radius = dimensions should be determined such that the allowable bearing capacities presented herein are not exceeded.
   b. Minimum Embedment Depth below the lowest adjacent exterior surface grade as shown in Table 6.2.4.
3. The allowable bearing capacities to be used for structural design of all shallow isolated square, rectangular or circular foundations founded in either non-expansive native soil or non-expansive engineered fill are presented in Table 6.2.4.

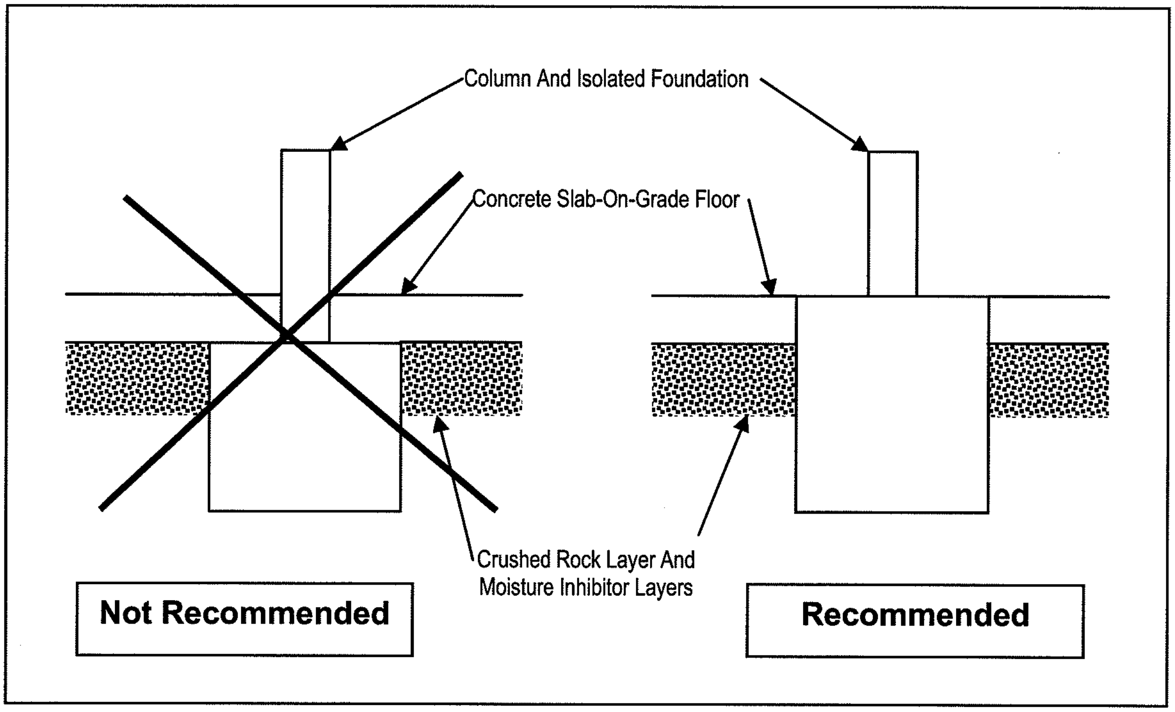
Table 6.2.4, Isolated Foundation Maximum Bearing Pressures

Minimum Foundation Embedment Depth (inches)	Maximum Bearing Pressures For Live + Dead Loads (psf)	Maximum Bearing Pressures For Live + Dead + Wind or Seismic Loads (psf)
12	1,500	1,995
18	2,000	2,660
24	2,500	3,325
30	3,000	3,990
36	3,500	4,655

4. Foundation lateral resistance may be computed from passive pressure along the side of the foundation and sliding friction resistance along the foundation base; however the larger of the two resistance forces should be reduced by 50 percent when combining these two forces. The passive pressure can be assumed to be equal to an equivalent fluid pressure per foot of depth. The passive pressure force and sliding friction coefficient for computing lateral resistance are as follows:
   a. Foundation bottom sliding friction coefficient = 0.40 (dimensionless).
   b. Passive pressure = 275 (H), where H = foundation depth below lowest adjacent soil surface.

5. Minimum steel reinforcement of all isolated square, rectangular and circular foundations should be designed by a California licensed structural engineer using ASTM A615 Grade 60 billet steel.
6. Concrete coverage over steel reinforcements should be a minimum of 3 inches as recommended by the American Concrete Institute (ACI);
7. Prior to placing concrete in any foundation excavations the contractor shall remove all loose soil, rock, wood, and debris, or other deleterious materials from the foundation excavations.
8. Foundation excavations should be saturated prior to placing concrete to aid concrete curing process; however, concrete should be not placed in standing water.
9. Total settlement of individual foundations will vary depending on the plan dimensions of the foundation and actual structural loading. Based on the anticipated foundation dimensions and loads, we estimated that the total post-construction settlement of foundations designed and constructed in accordance with our recommendations will be on the order of 1/2 inch. Differential settlement between similarly loaded, adjacent foundations is expected to be about 1/4 inch, provided the foundations are founded into similar materials (e.g., all on competent and firm engineered fill, native soil or rock). Differential settlement between adjacent foundations founded on dissimilar materials (e.g., one foundation on soil and one on rock) may approach the maximum estimated total settlement of 1/2 inch. Settlement of the all foundations are expected to occur rapidly and should be essentially complete shortly after the total design load has been applied.
10. Prior to placing concrete in any foundation excavation the project geotechnical engineer or his/her field representative should observe the excavations to document that the following requirements have been achieved: minimum foundation dimensions, minimum reinforcement steel placement, and dimensions, removal of all loose soil, rock, wood, and debris, or other deleterious materials, and that firm and competent native soil or engineered fill soil is exposed along the entire foundation excavation bottom. Strict adherence to these requirements is paramount to the satisfactory behavior of a building foundation. Minor deviations of these requirements can cause the foundations to undergo minor to severe amounts of settlement which can result in cracks developing in the foundation and adjacent structural members such as concrete slab-on-grade floors.
11. We do not recommend that concrete slab-on-grade floors be placed in direct contact with the top surface of isolated column concrete foundations. Our experience is that during curing period of the concrete slab-on-grade floors a significant thermal gradient may develop between the portions of the slab placed directly on the typically more massive isolated column concrete foundations and the portions of the slab placed over the wetted cushion sand, vapor-moisture

retarder membrane and crushed rock of the slab support layers. Additionally, during the curing period a source of water for the bottom of the concrete slab-on-grade floors will not be present for the portions of slab that are placed in direct contact with the top surface of the isolated column concrete foundations, unlike the portions of the slab placed over the wetted cushion sand layer. **The development of adverse thermal gradients and lack of curing water beneath portions of the slab may cause the development of significant orthogonal and/or circular shrinkage cracks in the floor slab around the isolated column foundations.**



6.2.5 Retaining Walls Design Parameters

A California licensed civil engineer should design all retaining walls with the following geotechnical engineering design criteria:

1. Retaining walls should be founded on firm competent native soil or engineered fill soil consistent with the requirements of Section 6.1.
2. The retaining wall should be designed by a California licensed civil engineer using the geotechnical engineering design parameters presented in Table 6.2.5.
3. The retaining wall backfill soil should be free draining material that meets or exceeds the material requirements of Section 6.2.6 and is placed and compacted consistent with the requirements of Section 6.2.6.

4. The static lateral earth pressures exerted on the retaining walls may be assumed to be equal to an equivalent fluid pressure per foot of depth below the top of the wall. The lateral pressures presented below do not include a safety factor, and assumes a free draining backfill (no hydrostatic forces acting on the wall) and no surcharge loads applied within a distance of 0.40H, where H equals the total vertical wall height.
5. The retaining wall backfill slope shall have a slope gradient no steeper than 2H:1V (horizontal to vertical slope ratio). If a steeper backfill slope ratio is desired then Holdrege & Kull should be notified and contracted to perform additional retaining wall designs.
6. The retaining wall foundation excavations should be saturated prior to placing concrete to aid the concrete curing process. However, concrete should not be placed in standing water.

Table 6.2.5, Retaining Wall Design Parameters

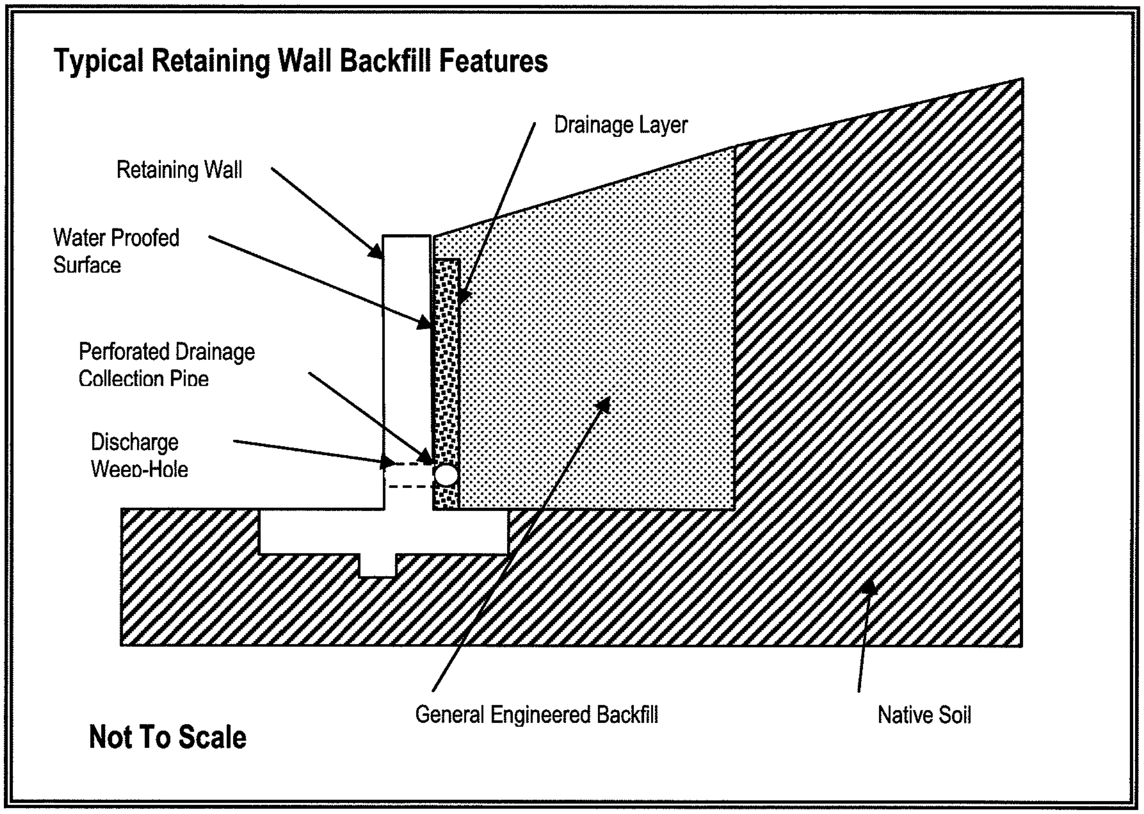
Loading Conditions	Retaining Wall With Horizontal Backfill Slope	Retaining Wall With Maximum 2H:1V Backfill Slope
Wall Active Pressures (psf) (1)	30 (H) (5)	45 (H)
Wall Passive Pressures (psf) (2)	275 (H)	275 (H)
Wall At-Rest Pressure (psf) (3)	50 (H)	65 (H)
Maximum Foundation Bearing Capacity (psf) (Live + Dead Loads)	2,000	2,000
Maximum Foundation Bearing Capacity (psf) (Live + Dead + Wind or Seismic Loads)	2,660	2,660
Minimum Foundation Embedment Depth (in)	18	18
Foundation Bottom Friction Coefficient (dim.) (4)	0.40	0.40

Notes:
(1) The active pressure condition applies to a retaining wall with an unrestrained top (deflection allowed).
(2) The passive condition applies to a retaining wall with soil resistance at the base. If passive pressures are used then H&K recommends that the top 1.0 feet of soil weight be ignored.
(3) The At-Rest pressure condition applies to a retaining wall with the top restrained (no deflection allowed).
(4) If the design horizontal resistance force acting on the wall foundation is computed by combining both the sliding friction force and passive soil pressure force, then the larger of the two forces should be reduced by 50 percent.
(5) H = The distance to a point in the backfill soil where the pressure is desired. The H distance is measured from the top of the wall for active and at-rest conditions and from one foot below the soil height at the toe of the wall (See Note 2 for passive condition).

6.2.6 Retaining Wall Backfill

Place and compact all retaining wall backfill and drainage layer materials as described below. Sub-structure retaining walls for below grade rooms, basements, garages, elevator shafts, etc. should also incorporate a water proofing sealant as described below. The water proofing sealant products should be installed by a qualified waterproofing contractor according to the manufacturer's directions. A typical retaining wall and backfill material zones is shown below.

1. **Water Proofing:** Water proofing materials should be installed behind retaining walls prior to backfilling if retaining walls will be constructed for below grade rooms, basements, garages, elevator shafts, etc. The water proofing materials should be installed by a qualified waterproofing contractor according to the manufacturer's directions.



2. **Drainage Layer:** A drainage layer should be placed between the wall and backfill material in order to prevent build up of hydrostatic pressures behind the wall. Additionally, care should be taken during placement of the drainage layer materials so as not to crush, tear, or damage the water proofing materials. The drainage layer can be constructed from drain rock, geosynthetic drain nets or a combination of both as described below.

   a. **Caltrans Class II Permeable Material Method:** Place a minimum 12-inch-thick layer of Caltrans Class II Permeable Material directly against the wall or water proofing system (as described below) without a geotextile wrapping to separate the backfill soil from the wall. The drainage material should extend form the wall bottom to within 12 inches of the wall top.

   b. **Geotextile Wrapped Drain Rock Method:** Place a minimum 12-inch-thick layer of drain rock wrapped in a geotextile filter fabric directly against the wall or water proofing system (as described below) to separate the backfill soil form the wall. The drain rock should extend from the wall bottom to within 12 inches of the wall top. A minimum 6-ounce per square yard (oz/sy) non-woven geo-textile fabric such as Amoco 4506 manufactured by Amoco Fabrics and Fibers Company or equivalent should be used.

c. **Geosynthetic Composite Drainnet (Geonet) Method:** Place a geosynthetic composite drain-net (geonet) directly against the wall or water proofing system (as described below) to separate the backfill soil from the wall. The composite geonet should extend from the wall bottom to within 12 inches of the wall top. A geosynthetic composite drainnet such a Hydroduct 200 or Hydroduct 220 distributed by Grace Construction Products or equivalent should be used.

3. **Drainage Layer Collection And Discharge Pipes:** A minimum 4-inch-diameter polyvinyl chloride (PVC) perforated drainpipe should be placed at the wall base inside the geotextile wrapped drain rock or wrapped by the composite geonet. Four 1/4–inch-diameter perforations should be drilled into the pipe. The perforations should be orientated in cross section view at 90 degrees to one another and along the pipe length on 6-inch-centers. A minimum of 3 inches of drain rock should be placed below the perforated PVC pipe. The pipe should direct water away from the wall by gravity with a minimum 1 percent slope. The pipe should collect ground water collected by the drainage layer discharge to the surface at the end of the wall or through weep-hole penetrations through the wall.
4. **General Backfill Placement Equipment:** Heavy conventional motorized compaction equipment should not be used directly adjacent to the retaining walls unless the wall is designed with sufficient steel reinforcements and/or bracing to resist the additional lateral pressures. Compaction of backfill materials within 5 feet of the retaining wall should be accomplished by light-weight hand operated, walk behind, and vibratory equipment. Additionally, care should be taken during placement of the general backfill materials so as not to crush, tear, or damage the water proofing and/or drainage layer materials.
5. **General Backfill Compaction:** The retaining wall backfill material placed between the drainage layer and temporary cut-slope should be compacted to a minimum of 90 percent and a maximum of 95 percent of the ASTM D1557 maximum dry density. If the backfill material is classified by the USCS as a coarse-grained material (i.e., GP, GW, GC, GM, SP, SW, SC, and SM) then it should be moisture conditioned to between ± 3 percentage points of the ASTM D1557 optimum moisture content. If the backfill material is classified by the USCS as a fine-grained material (i.e., CL, CH, ML, or MH) then it should be moisture conditioned to between 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content.

6.2.7 Concrete Slab-On-Grade Floors

In general, H&K recommends that subgrade elevations on which the concrete slab-on-grade floors are constructed be a minimum of 6 inches above the elevation of the surrounding parking and landscaped areas. Elevating the building will reduce the potential for subsurface water to enter beneath the concrete slab-on-grade floors and exterior surfaces and underground utility trenches.

The concrete slab-on-grade building floors, patios, sidewalks and garbage collection dumpster areas should be evaluated by a California licensed civil engineer for expected live and dead loads to determine if the minimum slab thickness and steel reinforcement recommendations presented in this report should be increased or redesigned.

6.2.7.1 Interior Floors

The interior concrete slab-on-grade floor components are described below from top to bottom. If static or intermittent live floor loads greater than 250 psf are anticipated, then a California licensed structural engineer should design the necessary concrete slab-on-grade floor thickness and steel reinforcements.

1. Minimum 4-Inch-Thick Concrete Slab: should be installed with a minimum 2,500 pounds per square inch (psi) compressive strength after 28 days of curing. H&K recommends that the concrete design uses and water to cement ratio of no greater than 0.42. The concrete mix design is the responsibility of the concrete supplier.
2. Prior to applying construction loads all exposed concrete slab-on-grade floors should be moisture cured for a minimum of 7 days following placement of the concrete. If concrete is placed during the hot summer months when the ambient air temperatures may be as low as 50 to 60 degrees Fahrenheit (F) in the early morning and in excess of 90 degrees F in the afternoon, then the contractor may need to implement special curing measures to minimize the development of shrinkage cracks. The concrete contractor is responsible for determining the appropriate curing process to be applied to the slab-on-grade floor.

   Concrete Slabs In Contact With Isolated Concrete Foundations: We do not recommend that concrete slab-on-grade floors be placed in direct contact with the top surface of isolated column concrete foundations. Our experience is that during curing period of the concrete slab-on-grade floors a significant thermal gradient may develop between the portions of the slab placed directly on the typically more massive isolated column concrete foundations and the portions of the slab placed over the wetted cushion sand, vapor-moisture retarder membrane and crushed rock of the slab support layers. Additionally, during the curing period a source of water for the bottom of the concrete slab-on-grade floors will not be present for the portions of slab that are placed in direct contact with the top surface of the isolated column concrete foundations, unlike the portions of the slab placed over the wetted cushion sand layer. **The development of adverse thermal gradients and lack of curing water beneath portions of the slab may cause the development of significant orthogonal and/or circular shrinkage cracks around the isolated column foundations. See report section 6.2.4 (Shallow Isolated Spread Foundations) for clarification figure.**

3. <u>Steel Reinforcements</u>: should be used to improve the load carrying capacity and to minimize cracking caused by shrinkage during curing and from both differential and repeated loadings. <u>It should be understood that it is nearly impossible to prevent all cracks from development in concrete slabs; in other words, it should be expected that some cracking will occur in all concrete slabs no matter how well they are reinforced.</u> Concrete slabs that will be subjected to heavy loads should be designed with steel reinforcements by a California licensed structural engineer.

   <u>Steel Rebar</u>: Use No. 3 ribbed steel rebar (ASTM A615 Grade 60 billet steel), tied and placed with 12-inch centers in both directions (perpendicular) and supported on concrete "dobies" to position the rebar in the center of the slab during concrete pouring. **We do not recommend that the steel reinforcements of the concrete slab-on-grade floor be tied into the perimeter or interior continuous strip foundations or interior isolated column foundations.** In other words, we recommend that the concrete slab-on-grade floors be constructed as independent structural members so that they can move (float) independently from the foundation structures.

4. <u>Minimum 2-Inch-Thick Sand Layer</u>: should be installed and compacted with a minimum of two passes with a walk behind vibratory plate compactor. Just prior to placing the concrete on the sand layer, the sand layer should be moistened to a saturated surface dry (SSD) condition. This measure will reduce the potential for water to be withdrawn from the bottom of the concrete slab while it is curing and will help minimize the development of shrinkage cracks and slab curling.

5. <u>Underslab Vapor-Moisture Retarder Membrane</u>: should be placed as a floor component that will minimize transmission of both liquid water and water vapor transmission through the concrete slab-on-grade floor. H&K recommends using a minimum 10-mil-thick, plastic, vapor-moisture, retarder membrane material such as: Moiststop® underslab vapor retarder membranes or equivalents. Additionally, the following materials are recommended: Moiststop® Tape™ to seal membrane joints and Moiststop® The Boot™ to seal pipe, conduit or other membrane penetrations, or equivalents.

   Regardless of the type of moisture-vapor retarder membrane used, moisture can wick up through a concrete slab-on-grade floor. Excessive moisture transmission through a concrete slab floor can cause adhesion loss, warping and peeling of resilient floor coverings, deterioration of adhesive, seam separation, formation of air pockets, mineral deposition beneath flooring, odor and both fungi and mold growth. Slabs can be tested for water transmissivity in areas that are moisture sensitive. Commercial sealants, polymer additives to the concrete at the batch plant, entrained air, flyash, and a reduced water to content ratio can be incorporated into the concrete slab-on-grade floor to reduce its permeability and water-vapor transmissivity properties. A waterproofing consultant should be contacted to provide detailed recommendations if moisture sensitive flooring materials will be installed on the concrete slab-on-grade floors.

6. <u>Minimum 4-Inch-Thick Crushed Rock Layer</u>: should be placed and compacted to a minimum of 95 percent of the ASTM D1557 dry density with a moisture content of ± 3 percentage points of the ASTM D1557 optimum moisture content. The crushed rock should be washed to produce an ASTM D422 test particle size distribution of 100 percent (by dry weight) passing the ¾ inch sieve and 0 to 5 percent passing the No. 4 sieve and 0 to 3 percent passing the No. 200 sieve. This relatively clean (washed) crushed rock will act as a capillary break for free water moisture transmission, as well as, provide a uniform bearing surface for the concrete slab-on-grade floor.

7. <u>Subgrade Soil Preparation</u>: The subgrade soil should be prepared and compacted consistent with the recommendations of Section 5.1. The top 12 inches of the non-expansive soil should be compacted to a minimum of 90 percent of the ASTM D1557 dry density with a relatively uniform moisture content of from 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content. If the expansive on-site soil is used it should be compacted to between 88 percent and 92 percent of the ASTM D1557 maximum dry density with a relatively uniform moisture content of from 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content.

   Prior to placing concrete and the moisture barrier membrane, but after placing the overlying crushed rock layer, the subgrade soil must be moisture conditioned to achieve a saturation of between 75 and 100 percent to a depth of 18 inches below the finished subgrade surface. Moisture conditioning should be performed for a minimum of 18 hours prior to concrete placement. If the soil is not moisture conditioned prior to placing concrete, moisture could be wicked (transmitted) out of the concrete by the underlying potentially dryer soil, which could cause shrinkage cracks to develop in the concrete slab during the curing period.

   Additionally, we believe that moisture conditioning the subgrade soil will reduce the swell (heave) potential of fine-grained soil with moderate to high expansion properties. Typically, concrete slabs impart relatively small loads on the order of about 50 pounds per square foot (psf) on the underlying subgrade soil. Therefore, some vertical movement of the concrete slab should be anticipated from possible expansion of the underlying subgrade soil, if it is not properly moisture conditioned as describe in the preceding.

8. Crack Control Grooves: should be installed during placement or saw cuts should be made in accordance with the American Concrete Institute (ACI) and Portland Cement Association (PCA) specifications. Generally, H&K recommend that expansion joints be provided between the slab and perimeter footings, and that crack control groves or saw cuts are installed on 10-foot-centers in both directions (perpendicular).

9. Field Observations: should be made by an H&K construction monitor of all concrete slab-on-grade subgrade surfaces and installed steel reinforcements prior to placing concrete.

6.2.7.2 Exterior Sidewalks, And Patios

The exterior concrete slab-on-grade surfaces components are described below from top to bottom. If static or intermittent live loads greater than 250 psf are anticipated, or if heavy traffic loads are anticipated then a California licensed structural engineer should design the necessary concrete slab-on-grade floor thickness and steel reinforcements.

1. Minimum 4-Inch-Thick Concrete Slab: should be installed with a minimum 2,500 pounds per square inch (psi) compressive strength after 28 days of curing. H&K recommends that the concrete design uses and water to cement ratio of no greater than 0.42. The concrete mix design is the responsibility of the concrete supplier.

   Prior to applying construction loads all exposed concrete slab-on-grade floors should be moisture cured for a minimum of 7 days following placement of the concrete. If concrete is placed during the hot summer months when the ambient air temperatures may be as low as 50 to 60 degrees Fahrenheit (F) in the early morning and in excess of 90 degrees F in the afternoon, then the contractor may need to implement special curing measures to minimize the development of shrinkage cracks. The concrete contractor is responsible for determining the appropriate curing process to be applied to the slab-on-grade floors.

   Concrete Slabs In Contact With Isolated Concrete Foundations: We do not recommend that concrete slab-on-grade floors be placed in direct contact with the top surface of isolated column concrete foundations. Our experience is that during curing period of the concrete slab-on-grade floor a significant thermal gradient may develop between the portions of the slab placed directly on the typically more massive isolated column concrete foundations and the portions of the slab placed over the wetted cushion sand, vapor-moisture retarder membrane and crushed rock layers. Additionally, during the curing period a source of water for the bottom of the concrete slab-on-grade floor will not be present for the portions of slab that are placed in direct contact with the top surface of the isolated column concrete foundations, unlike the portions of the slab placed over the wetted cushion sand layer. **The development of adverse thermal gradients and lack of curing water beneath portions of the slab may cause the development of significant**

orthogonal and/or circular shrinkage cracks around the isolated column foundations. See report section 6.2.4 (Shallow Isolated Spread Foundations) for clarification figure.

2. Steel Reinforcements: should be used to improve the load carrying capacity and to minimize cracking caused by shrinkage during curing and from both differential and repeated loadings. It should be understood that it is nearly impossible to prevent all cracks from development in concrete slabs; in other words, it should be expected that some cracking will occur in all concrete slabs no matter how well they are reinforced. Concrete slabs that will be subjected to heavy loads should be designed with steel reinforcements by a California licensed structural engineer.

   If the current property owner (developer) elects to eliminate the steel reinforcements from the exterior concrete slabs-on-grade for economic reasons, then there will be an inherent greater risk assumed by the developer for the development of both shrinkage and bearing related cracks in the associated slabs.

   Steel Rebar: Use No. 3 ribbed steel rebar (ASTM A615 Grade 60 billet steel), tied and placed with 18-inch centers in both directions (perpendicular) and supported on concrete "dobies" to position the rebar in the center of the slab during concrete pouring. **We do not recommend that the steel reinforcements of the concrete slab-on-grade floor be tied into adjacent building perimeter or isolated column foundations.** In other words, we recommend that the concrete slab-on-grade floors be constructed as independent structural members so that they can move (float) independently from the foundation structures.

3. Minimum 4-Inch-Thick Crushed Rock Layer: should be placed and compacted to a minimum of 95 percent of the ASTM D1557 dry density with a moisture content of ± 3 percentage points of the ASTM D1557 optimum moisture content. The crushed rock should be washed to produce a particle size distribution of 100 percent (by dry weight) passing the ¾ inch sieve and 5 percent passing the No. 4 sieve and 0 to 3 percent passing the No. 200 sieve. This relatively clean (washed) crushed rock will act as a capillary break for free water moisture transmission, as well as, provide a uniform bearing surface for the concrete slab-on-grade floor. However, just prior to pouring the concrete slab the crushed rock layer should be moistened to a saturated surface dry (SSD) condition. This measure will reduce the potential for water to be withdrawn from the bottom of the concrete slab while it is curing and will help minimize the development of shrinkage cracks.

   If the current property owner (developer) elects to eliminate the crushed rock layer beneath the exterior concrete slabs-on-grade for economic reasons, then there will be an inherent greater risk assumed by the developer for the development of both shrinkage and bearing related cracks in the associated slabs.

4. Subgrade Soil Preparation: The subgrade soil should be prepared and compacted consistent with the recommendations of Section 5.1. The top 12 inches of the non-expansive soil should be compacted to a minimum of 90 percent of the ASTM D1557 dry density with a moisture content of from 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content. If the expansive on-site soil is used it should be compacted to between 88 percent and 92 percent of the ASTM D1557 maximum dry density with a relatively uniform moisture content of from 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content.

   Prior to placing concrete, but after placing the overlying crushed rock layer, the subgrade soil must be moisture conditioned to achieve a saturation of between 75 and 100 percent to a depth of 18 inches below the finished subgrade surface. Moisture conditioning should be performed for a minimum of 18 hours prior to concrete placement. If the soil is not moisture conditioned prior to placing concrete, moisture could be wicked (transmitted) out of the concrete by the underlying potentially dryer soil, which could cause shrinkage cracks to develop in the concrete slab during the curing period.

   Additionally, we believe that moisture conditioning the subgrade soil will reduce the swell (heave) potential of fine-grained soil with moderate to high expansion properties. Typically, concrete slabs impart relatively small loads on the order of about 50 pounds per square foot (psf) on the underlying subgrade soil. Therefore, some vertical movement of the concrete slab should be anticipated from possible expansion of the underlying subgrade soil, if it is not properly moisture conditioned as describe in the preceding.

5. Crack Control Grooves: should be installed during placement or saw cuts should be made in accordance with the American Concrete Institute (ACI) and Portland Cement Association (PCA) specifications. Generally, H&K recommend that expansion joints be provided between the slab and perimeter footings, and that crack control groves or saw cuts be installed on 10-foot-centers in both directions (perpendicular).

6. Field Observations: should be made by an H&K construction monitor of all concrete slab-on-grade surfaces and installed steel reinforcements prior to pouring concrete.

6.2.7.3 Garage Floors And Driveways

The concrete slab-on-grade floor components are described below from top to bottom. If static or intermittent live floor loads greater than 250 psf are anticipated, then a California licensed structural engineer should design the necessary concrete slab-on-grade floor thickness and steel reinforcements.

1. Minimum 6-Inch-Thick Concrete Slab: should be installed with a minimum 2,500 pounds per square inch (psi) compressive strength after 28 days of curing. H&K

recommends that the concrete design uses and water to cement ratio of no greater than 0.42. The concrete mix design is the responsibility of the concrete supplier.

Prior to applying construction loads all exposed concrete slab-on-grade floors should be moisture cured for a minimum of 7 days following placement of the concrete. If concrete is placed during the hot summer months when the ambient air temperatures may be as low as 50 to 60 degrees Fahrenheit (F) in the early morning and in excess of 90 degrees F in the afternoon, then the contractor may need to implement special curing measures to minimize the development of shrinkage cracks. The concrete contractor is responsible for determining the appropriate curing process to be applied to the slab-on-grade floor.

Concrete Slabs In Contact With Isolated Concrete Foundations: We do not recommend that concrete slab-on-grade floors be placed in direct contact with the top surface of isolated column concrete foundations. Our experience is that during curing period of the concrete slab-on-grade floors a significant thermal gradient may develop between the portions of the slab placed directly on the typically more massive isolated column concrete foundations and the portions of the slab placed over the wetted cushion sand, vapor-moisture retarder membrane and crushed rock of the slab support layers. Additionally, during the curing period a source of water for the bottom of the concrete slab-on-grade floors will not be present for the portions of slab that are placed in direct contact with the top surface of the isolated column concrete foundations, unlike the portions of the slab placed over the wetted cushion sand layer. **The development of adverse thermal gradients and lack of curing water beneath portions of the slab may cause the development of significant orthogonal and/or circular shrinkage cracks around the isolated column foundations. See report section 5.2.3 (Shallow Isolated Spread Foundations) for clarification figure.**

2. Steel Reinforcements: should be used to improve the load carrying capacity and to minimize cracking caused by shrinkage during curing and from both differential and repeated loadings. It should be understood that it is nearly impossible to prevent all cracks from development in concrete slabs; in other words, it should be expected that some cracking will occur in all concrete slabs no matter how well they are reinforced. Concrete slabs that will be subjected to heavy loads should be designed with steel reinforcements by a California licensed structural engineer.

   Steel Rebar: Use No. 3 ribbed steel rebar (ASTM A615 Grade 60 billet steel), tied and placed with 12-inch centers in both directions (perpendicular) and supported on concrete "dobies" to position the rebar in the center of the slab during concrete pouring. **We do not recommend that the steel reinforcements of the concrete slab-on-grade floor be tied into the perimeter or interior continuous strip foundations or interior isolated column foundations.** In other words, we recommend that the concrete slab-on-grade floors be constructed as independent structural members so that they can move (float) independently from the foundation structures.

3. Minimum 6-Inch-Thick Crushed Rock Layer: should be placed and compacted to a minimum of 95 percent of the ASTM D1557 dry density with a moisture content of ± 3 percentage points of the ASTM D1557 optimum moisture content. The crushed rock should be washed to produce an ASTM D422 test particle size distribution of 100 percent (by dry weight) passing the ¾ inch sieve and 0 to 5 percent passing the No. 4 sieve and 0 to 3 percent passing the No. 200 sieve. This relatively clean (washed) crushed rock will act as a capillary break for free water moisture transmission, as well as, provide a uniform bearing surface for the concrete slab-on-grade floor.

4. Subgrade Soil Preparation: The subgrade soil should be prepared and compacted consistent with the recommendations of Section 5.1. The top 12 inches of the non-expansive soil should be compacted to a minimum of 95 percent of the ASTM D1557 dry density with a relatively uniform moisture content of from 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content. If the expansive on-site soil is used it should be compacted to between 88 percent and 92 percent of the ASTM D1557 maximum dry density with a relatively uniform moisture content of from 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content.

   Prior to placing concrete and the moisture barrier membrane, but after placing the overlying crushed rock layer, the subgrade soil must be moisture conditioned to achieve a saturation of between 75 and 100 percent to a depth of 18 inches below the finished subgrade surface. Moisture conditioning should be performed for a minimum of 18 hours prior to concrete placement. If the soil is not moisture conditioned prior to placing concrete, moisture could be wicked (transmitted) out of the concrete by the underlying potentially dryer soil, which could cause shrinkage cracks to develop in the concrete slab during the curing period.

   Additionally, we believe that moisture conditioning the subgrade soil will reduce the swell (heave) potential of fine-grained soil with moderate to high expansion properties. Typically, concrete slabs impart relatively small loads on the order of about 50 pounds per square foot (psf) on the underlying subgrade soil. Therefore, some vertical movement of the concrete slab should be anticipated from possible expansion of the underlying subgrade soil, if it is not properly moisture conditioned as describe in the preceding.

5. Crack Control Grooves: should be installed during placement or saw cuts should be made in accordance with the American Concrete Institute (ACI) and Portland Cement Association (PCA) specifications. Generally, H&K recommend that expansion joints be provided between the slab and perimeter footings, and that crack control groves or saw cuts are installed on 10-foot-centers in both directions (perpendicular).

6. Field Observations: should be made by an H&K construction monitor of all concrete slab-on-grade subgrade surfaces and installed steel reinforcements prior to placing concrete.

6.2.7.4 Loading Docks And Garbage Bin Enclosures

The concrete slab-on-grade floor components are described below from top to bottom. If static or intermittent live floor loads greater than 250 psf are anticipated, then a California licensed structural engineer should design the necessary concrete slab-on-grade floor thickness and steel reinforcements.

1. Minimum 8-Inch-Thick Concrete Slab: should be installed with a minimum 2,500 pounds per square inch (psi) compressive strength after 28 days of curing. H&K recommends that the concrete design uses and water to cement ratio of no greater than 0.42. The concrete mix design is the responsibility of the concrete supplier.

   Prior to applying construction loads all exposed concrete slab-on-grade floors should be moisture cured for a minimum of 7 days following placement of the concrete. If concrete is placed during the hot summer months when the ambient air temperatures may be as low as 50 to 60 degrees Fahrenheit (F) in the early morning and in excess of 90 degrees F in the afternoon, then the contractor may need to implement special curing measures to minimize the development of shrinkage cracks. The concrete contractor is responsible for determining the appropriate curing process to be applied to the slab-on-grade floor.

   Concrete Slabs In Contact With Isolated Concrete Foundations: We do not recommend that concrete slab-on-grade floors be placed in direct contact with the top surface of isolated column concrete foundations. Our experience is that during curing period of the concrete slab-on-grade floors a significant thermal gradient may develop between the portions of the slab placed directly on the typically more massive isolated column concrete foundations and the portions of the slab placed over the wetted cushion sand, vapor-moisture retarder membrane and crushed rock of the slab support layers. Additionally, during the curing period a source of water for the bottom of the concrete slab-on-grade floors will not be present for the portions of slab that are placed in direct contact with the top surface of the isolated column concrete foundations, unlike the portions of the slab placed over the wetted cushion sand layer. **The development of adverse thermal gradients and lack of curing water beneath portions of the slab may cause the development of significant orthogonal and/or circular shrinkage cracks around the isolated column foundations. See report section 5.2.3 (Shallow Isolated Spread Foundations) for clarification figure.** **See report section 5.2.3 (Shallow Isolated Spread Foundations) for clarification figure.**

2. Steel Reinforcements: should be used to improve the load carrying capacity and to minimize cracking caused by shrinkage during curing and from both differential

and repeated loadings. It should be understood that it is nearly impossible to prevent all cracks from development in concrete slabs; in other words, it should be expected that some cracking will occur in all concrete slabs no matter how well they are reinforced. Concrete slabs that will be subjected to heavy loads should be designed with steel reinforcements by a California licensed structural engineer.

Steel Rebar: Use No. 4 ribbed steel rebar (ASTM A615 Grade 60 billet steel), tied and placed with 12-inch centers in both directions (perpendicular) and supported on concrete "dobies" to position the rebar in the center of the slab during concrete pouring. **We do not recommend that the steel reinforcements of the concrete slab-on-grade floor be tied into the perimeter or interior continuous strip foundations or interior isolated column foundations.** In other words, we recommend that the concrete slab-on-grade floors be constructed as independent structural members so that they can move (float) independently from the foundation structures.

3. Minimum 8-Inch-Thick Crushed Rock Layer: should be placed and compacted to a minimum of 95 percent of the ASTM D1557 dry density with a moisture content of ± 3 percentage points of the ASTM D1557 optimum moisture content. The crushed rock should be washed to produce an ASTM D422 test particle size distribution of 100 percent (by dry weight) passing the ¾ inch sieve and 0 to 5 percent passing the No. 4 sieve and 0 to 3 percent passing the No. 200 sieve. This relatively clean (washed) crushed rock will act as a capillary break for free water moisture transmission, as well as, provide a uniform bearing surface for the concrete slab-on-grade floor.

4. Subgrade Soil Preparation: The subgrade soil should be prepared and compacted consistent with the recommendations of Section 5.1. The top 12 inches of the non-expansive soil should be compacted to a minimum of 95 percent of the ASTM D1557 dry density with a relatively uniform moisture content of from 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content. If the expansive on-site soil is used it should be compacted to between 88 percent and 92 percent of the ASTM D1557 maximum dry density with a relatively uniform moisture content of from 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content.

   Prior to placing concrete and the moisture barrier membrane, but after placing the overlying crushed rock layer, the subgrade soil must be moisture conditioned to achieve a saturation of between 75 and 100 percent to a depth of 18 inches below the finished subgrade surface. Moisture conditioning should be performed for a minimum of 18 hours prior to concrete placement. If the soil is not moisture conditioned prior to placing concrete, moisture could be wicked (transmitted) out of the concrete by the underlying potentially dryer soil, which could cause shrinkage cracks to develop in the concrete slab during the curing period.

Additionally, we believe that moisture conditioning the subgrade soil will reduce the swell (heave) potential of fine-grained soil with moderate to high expansion properties. Typically, concrete slabs impart relatively small loads on the order of about 50 pounds per square foot (psf) on the underlying subgrade soil. Therefore, some vertical movement of the concrete slab should be anticipated from possible expansion of the underlying subgrade soil, if it is not properly moisture conditioned as describe in the preceding.

5. Crack Control Grooves: should be installed during placement or saw cuts should be made in accordance with the American Concrete Institute (ACI) and Portland Cement Association (PCA) specifications. Generally, H&K recommend that expansion joints be provided between the slab and perimeter footings, and that crack control groves or saw cuts are installed on 10-foot-centers in both directions (perpendicular).
6. Field Observations: should be made by an H&K construction monitor of all concrete slab-on-grade subgrade surfaces and installed steel reinforcements prior to placing concrete.

6.2.8 Pavement Design And Construction

The design and construction of asphalt concrete (AC) pavements for the project site are discussed below.

6.2.8.1 Asphalt Concrete Pavement Design

H&K used the Caltrans Design Method D301 to develop several asphalt concrete (AC) pavement and aggregate base (AB) rock design alternatives to allow for different traffic loading conditions. H&K used a Traffic Index (TI) of from 4 to 8 which represents typical light passenger vehicle traffic to heavy truck traffic. The actual TI for the project pavement areas should be determined in accordance with Chapter 600 of the Caltrans Highway Design Manual.

H&K was not able to obtain samples of the on-site soil and rock during our field investigation due to the rocky nature of the soil material on the property. Therefore no R-Value test results are included in this report. We recommend using an R-Value of 50 for both the upper bluff Sandy Gravel (GM) and The actual subsurface soil conditions exposed at the finished subgrade surface of the roadways may be different from this R-Value. Please note that the Caltrans design method requires that the maximum R-Value of the subgrade soil not exceed 50.

H&K assumed that the pavement layers will be constructed with Class 2 Aggregate Base Rock (Minimum R-Value = 78) and Type A Asphalt Concrete in accordance with the requirements of Section 26 of the Caltrans Standard Specifications. Table 6.2.8.1 presents the road, driveway, and parking pavement design section. H&K recommends that the AB rock layer be constructed with a minimum thickness of

6-inches for constructability issues and to achieve a higher level of confidence that the road will achieve the expected service life.

Table 6.2.8.1, Flexible Pavement Design

Parameters	Design Values				
Traffic Description (approximate)	Light Automobiles	Light to Medium Autos and Trucks	Medium to Heavy Trucks	Heavy Trucks	Very Heavy Trucks
Traffic Index (TI)	4	5	6	7	8
Design R-Values Class II AB Rock Subgrade Soil	 78 50	 78 50	 78 50	 78 50	 78 50
AC Thickness (inch)	2.50	3.00	3.50	4.00	5.00
AB Rock Thickness (inch) (95% Relative Compaction)	2.00	3.00	4.00	5.00	6.00
Subgrade Soil Thickness (inch) (95% Relative Compaction)	12.0	12.0	12.0	12.0	12.0
Note: H&K recommends that the minimum thickness of AB rock should be 6 inches regardless of what the Caltrans design method indicates. This minimum thickness is necessary for constructability issues and will increase the level of confidence that the roads will achieve the expected service life					

The subgrade soil and AB rock should be placed and compacted as described below.

1. The subgrade soil to a depth of 12 inches from the finished grade surface should be compacted to a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density with a moisture content of ± 3 percentage points of the ASTM D1557 optimum moisture content. The compacted sub-grade soil shall be graded to achieve the design grades and tolerances. The native sub-grade shall be graded to within +0.00-feet higher and -0.10-feet lower than the design grade.
2. The stability of the compacted subgrade soil should be evaluated by proof wheel rolling prior to placing the overlying AB rock layer. Proof wheel rolling should be performed with a fully loaded 4,000-gallon water truck with tire pressures between 60 and 95 pounds per square inch (psi). The subgrade soil surface should exhibit only minor deflections as the wheel load passes by any given location. Any unstable areas should be reworked and then retested for percent relative compaction and percent moisture content and then proof rolled again. This process should be repeated until the area appears to be relatively stable.
3. The Caltrans Class II AB rock should be compacted to a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density with a moisture content of ± 3 percentage points of the ASTM D1557 optimum moisture content. The aggregate base rock sub-grade surface shall be graded to within +0.00-feet higher and -0.05-feet lower than the design grade surface.

4. The stability of the compacted AB rock should be evaluated by proof wheel rolling prior to placing the overlying AC layer. Proof wheel rolling should be performed with a fully loaded 4,000-gallon water truck with tire pressures between 60 and 95 psi. The AB rock surface should exhibit only minor deflections as the wheel load passes by any given location. Any unstable areas should be reworked and then retested for percent relative compaction and percent moisture content and then proof rolled again. This process should be repeated until the area appears to be relatively stable.
5. Concrete cut-off curbs should be constructed around all landscaped areas that are adjacent to AC paved driveways and parking areas. The curbs should extend to a minimum depth of 8 inches into the underlying subgrade soil. The extended curbs will reduce migration of irrigation and rain waters originating in the landscaped areas form entering the AB rock materials underlying the AC pavement material. This design is intended to minimize failures of the paved areas due to saturation of the underlying AB rock and subgrade soils.
6. Edge drains can also be installed along both sides of the paved street alignment to a minimum depth of 8 inches into the underlying subgrade soil. All edge drains should discharge to the surface into natural drainage courses or conveyance channels or into a storm drain drop inlet structure. Edge drains are highly effective in intercepting and removing irrigation water and ground water and minimizing their potential negative impacts on the AB rock and subgrade soils underlying the paved street areas.

6.2.8.2 <u>Asphalt Concrete Pavement Construction</u>

1. Asphalt concrete (AC) pavement shall be constructed as required in current Section 39 of the Caltrans Standard Specifications that are in effect at the time the pavement is placed and these requirements.
2. Asphalt Concrete Materials: Asphalt concrete shall comply with the following criteria:
   - Asphalt concrete shall be a Type "A" Medium gradation. The maximum nominal aggregate size shall be 1/2 inch for residential collector and 3/4 inch for arterial streets. Streets that will allow speed limits greater than 45 MPH shall have a surface course with a maximum nominal aggregate size of 3/4-inch unless otherwise directed by the project engineer. Asphalt concrete base courses that are in excess of 2.25-inches-thick may use a maximum nominal aggregate size of 3/4 inches.
   - Asphalt concrete samples shall be taken for mixture verification testing in accordance with CTM 125. The location of each sample shall be noted on the test report.
   - Asphalt concrete mixture verification tests shall be performed at the rate of one set of tests per each 250-tons of AC placed and compacted. A minimum of one test shall be performed for each day of paving.

- The following mixture tests shall be performed on each AC bulk sample:

Table 6.2.8.2.A, Asphalt Concrete Testing

Test Method	Description	Requirement
CTM202	Sieve Analysis Of Fine And Coarse Aggregates	Operating Range And Contract Compliance Range
CTfM304	Preparation Of Bituminous Mixtures For Testing	Not Applicable
CTM308	Bulk Specific Gravity And Density	Maximum Values
CTM309	Theoretical Maximum Specific Gravity And Density	Maximum Values
CTM310	Asphalt And Moisture Content(1)	±0.5 percent of design mix
CTM366	Stabilometer Value	Minimum = 35
CTM367	Optimum Bitumen Content	Mix Voids = 3 to 5 percent
CTM375	In-Place Density And Relative Compaction	Field Test Values
CTM382	Asphalt Binder Content(1)	±0.5 percent of design mix

Note:
(1) Asphalt content may be determined by test methods CTM310 or CTM382.

3. Minimum Thickness And Grade Tolerances: The minimum AC grade thickness and grade tolerances are described below.
   - The minimum AC construction placement lift thickness shall be 1½-inch for ½-inch material and 2-inches for ¾-inch material. The average finished AC pavement thickness shall be equal to or greater than the design thickness.
   - Layer thickness shall be verified either by continuous inspection or by coring. If continuous visual inspection is used, a minimum lay-down thickness of 1.25 times the design layer thickness shall be used. If the thickness is verified by coring, then randomly selected core sample will be required as described in "Compaction Testing" below.
   - The AC finished grade surface shall be graded within a tolerance of ±0.25 inches.
4. Compaction Method And Criteria: The provisions in the current Caltrans Section 39-5.02, "Compacting Equipment", of the Standard Specifications that are in effect at the time the pavement is placed shall apply. The compaction method and criteria are summarized below.
   - After roller compacting, the finished AC surface shall be free of coarse and fine pockets (clusters) of voids. The handwork laid and compacted areas shall closely match the texture of the machine laid and compacted areas. All handwork areas shall be compacted concurrently with breakdown rolling.
   - No prime coat shall be required. Tack coat shall be applied between each lift. All vertical edges of the asphalt concrete and adjacent concrete facilities such

as gutters, cross gutters, swales, etc. shall be tack coated. If the tack coat is scraped off or contaminated, it shall be reapplied.

- The temperature of the asphalt concrete when placed and ready for compaction shall not be less than 250-degrees Fahrenheit and all breakdown compaction shall be completed before the temperature drops to 95-degrees Fahrenheit. The atmospheric temperature shall be at least 50-degrees Fahrenheit to place asphalt concrete. If asphalt concrete base is shown on the plans, the atmospheric temperature shall be at least 40-degrees Fahrenheit to begin placement. Placement of asphalt concrete materials shall not commence during fog, rain, or other unsuitable conditions as determined by the project engineer.
- The existing pavement at all cold joints shall be heated with a torch prior to placing new asphalt concrete. Joints on new pavement placed after 3 hours shall also be heated with a torch prior to placement of the adjacent pavement.
- Areas of coarse handwork or unacceptable joints shall be reheated using an infrared heater and reworked until the work complies with these requirements. Skin (thin) patching will not be allowed.
- Existing AC surfaces shall be cut to a neat, straight line parallel with the street centerline and the exposed edge shall be tack coated with emulsion prior to paving. The exposed base material shall be graded and re-compacted prior to paving.

5. Compaction Testing: Compaction testing of asphalt concrete shall be performed using both field and laboratory test methods as described below.
   - Compaction testing of asphalt concrete shall be performed consistent with CTM 375 using both a nuclear gauge and core samples. Core sample density shall be taken consistent with CTM308. If a core correlation correction factor is applied to the nuclear test method compaction test results, then core sample correlation test results shall be provided with each set of test material results.
   - Compaction of asphalt concrete shall comply with the following criteria:

Table 6.2.8.2.B, Asphalt Concrete Relative Compaction Criteria

Street Area Description	CTM 309 Percent Compaction		CTM 308 Percent Compaction	
Residential, Collector Or Arterial Roads	93.0 % Average	91.5 % Minimum	96.0 % Average	95.0 % Minimum
Shoulders, Non-Traffic Areas And Trench Patches Less Than 5-Feet-Wide	91.5 % Average	90.0 % Minimum	94.5 % Average	93.5 % Minimum

   - Asphalt concrete cores shall be collected at the rate of one test per 2,500-square feet of pavement area with a minimum of 3 core samples for any street segment or cul-de-sac. The location of each sample shall be noted on the test report. Sample location should include at a minimum the following locations: 1-foot from left lip of gutter, 1-foot from crown (either side), and 1-foot from right lip of gutter.

- One density test shall be taken for each 2,500-square feet of pavement area with a minimum of 3 tests per street segment. Each street segment may be averaged if the minimum number of tests per pavement area as shown below are met.

Table 6.2.8.2.C, Asphalt Concrete Pavement Testing Criteria

Pavement Area	Minimum Number Of Density Tests
0 to 5,000-square feet (sf)	3
>5,000-sf to 10,000-sf	5
>10,000-sf to 15,000-sf	8
Over 15,000-sf	10 or 1 per 2,500-sf (whichever is greater)

- If the average pavement compaction test results, obtained by the nuclear gauge method, fail to meet the requirements of presented in the above, then cores samples of the AC shall be taken approximately 10-feet away from the original failing test location. If the average of these three tests fail to meet the minimum compaction requirements, then the pavement area shall be cold planed (grind) to the depth of the underlying pavement course layer or aggregate base layer and replaced with new asphalt concrete.
- The core test results shall govern when compaction is being determined by both core samples and nuclear gauge tests. If the average test results obtained from the cores fails to meet the minimum average compaction requirement, because one specific area has low test results, then the asphalt concrete pavement in the area of low test results shall be removed and replaced. If no one distinct area can be identified, then the entire pavement layer shall be removed and replaced for the full width of the pavement and to the limits of the failing areas.

7 LIMITATIONS

The following limitations apply to the findings, conclusions and recommendations presented in this report:

1. Our professional services were performed consistent with the generally accepted geotechnical engineering principles and practices employed in northern California. This warranty is in lieu of all other warranties, either expressed or implied.
2. H&K provided engineering services for the site project consistent with the work scope and contract agreement presented in our proposal and agreed to by our client. The findings, conclusions and recommendations presented in this report apply to the conditions existing when H&K performed our services and are intended only for our client, purposes, locations, time frames, and project parameters described herein. H&K are not responsible for the impacts of any changes in environmental standards, practices, or regulations subsequent to completing our services. H&K do not warrant the accuracy of information supplied by others, or the use of segregated portions of this report. This report is solely for the use of our client unless noted otherwise. Any reliance on this report by a third party is at the party's sole risk.
3. If changes are made to the nature or design of the project as described in this report, then the conclusions and recommendations presented in this report should be considered invalid by all parties. The validity of the conclusions and recommendations presented in this report can only be made by our firm; therefore, H&K should be allowed to review all project changes and prepare written responses with regards to their impacts on our conclusions and recommendations. However, additional fieldwork and laboratory testing may be required for us to develop any modifications to our recommendations. The cost to review project changes and perform additional fieldwork and laboratory testing necessary to modify our recommendations is beyond the scope-of-services presented in this report. Any additional work will be performed only after receipt of an approved scope-of-work, budget and written authorization to proceed.
4. The analyses, conclusions an recommendations presented in this report are based on the site conditions as they existed at the time H&K performed the surface and subsurface field investigations. H&K have assumed that the subsurface soil and ground water conditions encountered at the location of the exploratory trenches and/or trenches are generally representative of the subsurface conditions throughout the entire project site. However, if the actual subsurface conditions encountered during construction are different than those described in this report, then H&K should be notified immediately so that we can review these differences and, if necessary, modify our recommendations.
5. The elevation or depth to the ground water table underlying the project site may differ with time and location. Therefore, the depth to the ground water table

encountered in our exploratory trenches and/or trenches is only representative of the specific time and location where it was observed.

6. The project site map shows approximate exploratory trench and/or trench locations as determined by pacing distances from identifiable site features; therefore, their locations should not be relied upon as being exact nor located with the accuracy of a California licensed land surveyor.

7. Our geotechnical investigation scope-of-services did not include an evaluation of the project site for the presence of hazardous materials. Although, H&K did not observe the presence of hazardous materials at the time of our field investigation all project personnel should be careful and take the necessary precautions should hazardous materials be encountered during construction.

8. Our geotechnical investigation scope-of-services did not include an evaluation of the project site for the presence of mold nor for the future potential development of mold at the project site. If an evaluation of the presence of mold and/or for the future potential development of mold at the site is desired, then the property owner should contact a consulting firm specializing in these types of investigations. Holdrege & Kull does not perform mold evaluation investigations.

9. Our experience and that of the civil engineering profession clearly indicates that during the construction phase of a project the risks of costly design, construction and maintenance problems can be significantly reduced by retaining the design geotechnical engineering firm to review the project plans and specifications and to provide geotechnical engineering construction quality assurance (CQA) observation and testing services. Upon your request we will prepare a CQA geotechnical engineering services proposal that will present a work scope, tentative schedule and fee estimate for you consideration and authorization. If H&K is not retained to provide geotechnical engineering CQA services during the construction phase of the project, then H&K will not be responsible for geotechnical engineering CQA services provided by others nor any aspect of the project that fails to meet your or a third party's expectations in the future.

FIGURES:

APPENDIX A:

Geotechnical Engineering Investigation Proposal for the Oroville Riverfront Road Development, November 7, 2006, PC06.030 (fee and contract agreement sections excluded).

APPENDIX B:

Important Information About Your Geotechnical Engineering Investigation Report (Presented with permission of ASFE, Copyright 2004)

APPENDIX C:

Exploratory Trench Logs

APPENDIX D:

Soil Laboratory Test Sheets

APPENDIX E:

UBCSEIS Computer Program Output Files
FRISKSP Computer Program Output Files
EQFAULT Computer Program Output Files

APPENDIX F:

SLIDE™ 5.0 Computer Program Output Files

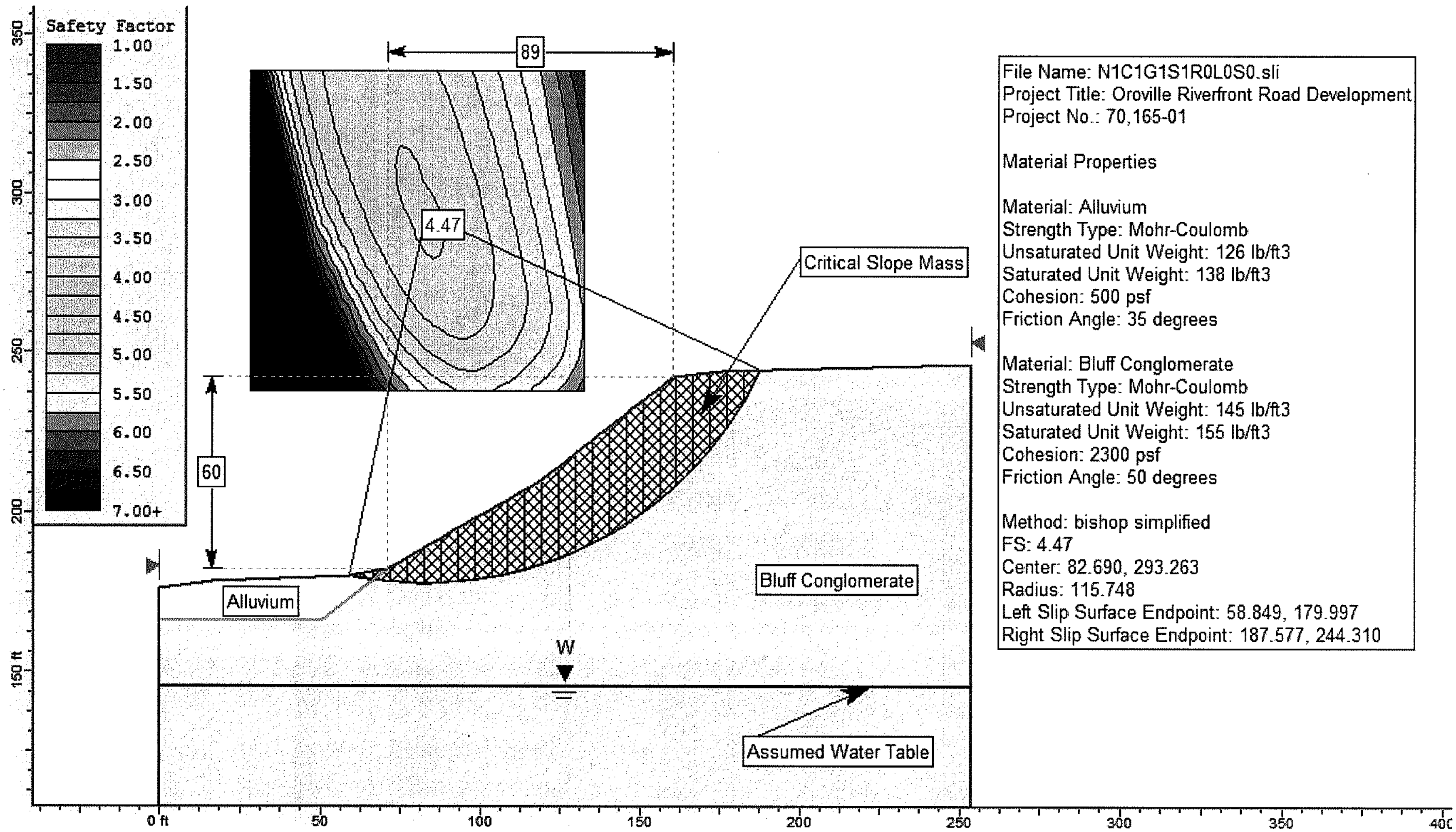


Figure F1, Section A-A' Upper Bluff Southwestern Slope Static Stability Analysis (Critical Slope Mass)

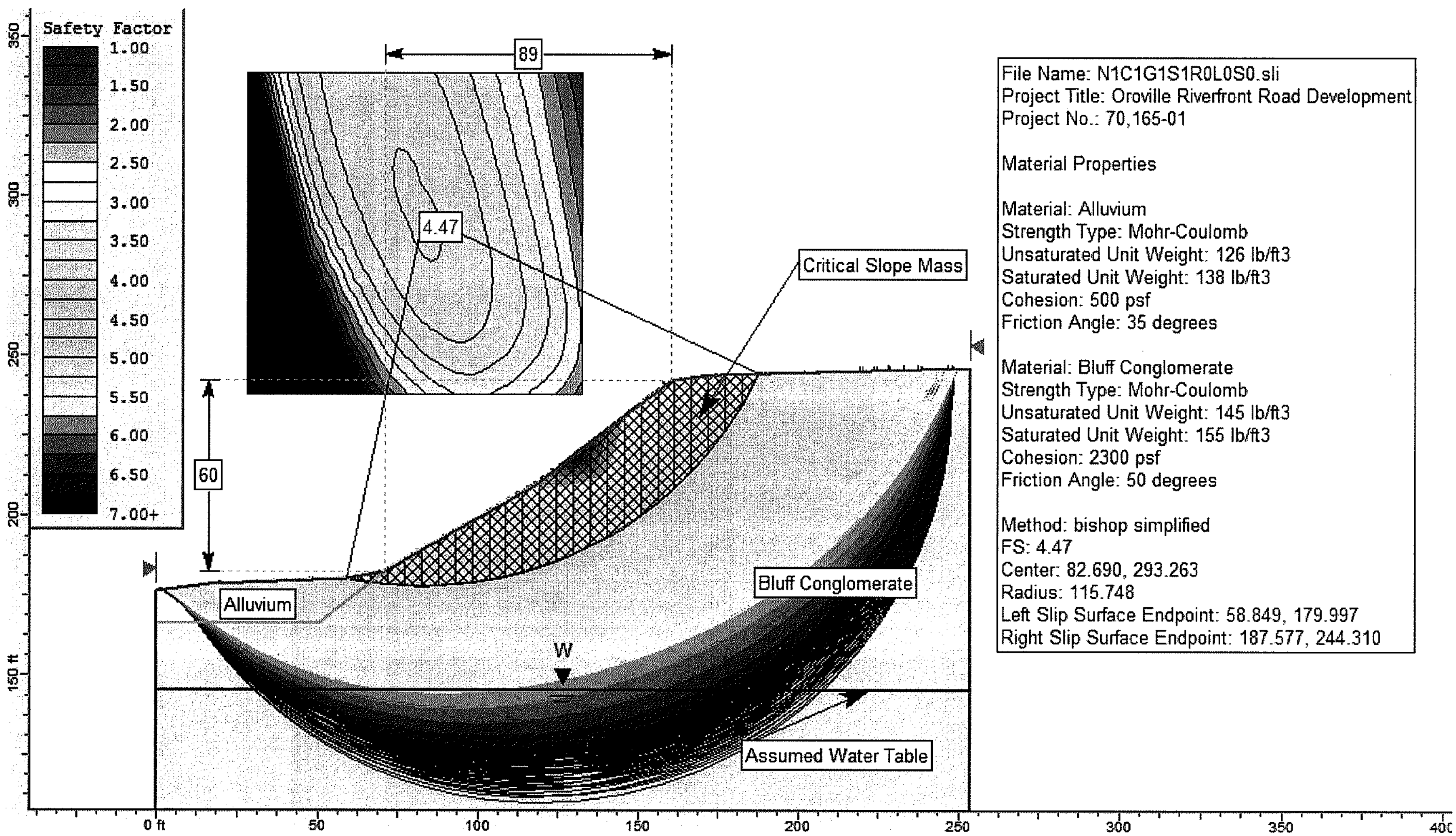


Figure F2, , Section A-A' Upper Bluff Southwestern Slope Static Stability Analysis (All Surfaces Evaluated)

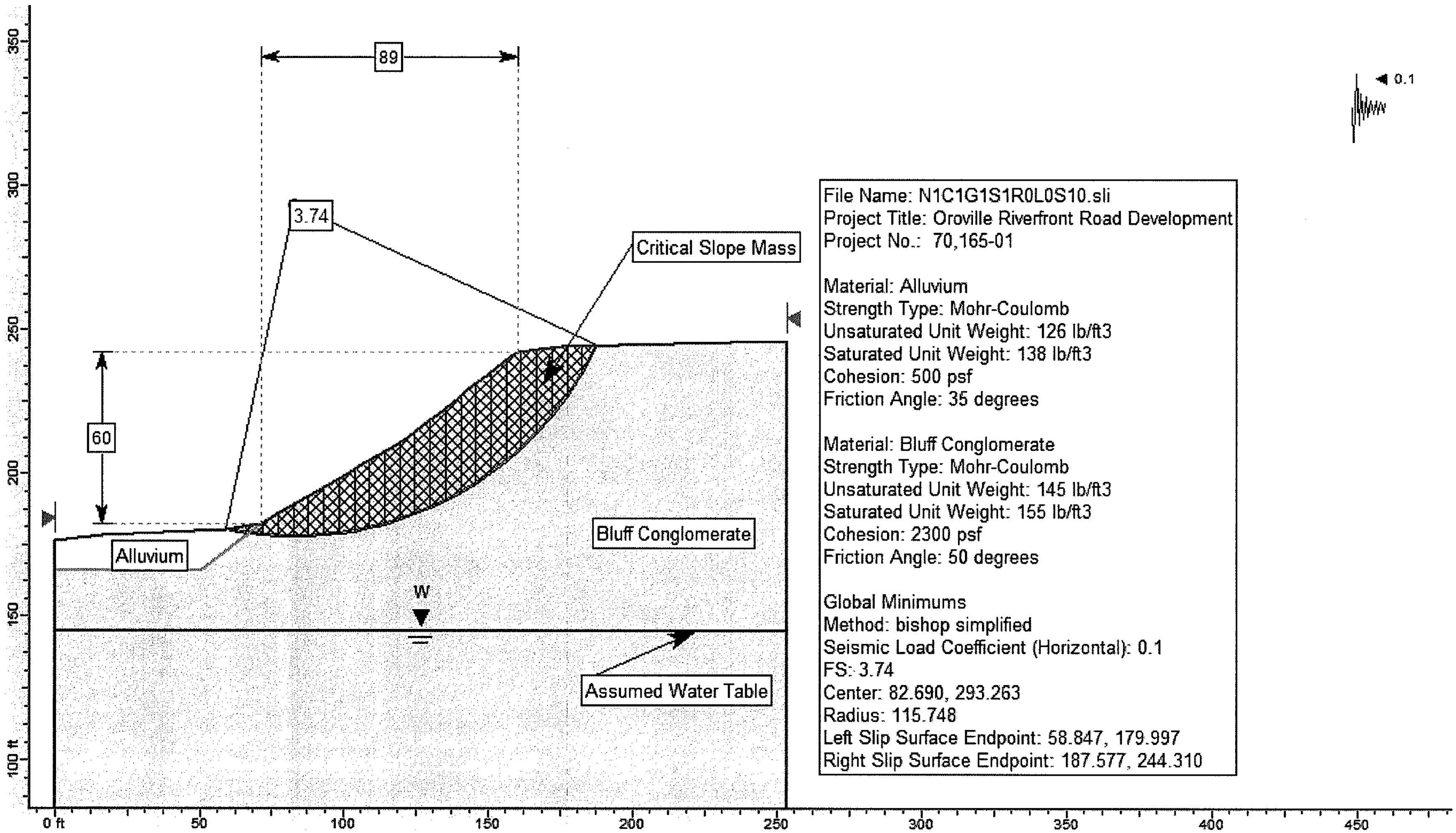


Figure F3, Section A-A' Upper Bluff Southwestern Slope Seismic Stability Analysis

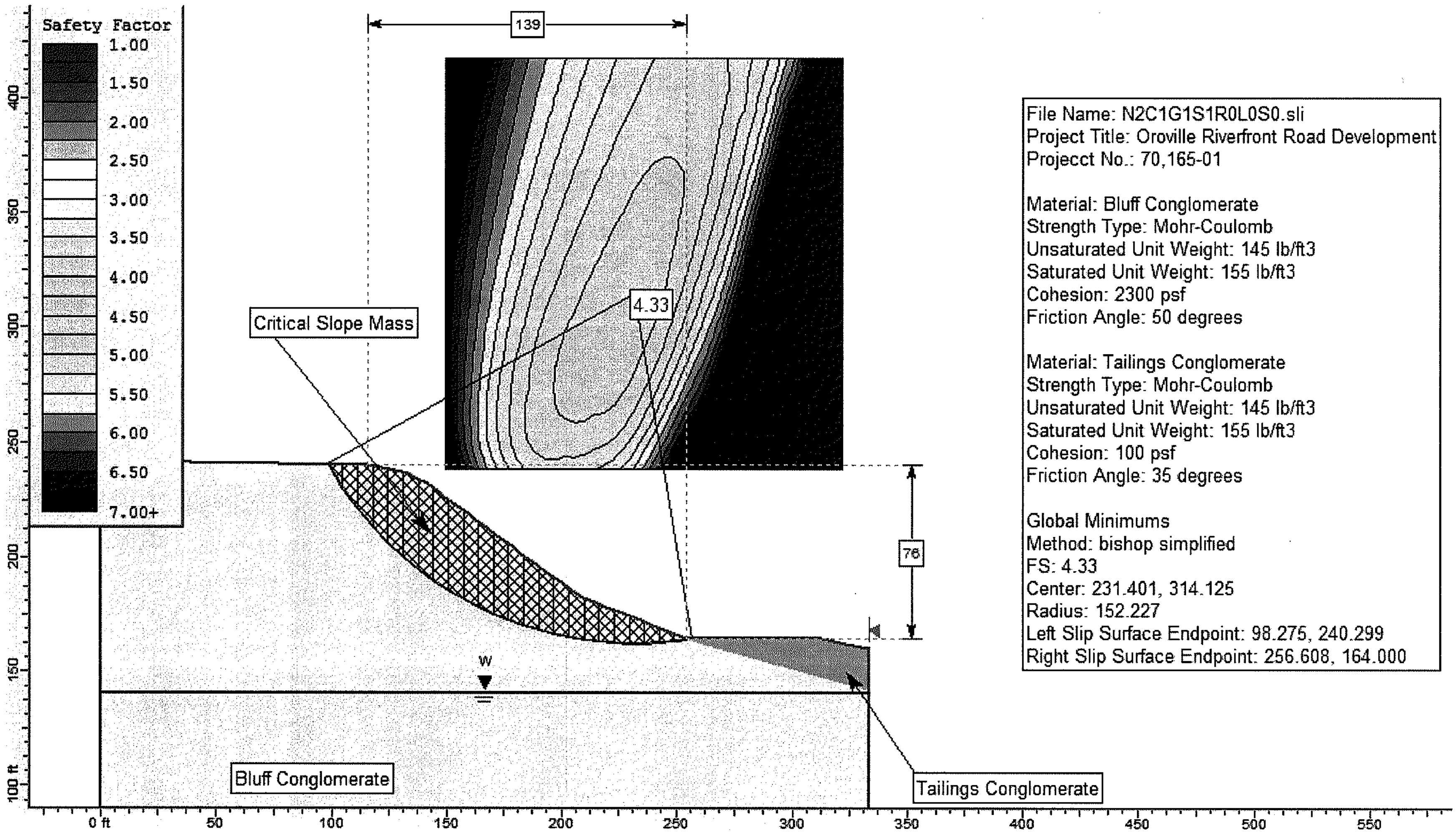


Figure F4, Section B-B' Upper Bluff Southeastern Slope Static Stability Analysis (Critical Slope Mass)

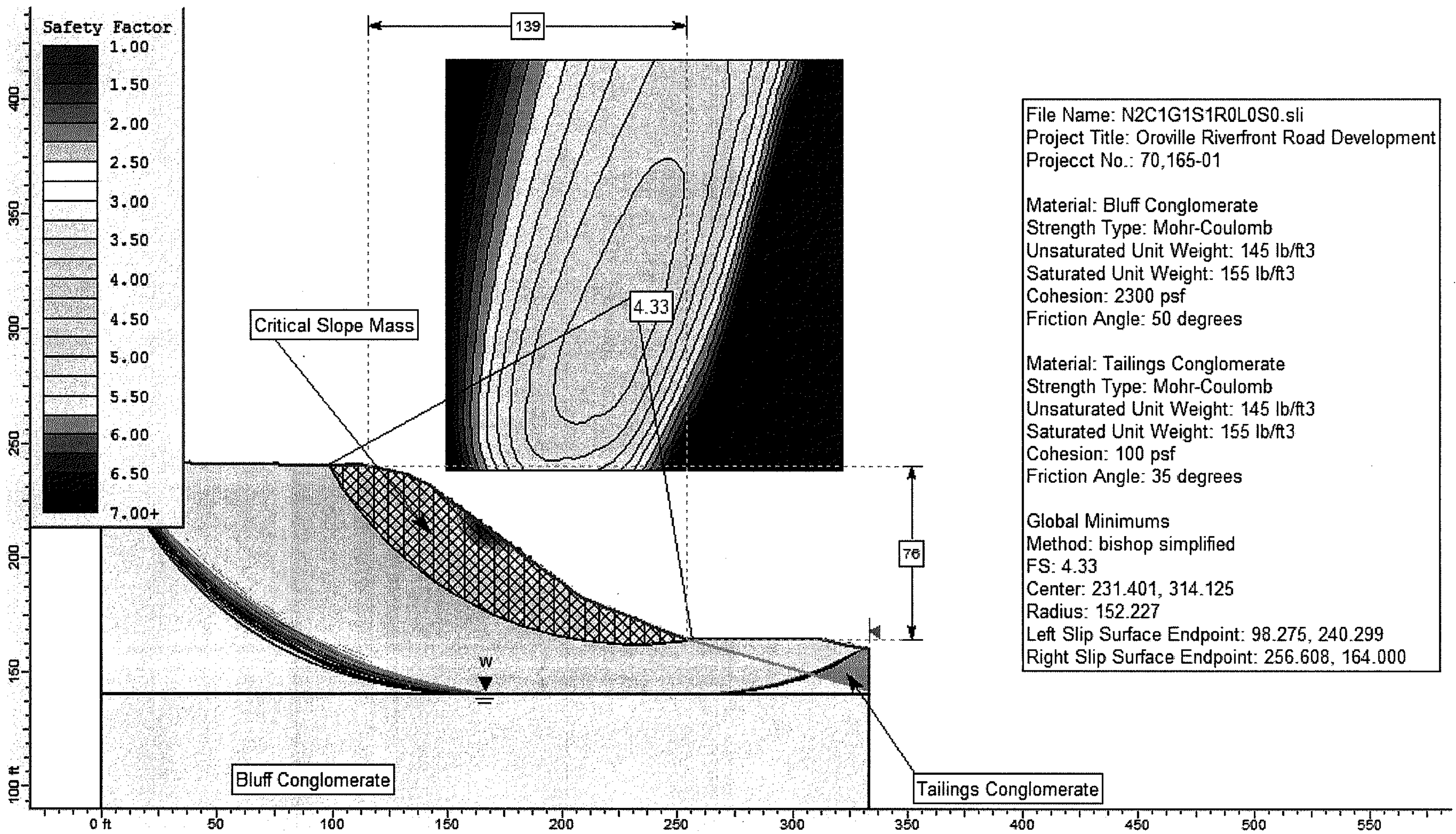


Figure F5, Section B-B′ Upper Bluff Southeastern Slope Static Stability Analysis (All Surfaces Evaluated)

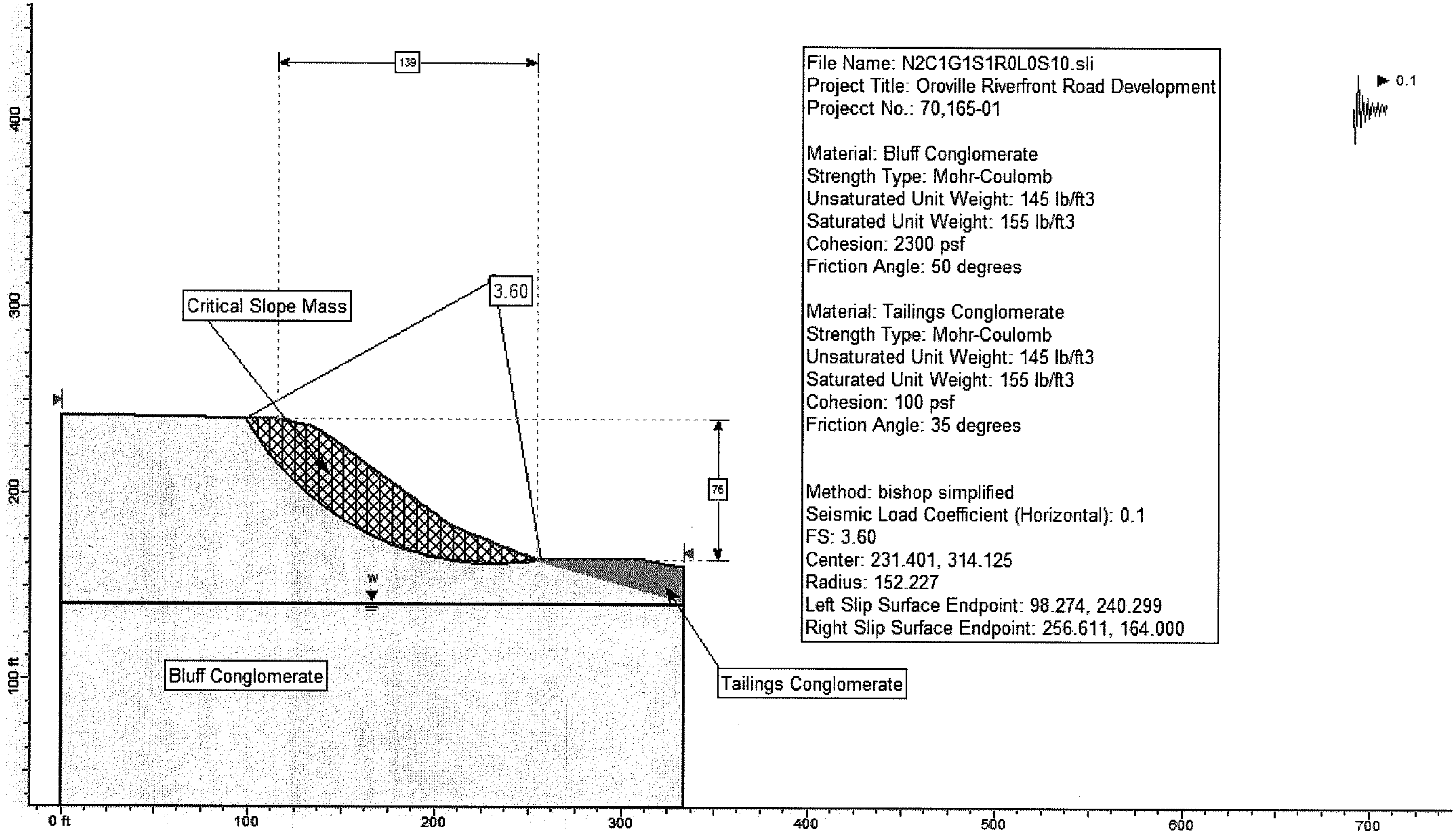


Figure F6, Section B-B' Upper Bluff Southeastern Slope Seismic Stability Analysis

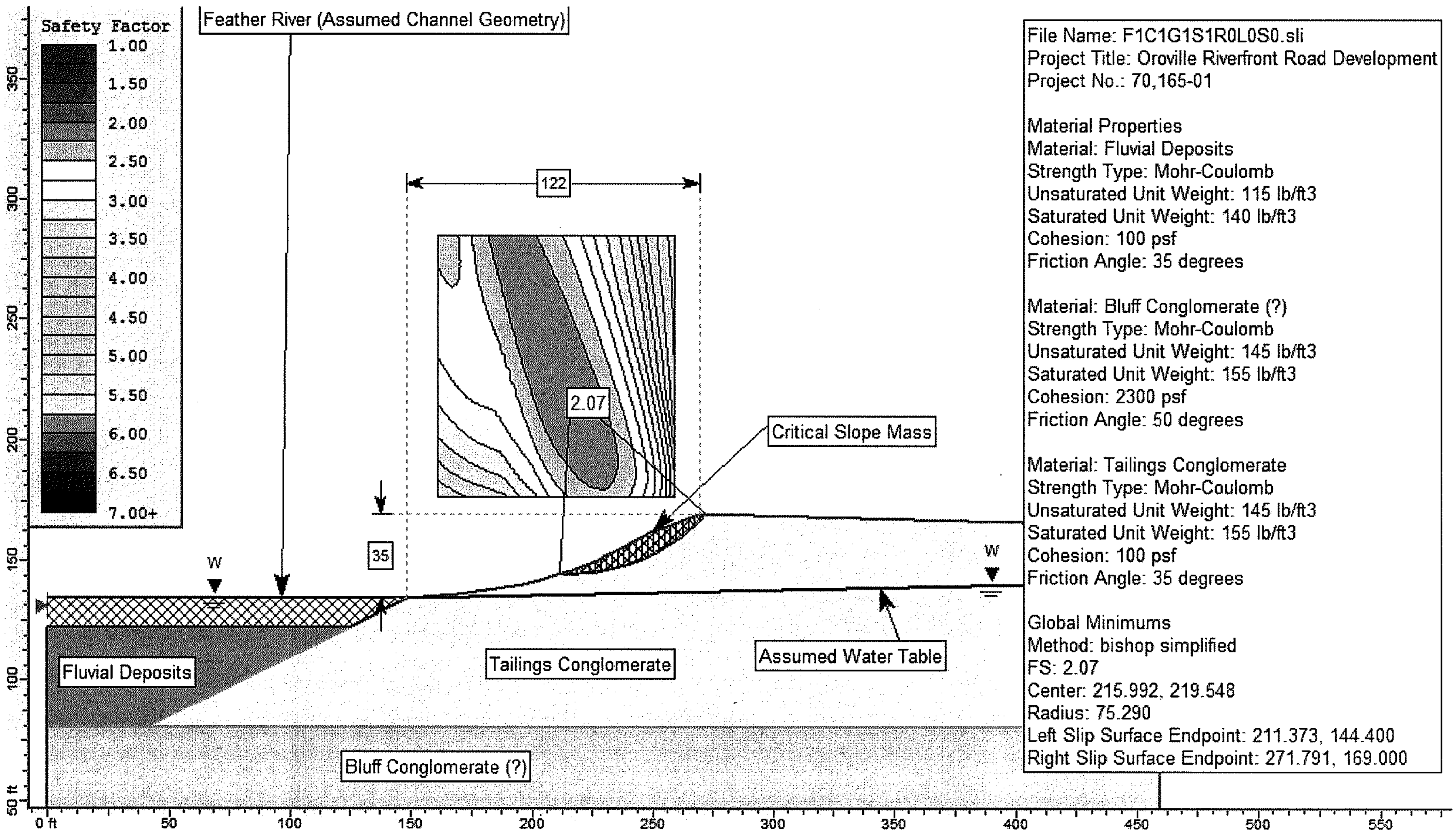


Figure F7, Section C-C' Lower Bluff Southwestern Slope Static Stability Analysis (Critical Slope Mass)

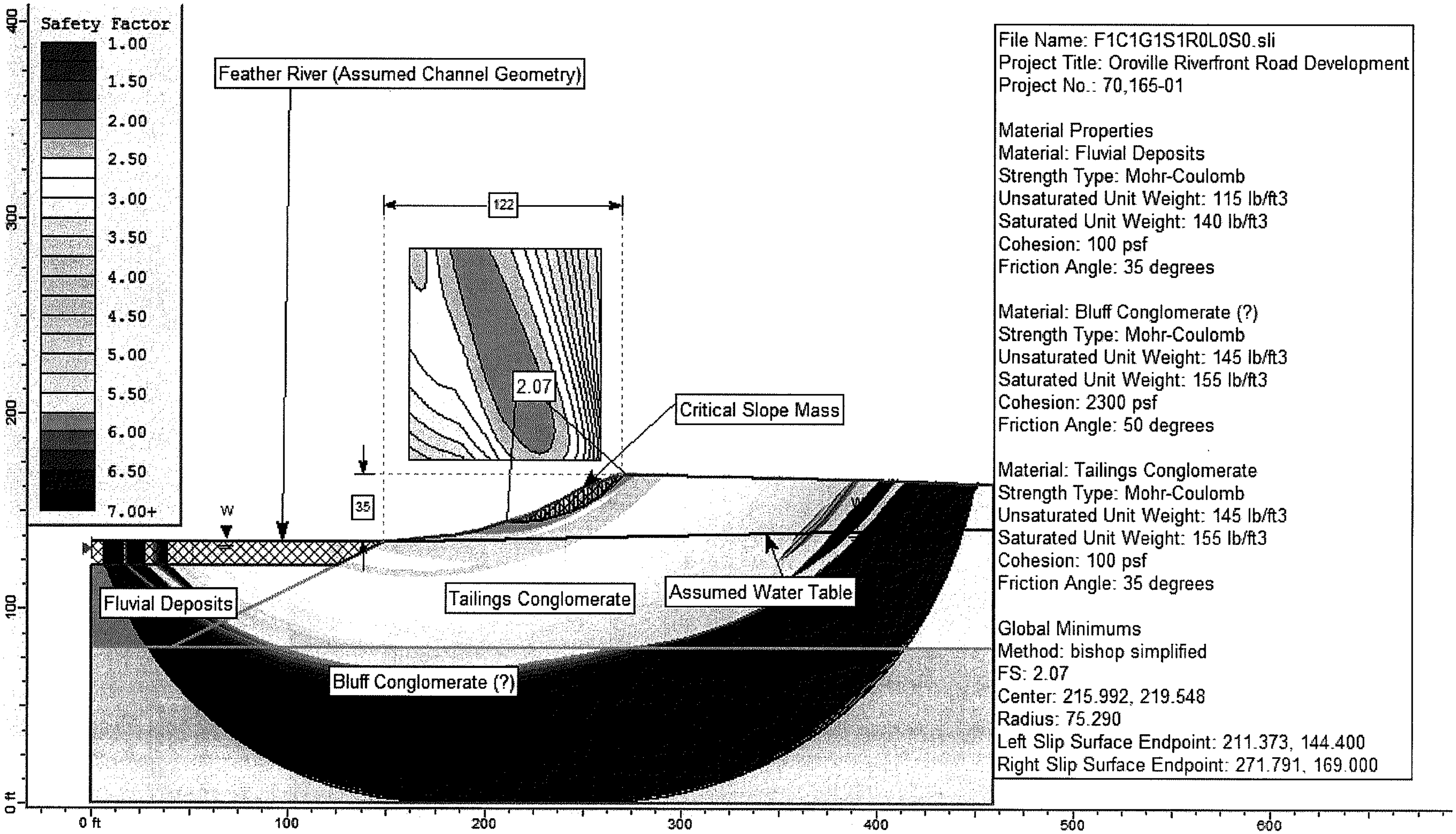


Figure F8, Section C-C' Lower Bluff Southwestern Slope Static Stability Analysis (All Surfaces Evaluated)

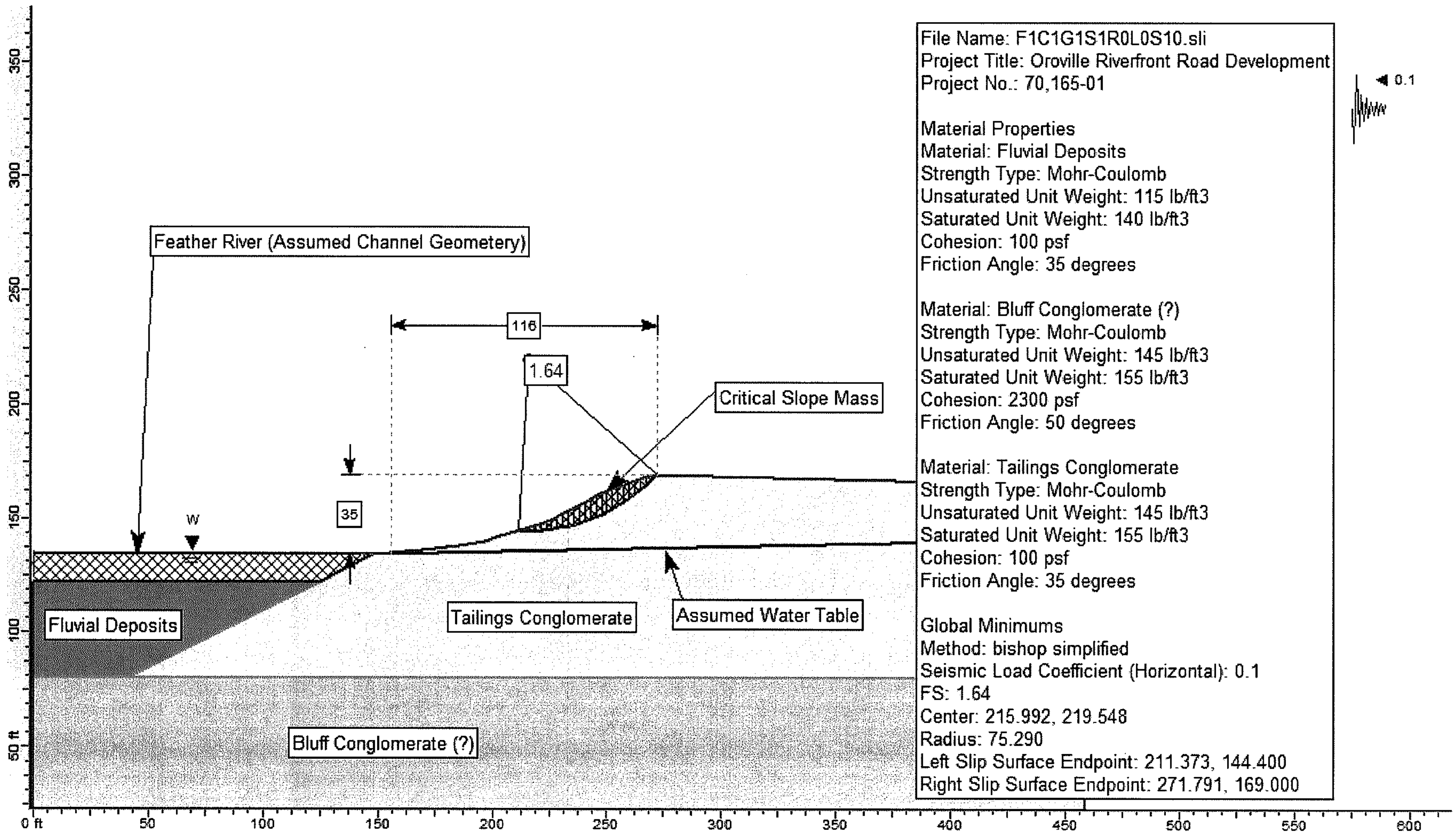


Figure F9, Section C-C' Lower Bluff Southwestern Slope Seismic Stability Analysis

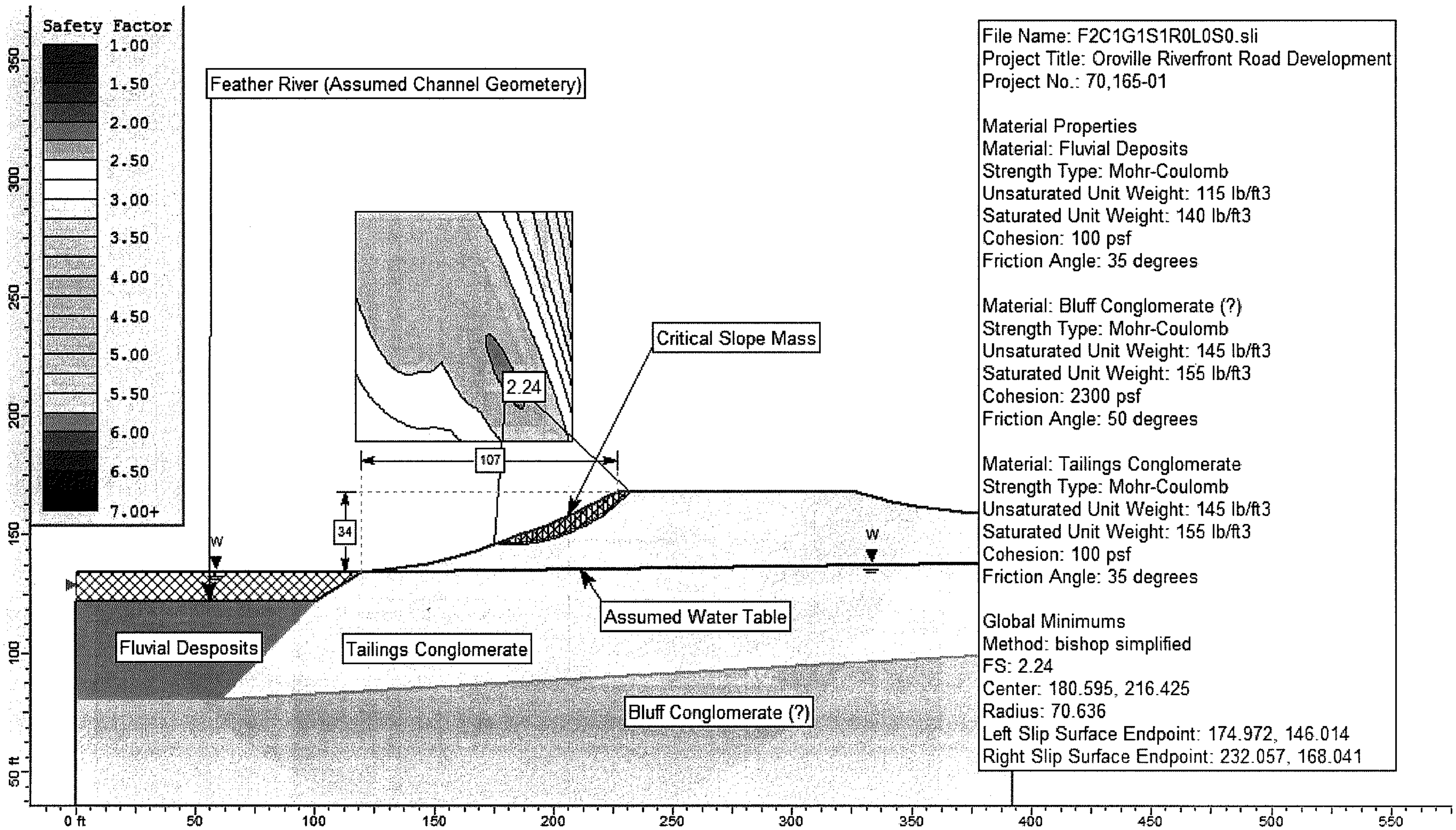


Figure F10, Section D-D' Lower Bluff Southeastern Slope Static Stability Analysis (Critical Slope Mass)