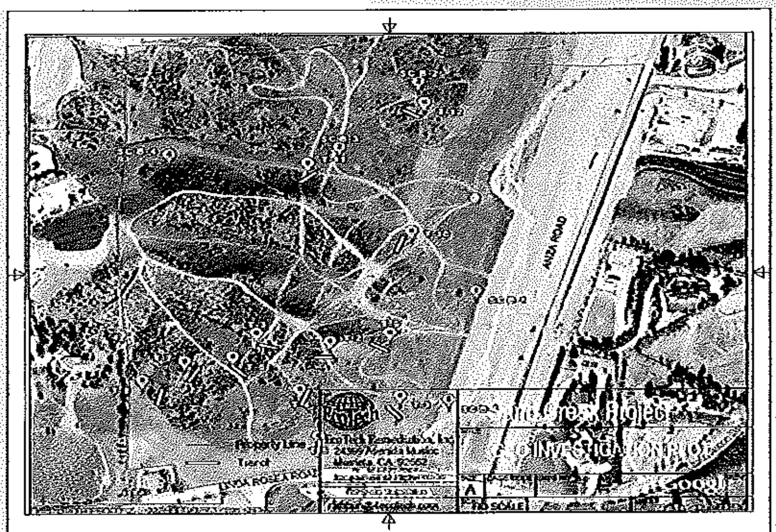
EcoTech

Prepared by:

EcoTech Remediation, Inc.
24399 Avenida Musico
Murrieta, CA 92562

Phone (951) 677-3377
Fax (951) 677-7077



Sand Creek Project

Preliminary Geotechnical Investigation

42 Acres located on Anza Road at Linda Rosea Road
Temecula, California, County of Riverside

APN 952-220-002

Prepared on behalf of:

Mr. Ken Weidrich

c/o Mr. Frank Artiga, P.E.
ACD Engineering
43430 Business Park Drive
Temecula CA

Ph. (951) 506-1425 Fax (951) 506-1426

June 17th, 2008

Mr. Ken Weidrich
c/o Frank Artiga, PE
ACD Engineering
43430 Business Park Drive
Temecula, CA 92590

SUBJECT: PRELIMINARY GEOTECHNICAL INVESTIGATION
42 Acre Property Located on Anza Road at Linda Rosea Road, Temecula, California, (County of Riverside) - APN 951-220-002,

Mr. Weidrich:

In accordance with your authorization, we have conducted a Preliminary Geotechnical Investigation for the above referenced site. The purpose of our investigation was to evaluate the geologic and soil engineering conditions of the site relative to the proposed development and provide specific recommendations for site earthwork and construction from a geotechnical standpoint.

For the purpose of our investigation, we have been provided a preliminary development plot plan by our client in the form of Tentative Tract Map 33356 dated 12/9/2006 which proposes approximately 21 homes for the site. The site topo plan was reduced to indicated-scale for our use in depicting the locations of our exploratory excavations and relevant geotechnical data.

We appreciate the opportunity to be of service on this project. Should you have any questions, or require additional information, please contact our office.

Respectfully submitted,

EcoTech Remediation, Inc.

James Evans, CEG 974
Project Engineering Geologist

Craig Schroeder, RCE 33529
Project Engineer

Peter Rathbun
General Manager

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1.0 INTRODUCTION

1.1 Site Description

The subject 42 +/- acre property consists of a nearly square piece of land composed of a single parcel of land located northwest of the intersection of Anza Road and Linda Rosea Road, in Temecula, California. The site is located within a rural to light rural and agricultural area. This project, being low density rural will not change the existing area ambiance. It is bounded on the north by several rural ranchettes, and a street, on the east by a natural drainage channel draining southward, which in turn is bounded by Anza Road, and on the south by Linda Rosea Road. The geographical relationships of the site and surrounding area are depicted on our Site Location Map, Figure 1. The natural drainage has been named Sand Creek. Our observation indicate that the intended purpose of the site is the same or very similar to its surrounds.

The site is currently undeveloped and aerial photographic review reveals that no previous development has occurred on site. The site topography and other features are depicted on our Geotechnical Map, Figure 2.

Vegetation across most of the site consists of a sparse growth of annual weeds and grasses that are routinely plowed for weed abatement.

1.2 Proposed Development

The proposed development includes construction of an excess road roughly parallel to Anza Road and west of the existing drainage channel. With cul-de-sac roads servicing large lot pads. Moderate to deep cut cuts and cut-fill lots are proposed to create the large lot pads. No brow ditches or terrace drains are proposed for the slopes.

1.3 Scope of Work

The following scope of work was performed in substantial conformance with the generally accepted standard of care for similar projects in this location:

- Review of selected geologic and soil reports and/or maps in our files and at the, County of Riverside, County of Riverside Flood Control, and listed references (see Appendix).
- Examination and geologic interpretation of stereoscopic aerial photographs showing the site and immediate vicinity.
- Field reconnaissance and geologic mapping of the surficial earth materials.
- Excavation, logging, and soil sampling of 12 exploratory trenches and 5 soil borings
- Laboratory testing of representative soil samples to determine relevant soil

engineering properties.

- Engineering analysis to determine engineering parameters including lateral earth pressures, allowable soil bearing capacity, classification, and other soil engineering factors.
- Discussion of related secondary seismic hazards.
- Preparation of this report presenting our preliminary findings, conclusions, and recommendations for future site planning, design, and development.

2.0 SITE INVESTIGATION

2.1 Background Research and Literature Review

Several published and unpublished reports and geologic maps were reviewed for the purpose of preparing this report. Documents were reviewed where available at the County of Riverside, and County of Riverside Flood Control regarding similar developments on adjacent parcels as well as regional geologic studies and aerial photographs. A complete list of the publications, reports, and aerial photographs reviewed for this investigation is presented in **Appendix A**.

2.2 Field Investigation

Our field investigation included geologic field mapping to delineate the surficial distribution of earth materials. Subsurface exploration, including a total of 12 exploratory trenches and 5 soil borings, was also conducted to verify our geologic mapping, examine the exposed lithology, and collect representative soil samples for laboratory testing. Our trenches were excavated to a maximum depth of 15 feet below the ground surface utilizing a CAT 230 Tracked Excavator equipped with a 30 inch wide bucket. The soil borings were drilled to a maximum depth of 26.5 and 52 feet below ground surface (B-1) using a B-61 drill rig equipped with hollow stem augers.

Our field personnel supervised the subsurface exploration and prepared field logs describing the exposed lithology and related conditions. Representative bulk and in-place soil samples were obtained from the exploratory trenches and soil samples were collected from the soil borings for laboratory testing. Copies of our exploratory trench logs and soil boring logs are presented in **Appendix B**.

2.3 Laboratory Testing Program

Bulk and in-place soil samples representing the lithologic sequences encountered during our subsurface exploration were obtained for laboratory testing which included maximum density/optimum moisture determinations, expansion index, sieve analysis, direct shear strength, and soluble sulfate content.

Laboratory testing was conducted in accordance with ASTM, Caltrans, and Uniform Building Code (UBC) test specifications, where applicable. The results of our laboratory tests are presented in **Appendix C** of this report.

3.0 Geologic Setting

3.1 Regional Geology

The subject site is situated within a natural geomorphic province in southern California known as the Peninsular Ranges, which is bordered to the east by the Salton Trough, the north by the Transverse Ranges (San Bernardino, San Gabriel, and Santa Monica Mountains). The Peninsular Range province extends southerly to the Baja peninsula and westerly to the Pacific Ocean.

The site lies in a basin filled with Fluvial sand deposits of Pleistocene age. The basin is surrounded by mountains underlain by crystalline basement rock from which the sediments in the basin have been derived. The southwestern limit of the basin is the Elsinore Fault Zone which lies at the base of the Santa Ana Mountains to the southwest. The Pauba Formation comprises the Pleistocene sedimentary unit in the area has been gently folded or is unfolded depending on the proximity to tectonically affected areas.

This province is generally characterized by structurally controlled, elongated northwesterly-trending valleys and mountains, with elevated erosional surfaces. The eastern portion of the province has been extensively uplifted by faulting and represents the highest and most rugged terrain. From the east, the province gradually descends to the west toward the Pacific Ocean.

The Peninsular Ranges are traversed by numerous northwest trending faults creating and subdividing the province into many sub-parallel, northwest trending ranges and valleys. The northwesterly trending mountains and valleys are flanked by regional faults, which remain active today, including the San Andreas, San Jacinto, and Elsinore Fault zones.

The site lies in a basin filled with Fluvial sand deposits of Pleistocene age. The basin is surrounded by mountains underlain by crystalline basement rock from which the sediments in the basin have been derived. The southwestern limit of the basin is the Elsinore Fault Zone which lies at the base of the Santa Ana mountains to the southwest. The Pauba Formation which comprises the Pleistocene sedimentary unit in the area has been gently folded or is unfolded depending on the proximity to tectonically affected areas.

3.2 Local Geology

The site lies in the Temecula Valley, which is an alluvium-filled, very broad valley draining from mountains to the east, which are composed of crystalline basement rocks and with hills projecting out of the valley also underlain by crystalline basement material. The valley drains to the west at a very low gradient into Temecula Creek and thence towards the Pacific Ocean becoming part of larger drainage features en-route. The site is between the Elsinore Fault Zone and the San Jacinto Fault Zones in an area with significant previously mapped fault systems. More specifically the site is approximately 4.2 miles north, northeast of the closest major fault zone (the Elsinore Fault Zone) and 2.1 miles north, northeast of the closest county mapped fault.

3.3 Site Geology

The site is underlain by the Pauba Formation which is exposed only on the ridges on the site. The flanks of the ridges consist of either debris flow landslide deposits or colluvium derived from the ridges. Alluvial sands have been deposited in the channel of Sand Creek along the easterly side of the site.

The Pauba Formation appears to have been very gently folded with the southwesterly area of the site dipping slightly easterly. The formation is composed of dense sands with some very fine sandy silt layers. The debris flow landslides appear to have been generated by saturation above the silt layers which in turn resulted in debris flows since the saturated sands lost cohesion due to the saturation. Test Pits 2 & 3 encountered one of these siltstone beds which was estimated to have a dip to the southeast. The attitude of the stratum was estimated based on a projection assuming a 10 degree dip since the caving of the sidewalls of the test pit prevented direct measurement. This same projections was used to locate the stratum on the geotechnical sections. The projected bed is believed to account for slight changes in topography.

3.3.1 Soil Conditions

The site soils are composed entirely of sand with generally variable but small amounts of silt and clay. A few beds of silty fine sand were observed. The Pauba Formation sands were found to be in dense condition. All other sand units had generally medium dense blow counts, and when tested were 80% minimum relative compaction with the exceptions of the Sand Creek steam bed area, more specifically, lots # 21, 22, "D" Street, Linda Rosea Road, and Anza Road. This Area had loose sands as deep as 25 feet. Expansion testing revealed low expansion properties and minimal sulfate content.

4.0 SUBSURFACE CONDITIONS

4.1 Alluvium (Qal) & Colluvium (Qc

Alluvial soil deposits are found in the San Creek drainage channel. Colluviums was found in the coalescing fans from the side channels draining from the hills.

4.2 Debris Flow (Qls)

Small to large areas underlain by debris flow originating from the ridge slopes were identified. They consist of sand deposits mixed with former sandy topsoil materials derived from the Pauba formation.

4.3 Pauba Formation (Qp)

Consists of fine to coarse sand deposits with some fine sand and very silty fine sand strata. All ridges on the site are underlain by these materials and the entire site is underlain at depth by this formation. Trench – 6, on the ridge revealed these materials and it is believed this is true across the site.

4.4 Groundwater

Groundwater or seepage was not encountered in any of our exploratory trenches to a maximum depth of 15 feet below the ground surface. Groundwater was not observed in the soil borings at approximately 50 to 55 feet below ground surface.

In summary, groundwater elevations are expected to be approximately 65 to 75 feet below the ground surface throughout the lower elevations of the property and are not expected to adversely impact the proposed earthwork or development. Groundwater elevations can fluctuate with the influence of the drainage course on-site and the presence of perched shallow water can not be precluded.

4.4 Excavation Characteristics

Excavation of all materials on site is anticipated to be excavated with moderate to relative ease of excavations utilizing conventional grading equipment in proper working condition.

We do not expect on-site excavations to be impacted by shallow groundwater considering the findings of our subsurface exploration. Seasonal fluctuation in the groundwater elevations, however, may occur and shallower perched groundwater conditions may be encountered during the winter or spring months.

5.0 FAULTING AND SEISMICITY

5.1 Faulting

As was stated above, the site is located between the Elsinore and San Jacinto Fault Zones in the seismically active Southern California Region. Other faults occur with 100 km of the site but none can have the site effects that these can. The site is located 4.2 miles from the near source zone of the class B Elsinore Fault and 17 miles from the near source zone of the class A portion of the San Jacinto Fault.

It is our opinion that the 2007 CBC (California Building Code) Seismic Soil Site Class is "D" and the site is in Seismic Zone 4.

Latitude 33.5069 N Long 117.0531W

Which via the USGS Seismic Research/Design values yields the following results:

Conterminous 48 States
2006 NEHRP Seismic Design Provisions
Latitude = 33.5
Longitude = -117.0
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.01 deg grid spacing

Period (sec)	Sa (g)
0.2	1.500 (Ss, Site Class B)
1.0	0.600 (S1, Site Class B)

Conterminous 48 States
2006 NEHRP Seismic Design Provisions
Latitude = 33.5
Longitude = -117.0
Spectral Response Accelerations SMs and SM1
SMs = FaSs and SM1 = FvS1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)
0.2	1.500 (SMs, Site Class D)
1.0	0.900 (SM1, Site Class D)

Conterminous 48 States
2006 NEHRP Seismic Design Provisions
Latitude = 33.5
Longitude = -117.0
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)
0.2	1.000 (SDs, Site Class D)
1.0	0.600 (SD1, Site Class D)

Based on the distance to the above faults, the site could experience an earthquake with vibrations with a maximum horizontal acceleration of between 0.25g and 0.55g and with durations of strong shaking exceeding 20 seconds.

5.2 Probable Seismicity

The site is located in a region of generally high seismicity, as is all of southern California. During its design life, the site is expected to experience strong ground motions from earthquakes on regional and/or local causative faults. Accordingly, the primary geologic hazard that exists at the site is that of ground shaking. The strength of earthquake-induced ground shaking is commonly measured as maximum or peak ground acceleration. Acceleration is defined as the time rate of change of velocity of a referenced point during an earthquake, commonly expressed in percentage of gravity (g). Its value at a particular site is a function of many factors, including, but not limited to, earthquake magnitude, distance to causative earthquake, various seismic-source parameters, site location relative to direction of energy propagation, and geologic conditions at the site.

Considering the location of the property relative to the Elsinore and San Jacinto Fault zones, the site is likely to experience strong ground shaking during the design life of the proposed development.

Based on a state wide probabilistic seismic hazard assessment (Petersen et. al., 1996) there is a 10 percent chance in 50 years that the site will experience peak horizontal ground accelerations greater than 70 percent of gravity or 0.70g.

6.0 SECONDARY HAZARDS

6.1 Liquefaction

Liquefaction is the loss of soil strength due to increased pore water pressures caused by significant ground shaking (seismic event) of saturated fine grained, cohesionless soils. Liquefaction typically consists of the re-arrangement of the soil particles into a denser condition resulting in localized areas of settlement, sand boils, flow failures, and loss of bearing capacity. Areas underlain by loose cohesionless soils, where groundwater is within 30 to 40 feet of the ground surface, are particularly susceptible to liquefaction when subject to significant ground shaking such as those due to earthquakes or seismic activity. The liquefaction potential is generally considered greatest in saturated, loose, poorly graded fine sands with a mean grain size (D_{50}) in the range of 0.075 to 0.2mm.

Liquefaction in the alluvium of the Sand Creek bed near B-1 and B-5 is negligible under current conditions, however, if we have shallow groundwater the hazard becomes substantial. If water is 20 feet deep with a 0.7 g earthquake (10% probability in 50 years) the total settlement due to liquefaction could be 6 to 8 inches. This could cause settlement and maybe lateral spreading in the valley areas and affecting the building pads as well. Please refer to the two "A-1" plates for Liquefaction modeling presented in Appendix "D" under "Calculations".

6.2 Ground Rupture

Ground rupture during a seismic event normally occurs along pre-existing faults. As previously discussed, the property does not lie within a California Earthquake Fault Zone.

In summary, based on the location of the property relative to the demonstrably active Elsinore and San Jacinto Faults, there is low potential for ground rupture from tectonic sources to impact the property.

6.3 Seismically Induced Soil Settlement

In its current condition, the on-site, near surface alluvial soil is considered susceptible to seismically induced settlement caused by intense ground shaking during an earthquake. This could occur on the order of 2.58 to 3.8 inches in dry sand as encountered close to and within the drainage basin, and as high as 5.9 to 8.11 inches if the seismic event should occur during the wet season and ground water is at 20 ft. bgs.

6.4 Landsliding

The site is relatively hilly with a maximum overall elevation change of approximately 200 feet. Accordingly, and due to sandy soil conditions there is a moderate to significant landslide hazard or slope instability without further mitigation. Boring 2 , trench 12 and section D-D' above Lot 18 does not show Debris flow or out of slope siltstone bedding.

Based on slope stability analysis any slopes with old debris flow and/or out of slope siltstone bedding planes will need mitigation. This includes the slopes above Lot 5 (section A-A'), Lot 6, Lot 8, Lot 9 (section B-B'), Lot 14 (section C-C'), and Lot 15.

6.5 Rockfall Potential

The site is relatively hilly with a maximum overall elevation change of approximately 200 feet. No rocks and/or boulders were observed, there is no potential for rockfall hazard to impact the property.

6.6 Erosion Potential

The soils encountered are nearly uniformly subject to severe erosion during heavy rainfall whether in excavated slopes or fill slopes.

7.0 CONCLUSIONS

- Development of the subject property as proposed is considered feasible from a geotechnical standpoint, provided the specific recommendations of this report are addressed during future site planning, design, and construction.

- Alluvial materials: The upper 5 to 20 feet in the valley areas appear to be in a loose state which may consolidate. The areas away from the valley appear to be a medium dense material. The Sand Creek Channel area will need substantially more removal and should have a geofabric such as Myrafi 600X (or equivalent) placed in the bottoms of the removals and maybe at intervals within the re-compacted fill. This is to reduce the differential settlement.

- Loose top soils and fill soils derived from the sandy soils are Surficially stable when saturated to a depth of 3 feet, However they are Surficially unstable when saturated to a depth of 4 feet. These slopes will require some mitigation. The cut slopes of dense sand and Pauba Formation are Surficially stable when saturated to a depth of 4 feet.

- The site is relatively hilly with a maximum overall elevation change of approximately 200 feet. Accordingly, and due to sandy soil conditions there is a moderate to significant landslide hazard or slope instability without further mitigation. Based on slope stability analysis any slopes with old debris flow and/or out of slope siltstone bedding planes will need mitigation. This includes the slopes above Lot 5 (section A-A'), Lot 6, Lot 8, 9 (section B-B'), Lot 14 (section C-C'), and Lot 15.

- Groundwater was not encountered within the 55 ft. subgrade of our boring investigations. Subsequent questions of the Water Master at Eastern Municipal Water supplied us with the information that groundwater varied greatly within the drainage channel on the southeast side of the property parallel to Anza Road depending on the seasons of the year. Under normal circumstances groundwater would be encountered at 65-75 ft. During the rainy season, that level could shallow significantly after the wetter portions of the year.

- Based on our limited subsurface exploration, all materials exposed on the subject site are expected to excavate with moderate ease utilizing conventional grading equipment in proper working condition.

- The site is located within a very active earth quake area. However, the site is not within a fault zone as defined by the "Alquist-Priolo Earthquake Faults Zone Act of 1972" or the County Earthquake Zones as defined in Riverside County GIS. The structures will most likely be subjected to severe ground shaking during their lifetime. It is generally recommended that the structures be designed to at least meet the current seismic building code provisions in the latest CBC edition. However, it should be noted that the building code is described as a minimum design condition and is often the maximum level to which buildings are designed (Holden & Real, 1990). The majority of property owners are not aware that structures, built to code, are designed to remain standing after an earthquake, in order for occupants to safely evacuate, but then may have to ultimately be demolished (Larson & Slosson, 1992).

- Liquefaction in the alluvium of the Sand Creek bed near B-1 and B-5 is negligible under current conditions, however, if we have shallow groundwater the hazard becomes substantial. If water is 20 feet deep with a 0.7 g earthquake (10% probability in 50 years) the total settlement due to liquefaction could be 6 to 8 inches. This could cause settlement and maybe lateral spreading in the valley areas and affecting the building pads as well.

- Most site materials encountered are subject high probability of erosion from runoff or heavy irrigation. Mitigation measures will be necessary to control the degree of erosion on any and all slopes.

8.0 RECOMMENDATIONS

8.1 General Earthwork and Grading

Preliminary recommendations for site grading and foundation design are presented in the following sections of this report. The recommendations presented herein are preliminary based on our understanding of the proposed development and should be reviewed when more specific site planning and design details are available.

Prior to the commencement of grading, the site should be cleared of any trash, existing structures, foundations, basements, septic systems, construction debris, trees/stumps and all other vegetation. This material should be hauled off-site in an approved manner. The holes resulting from the removal of trees, septic tanks, or any other and subterranean facilities, should be backfilled and compacted as approved by the project Geotechnical Consultant.

The client, prior to any site preparation or grading, should arrange and attend a meeting among the grading contractor, the design civil engineer/surveyor, the soils engineer and/or geologist, and a representative of the appropriate governing authorities as well as any other concerned parties.

Earthwork should be conducted in accordance with the provisions of the California Building Code and the Standard Earthwork and Grading Specifications provided in **Appendix D**, except where specified in this report.

8.2 Alluvial Removal, old fill removal and Preparation of Existing Ground

The alluvial soil can be used as compacted fill provided it is free of deleterious material. The soil should be conditioned to near, to slightly over optimum moisture content prior to use as fill. Large variations in moisture conditioning are not acceptable.

Depths of alluvial/colluvial removals are anticipated to vary from 3 to 20 feet below the ground surface. The alluvium/colluvium removal in the areas not associated with the Sand Creek channel is expected to be between 3 and 5 feet.. The Sand Creek Channel area will need substantially more removal and should have a geofabric such as Myrafi 600X (or equivalent) placed in the bottoms of the removals and maybe at intervals within the re-compacted fill. This is to reduce the differential settlement. These estimated depths of removal are based on our limited subsurface exploration and may vary depending on the depth of alluvium across the site, height of proposed embankment, and building loads. The project soils engineer and/or engineering geologist should verify the depth of removals in the field considering the actual soil and geologic conditions exposed.

All landslide debris flow materials should be removed and replaced as compacted fill.

Prior to the placement of fill, the exposed earth materials should be scarified a minimum of 12-inches below the ground surface, moisture conditioned to near optimum moisture, and re-compacted to a minimum of 90 percent of the maximum dry density (as determined by ASTM D-1557).

8.3 Fill Placement

On-site earth materials are expected to be suitable for use as structural fill provided they are properly moisture conditioned and free of deleterious material including rocks greater than 6 inches in diameter. Import soil, if required, would require additional testing to determine their feasibility for use as structural fill.

Approved fill material should be placed in 6 to 8-inch lifts, brought to optimum moisture content, and compacted to a minimum of 90 percent of the maximum laboratory dry density, as determined by the ASTM D 1557 test method. No rocks larger than 6 inches in diameter should be used as fill material unless specifically approved by the project Geotechnical Consultant.

8.4 Cut/Fill Transitions

Cut-to-fill transitions should be eliminated from building pads where the depth of fill exceeds 12-inches. This should be accomplished by over excavating the cut portion and replacing the materials as properly compact fill. Limits of over excavating the cut should extend a minimum of five feet beyond the footing elements in plan view. Recommended depths of over excavation are dependent on the depth of embankment fill and can be provided following a review of the final grading plan. The following table can be used as a guideline for planning purposes.

Depth of Fill on "Fill" Portion	Depth of Over excavation "Cut" Portion below the bottom of the footings
0 to 6-ft	2.0-ft
> 6-ft	1/3 Depth of Fill to Maximum Depth of 15-ft

8.5 Expansion Potential

Laboratory testing of soil material representative of the on-site soil indicates an expansion index of (EI = 1 to 5) which corresponds to a very low potential for soil expansion. Following grading and mixing of the soil as recommended in this report, we expect the expansion potential for soil material exposed at grade to be very low and the following recommendations should reduce the potential for damage caused by these soils. NOTE: This must be verified at the end of grading by confirmatory expansion tests.

8.6 Sulfate Content

Laboratory tests of representative on-site soil indicate a soluble sulfate content of less than 20 ppm = <0.002% Sulfates. Considering the preliminary test results, it is anticipated that conventional Type II Portland Cement can be used for construction of footings and other foundation elements to be in contact with the ground.

Sulfate content testing should be conducted within the building pads at the completion of grading and on imported soils prior to their approval as structural fill material.

8.7 Earthwork Factors

The following shrinkage/bulkage factors should be considered for on-site earth materials excavated and compacted during site construction.

Alluvial Soils and debris flow materials	8-15% shrinkage
Pauba Formation materials	0 to 8% shrinkage
	* this is based on relative compaction

The above shrinkage values are estimated considering an average relative compaction at the completion of grading of 92 percent for the on-site soils. An increase in relative compaction, or deeper removals, could correspond to an increase in shrinkage values. Subsidence, as a result of ground preparation, may also be anticipated on the order of 0.10 feet, occurring mostly during site construction.

8.8 Lateral Load Resistance

The following parameters should be considered for lateral loads against permanent structures founded on materials compacted to 90 percent of the maximum dry density. Soil engineering parameters for imported soil may vary.

The footings of the retaining walls should be placed on compacted soil or firm natural material. All walls should be designed with granular backfill and sub drains. The sub drain pipe should be 3" Schedule 40 abs or pvc plastic pipe and be surrounded with a 3/4 gravel which is surrounded by Mirafi 140N filter fabric.

coefficient of friction---------------------------	.35
unit weight of soil----------------------------	115 pcf
active equivalent fluid pressure (level)----	35 pcf
active equivalent fluid pressure (2:1)----	45 pcf
at rest equivalent fluid pressure ---------- (all building walls)	55 pcf
passive equivalent fluid pressure(level)---	350 pcf to max. of 3500
passive equivalent fluid pressure(2:1)---	250 pcf to max. of 2500

If passive earth pressure and friction are combined to provide required resistance to lateral forces, the value of the passive pressure should be reduced to two thirds of the above recommendations. These values may be increased by one third when considering short-term loads such as wind or seismic forces.

8.9 Allowable Safe Bearing Capacity

An allowable safe bearing capacity of 1,500 pounds per square foot (psf) may be used for design of continuous footings that maintain a minimum width of 12-inches and a minimum depth of at least 12-inches below the lowest adjacent grade. The bearing value may be increased by 10% for each additional foot of depth to a maximum of 2,200 psf if there is at least 2 feet of re-compacted soil below the footing or footings placed on bedrock. The allowable bearing capacity may be increased by one-third for short term or temporary loads. Note: This may increase the depth of removal.

Total settlements under static loads of footings supported on properly compacted fill and sized for the allowable bearing pressures are not expected to exceed about 3/4 of one inch. However, Liquefaction in the alluvium of the Sand Creek bed near B-1 and B-5 is minor under current conditions. If there is shallow groundwater the settlement becomes substantial. If water is 20 feet deep with a 0.7 g earthquake (10% probability in 50 years) the total settlement due to liquefaction could be 6 to 8 inches. This could cause settlement and maybe lateral spreading in the valley areas and affecting the building pads as well. Deep removals and the use of geofabric will reduce this liquefaction settlement and liquefaction differential settlement. Differential settlements between footings designed for the maximum recommended bearing value are expected to be less than 1/2-inch in 30 feet if liquefaction were not to occur

8.10 Foundation System Design

Foundation elements should be supported on firm natural soil or properly engineered and approved fill compacted to a minimum of 90 percent of the maximum dry density. For one-story residential or commercial/retail structures, continuous spread footings should be a minimum of 12-inches wide and 18-inches below the lowest adjacent grade. For two and three story residential or commercial/retail structures, continuous spread footings should be a minimum of 15-inches wide and 18-inches below the lowest adjacent grade. As a minimum, all footings should have two No. 4 reinforcing bar placed at the top and bottom of the footing, or as approved by the structural engineer.

Concrete slabs should be underlain with a vapor barrier consisting of a minimum of 10 mil polyvinyl chloride membrane with all laps sealed. The moisture barrier should be placed within a 2-inch layer of clean sand to protect the moisture barrier and aid in the curing of the concrete.

The structural engineer should design all footings and concrete slabs in accordance with the anticipated loads and the soil parameters given.

8.11 Utility Trench Backfill

Utility trench backfill should be compacted to a minimum of 90 percent of the maximum dry density determined in laboratory testing by the ASTM D 1557 test method. It is our opinion that utility trench backfill consisting of on-site or approved sandy soils can best be placed by mechanical compaction to a minimum of 90 percent of the maximum dry density. All trench excavations should be conducted in accordance with Cal-OSHA standards as a minimum. Flooding or Jetting of backfill is not acceptable.

8.12 Surface Drainage

Surface drainage should be directed away from foundations of buildings or appurtenant structures. All drainage should be directed toward streets or approved permanent drainage devices. Where landscaping and planters are proposed adjacent to foundations, subsurface drains should be provided to prevent standing water or saturation of foundations by landscape irrigation water.

8.13 Slope Stability & Construction

Due to the very sandy nature of the overall site and most specifically in the area of the major cut slopes, it is imperative that an engineered and properly supervised mitigation for the slopes occur in order to ensure proper stability. Based on slope stability analysis any slopes with old debris flow and/or out of slope siltstone bedding planes will need mitigation. This includes the slopes above Lot 5 (section A-A'), Lot 6, Lot 8, 9 (section B-B'), Lot 14 (section C-C'), and Lot 15. Loose top soils and fill soils derived from the sandy soils are Surficially stable when saturated to a depth of 3 feet, However they are Surficially unstable when saturated to a depth of 4 feet. These slopes will require some mitigation. The cut slopes of dense sand and Pauba Formation are Surficially stable when saturated to a depth of 4 feet and will only require type slope maintenance. All slopes are nearly uniformly subject to severe erosion during heavy rainfall whether in natural, cut slopes or fill slopes.

There are many options that would meet these criteria and 3 of the most probable are as follows:

- Engineered retaining walls.
- Engineered slope retention systems also commonly referred to as Geotextile Retention and Stability Systems (refer to Appendix for several examples.) If this option is used specifications for construction by the engineers of the providers of the fabric should be followed.
- Engineered slope retention lock block and/or link block systems.
- Geotextile slope erosion fabric on the slope face to reduce erosion.
- Drainage ditches at the top and within slopes to reduce erosion and to reduce saturation of the surface of the slope.

Fill slopes should be overfilled to an extent determined by the contractor, but not less than 2-ft measured perpendicular to the slope face, so that when trimmed back to the design grade a minimum of 90 percent relative compaction is achieved at the slope face. Compaction of each fill lift should extend out to the temporary slope face.

As an alternative to overfilling, fill slopes may be built to the grade providing compaction of each fill lift extends to the face of the slope, and backrolling is undertaken at vertical intervals not exceeding 4-ft in height. Backrolling at more frequent intervals may be required. Care should be taken to avoid spillage of loose materials down the face of any slope during grading.

Proper seeding and planting of the slopes should follow as soon as practical to inhibit erosion and deterioration of the slope surfaces. Proper moisture control will enhance the long-term stability of the finish slope surface.

8.14 Grading Plan Review

EcoTech Remediation Inc. should review the final grading plans, when available, to verify conformance with the intentions of these recommendations.

8.15 Foundation Plan Review

EcoTech Remediation, Inc. should review the final foundation plans, when available, to verify conformance with the intentions of these recommendations.

8.16 Construction Monitoring

Continuous observation and testing under the direction of qualified soils engineer and engineering geologist is essential to verify compliance with the recommendations of this report and to confirm that the geotechnical conditions exposed are consistent with the findings of this investigation. Monitoring of the earthwork operations should be conducted by a qualified engineering geologist and soil engineer at the following stages of construction:

- During the earthwork and develop of all cutslopes
- During alluvial removals and site excavation.
- During the removal of debris flow areas.
- During preparation of the areas to receive compacted fill materials.
- During placement of fill material and benching operations.
- Following excavation of footings for foundations.
- During utility trench backfill operations.
- During sidewalk and curb and gutter operations.
- During sub-grade and base operations.
- When any unusual conditions are encountered during grading.

A final geotechnical report summarizing conditions encountered during grading and soil compaction test results should be submitted upon completion of grading. The final geotechnical report should include an "As-Built" grading plan showing areas of over excavation and fill placement.

9.0 LIMITATIONS OF INVESTIGATION

Our investigation was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable Soils Engineers and Geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

The samples taken and used for testing and the observations made are believed representative of the entire project; however, soil and geologic conditions can vary significantly between test locations.

As in most projects, conditions revealed by excavation may be at variance with preliminary findings. If this occurs, the project Soil Engineer and Geologist must evaluate the changed conditions and provide modified recommendations as required.

This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are brought to the attention of the project architect and engineer. Our specific recommendations should be incorporated into the plans, and the necessary steps taken to see that the contractor and subcontractors carry out such recommendations in the field.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge.

Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and revision as changed conditions are identified.

APPENDIX A

References

LEGEND:

AFu =	Fill placed without engineers testing and approval
Qc/Qal or Qa=	Alluvium and colluvium undifferentiated
Qls =	Landslide debris or debris flow materials
QP =	Pauba Formation

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Seed, H.B., Idriss, I.M., 1982, Ground Motion and Soil Liquefaction during Earthquakes, Earthquake Engineering Research Institute.

Weber, F.H., Jr., 1977, Seismic Hazards Related to Geologic Factors, Elsinore and Chino Fault Zones, Northwestern Riverside County, California Division of Mines and Geology, Open File Report, 77-4 L.A., 96 pages.

AERIAL PHOTOGRAPHS UTILIZED

Source	Date	Photo No.	Scale
Riverside Co. Flood Control	4-14-05	18-27 & 28	1"=1600'
Riverside Co. Flood Control	4-12-00	18-27 & 28	1"=1600'
Riverside Co. Flood Control	01-29-95	18-25 & 26	1"=1600'
Riverside Co. Flood Control	01-28-90	18-27 & 28	1"=1600'
Riverside Co. Flood Control	04-23-84	21387 & 21388	1"=1600'
Riverside Co. Flood Control	06-20-74	692 & 693	1"=2000'

APPENDIX B

Exploratory Boring & Trench Logs

APPENDIX C

Laboratory Test Results

LABORATORY TESTING

A. Classification

Soils were visually classified according to the Unified Soil Classification System. Classification was supplemented by index tests such as particle size analysis and moisture content.

B. Maximum Density/Optimum Moisture Content

Maximum density/optimum moisture content relationships were determined for typical samples of the on-site soils. The laboratory standard used was ASTM 1557-Method A.

C. Particle Size Determination

Particle size determination, consisting of mechanical analyses (sieve), was performed on representative samples of the on-site soils in accordance with ASTM D 422-63.

E. Direct Shear

Direct shear strength tests were performed on representative samples of the on-site undisturbed soils. To simulate possible adverse field conditions, the samples were saturated prior to shearing. A saturating device was used which permitted the samples to absorb moisture while preventing volume change.

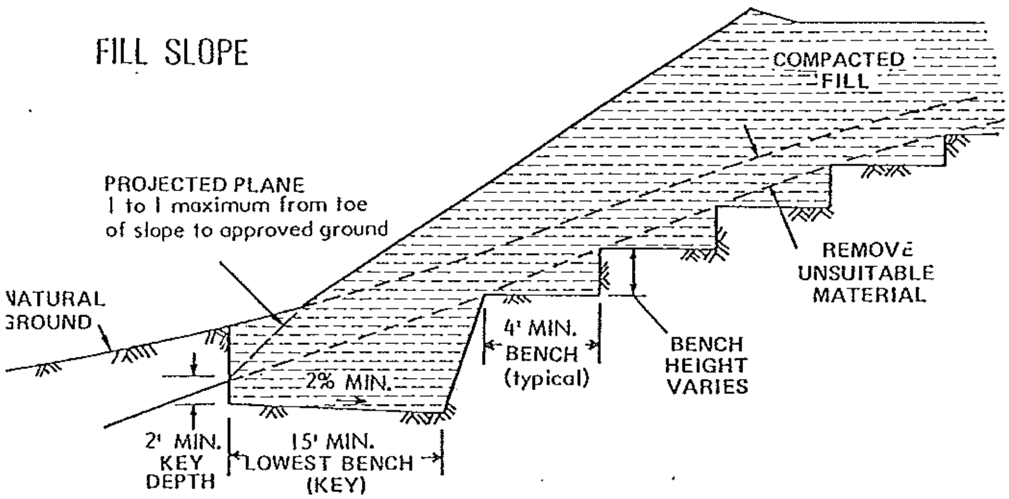
H. Sulfate Content

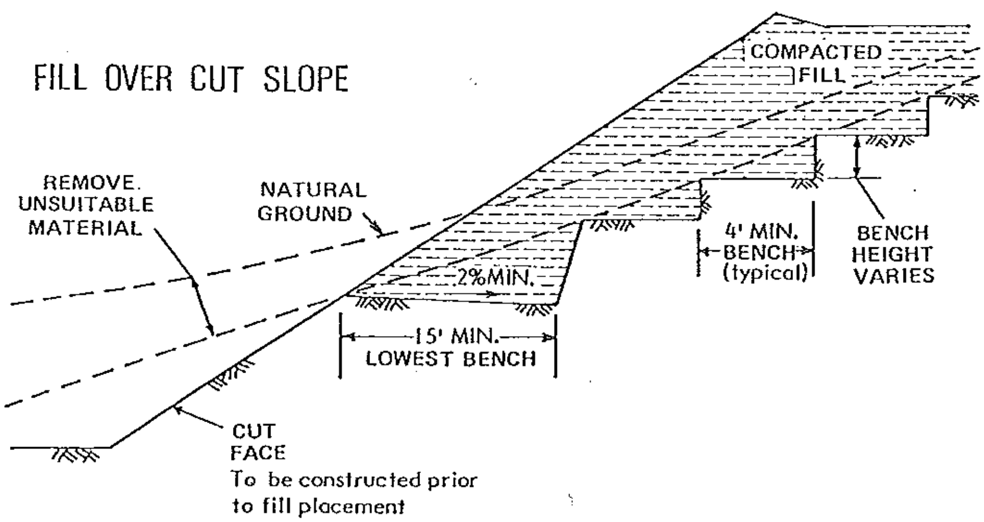
Sulfate content tests were performed on representative samples of the on-site soils. The Hach Kit was used for laboratory screen testing.

APPENDIX D

Standard Earthwork and Grading Specifications

BENCHING DETAILS



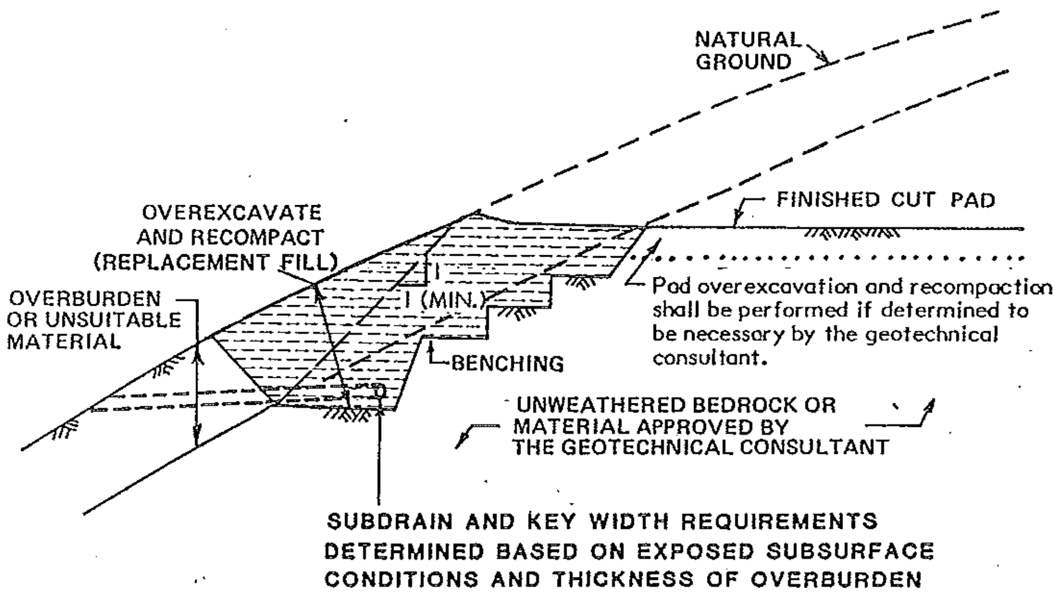


NOTES:

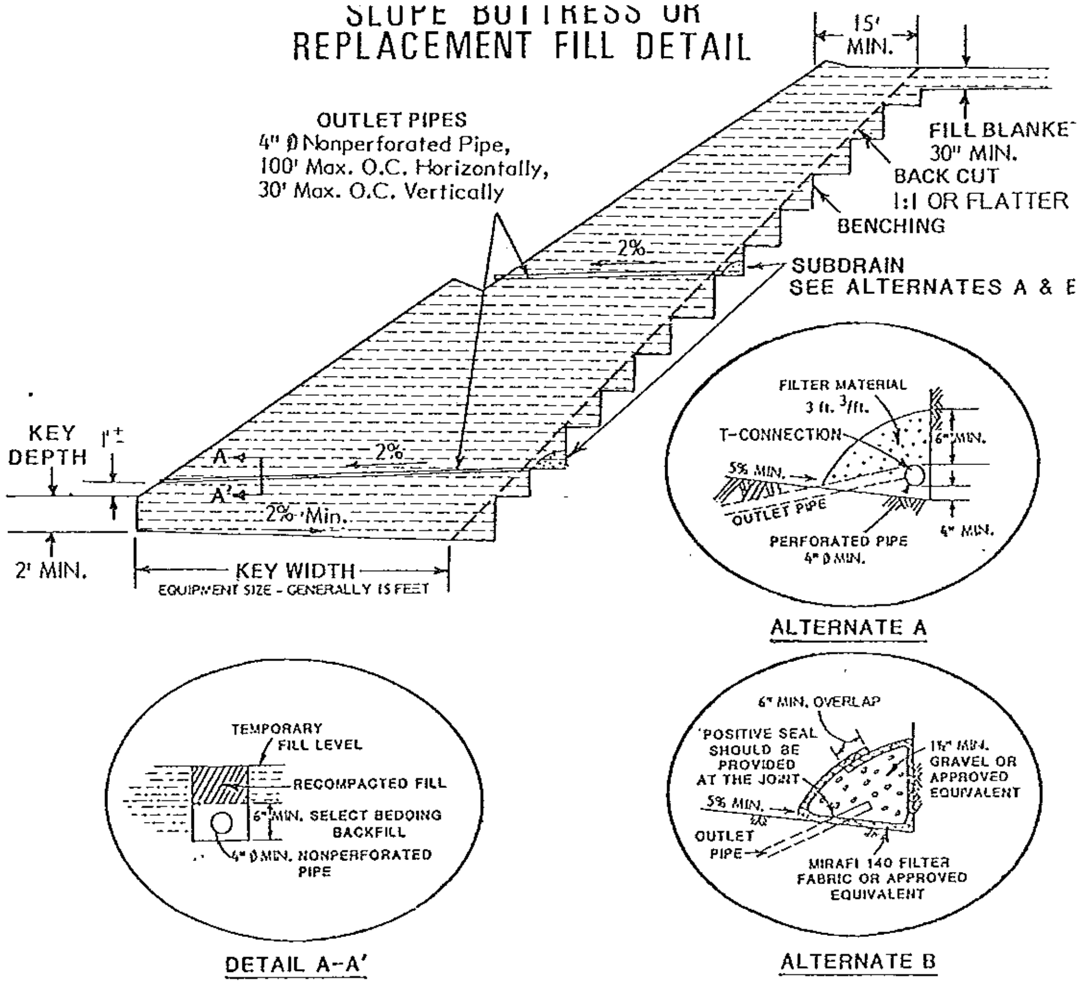
LOWEST BENCH: Depth and width subject to field change based on consultant's inspection.

SUBDRAINAGE: Back drains may be required at the discretion of the geotechnical consultant.

SIDE HILL
CUT PAD DETAIL



SLOPE BUTTRESS OR REPLACEMENT FILL DETAIL



NOTES :

- Fill blanket, back cut, key width and key depth are subject to field change, per report/plans.
- Key heel subdrain, blanket drain, or vertical drain may be required at the discretion of the geotechnical consultant.
- SUBDRAIN INSTALLATION - Subdrain pipe shall be installed with perforations down or, at locations designated by the geotechnical consultant, shall be nonperforated pipe.
- SUBDRAIN TYPE - Subdrain type shall be ASTM C508 Asbestos Cement Pipe (ACP) or ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40 Acrylonitrile Butadiene Styrene (ABS) or ASTM D3034 SDR 23.5 or ASTM D1785, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe or approved

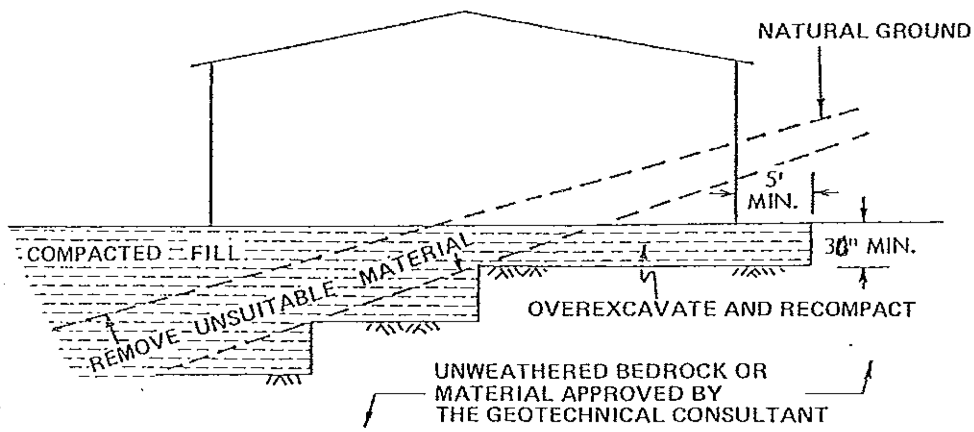
FILTER MATERIAL:

Filter material shall be Class 2 permeable material per State of California Standard Specifications, or approved alternate.
Class 2 grading as follows:

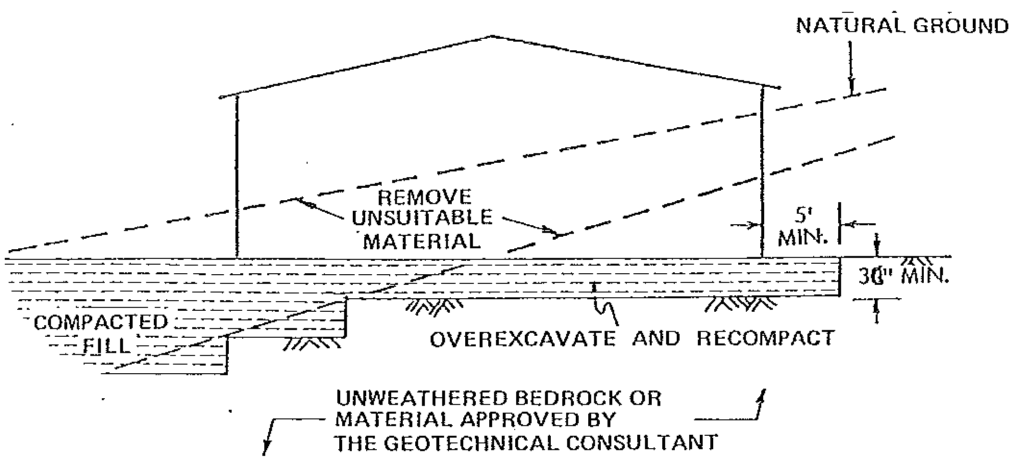
SIEVE SIZE	PERCENT PASSING
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

TRANSITION LOT DETAILS

CUT-FILL LOT

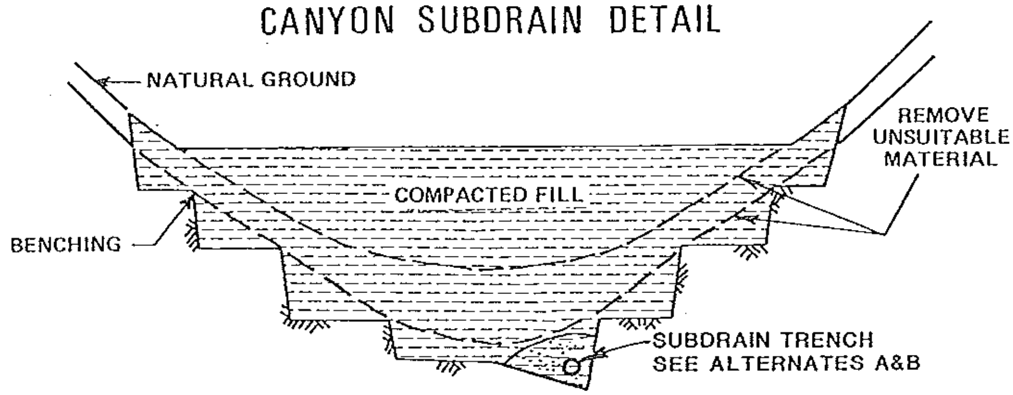


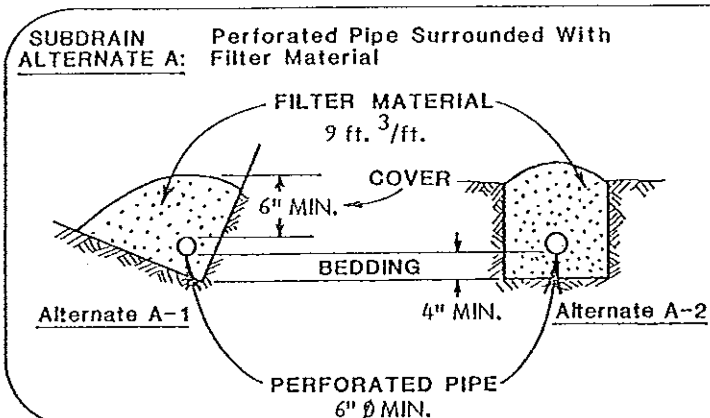
CUT LOT



NOTE:
Deeper overexcavation and recompaction shall be performed if determined to be necessary by the geotechnical consultant.

CANYON SUBDRAIN DETAIL

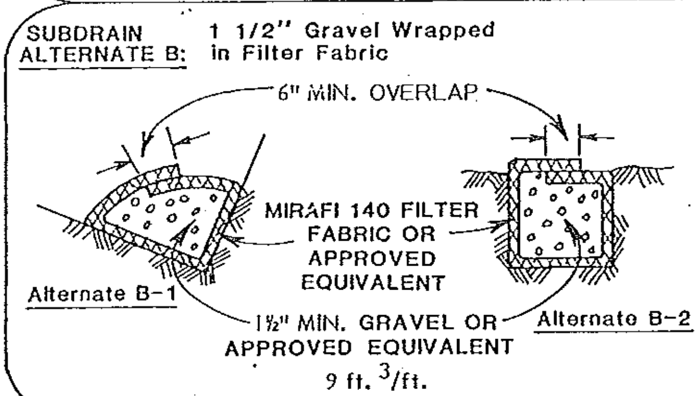




FILTER MATERIAL:

Filter material shall be Class 2 permeable material per State of California Standard Specifications, or approved alternate. Class 2 grading as follows:

SIEVE SIZE	PERCENT PASSING
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

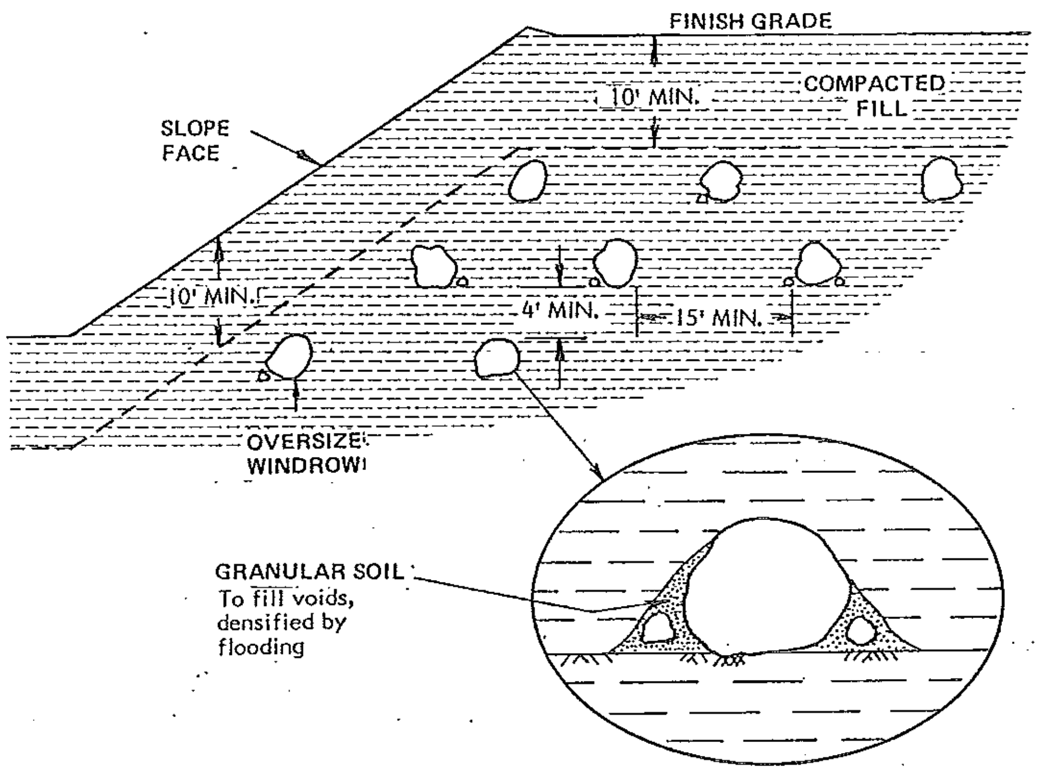


NOTE:

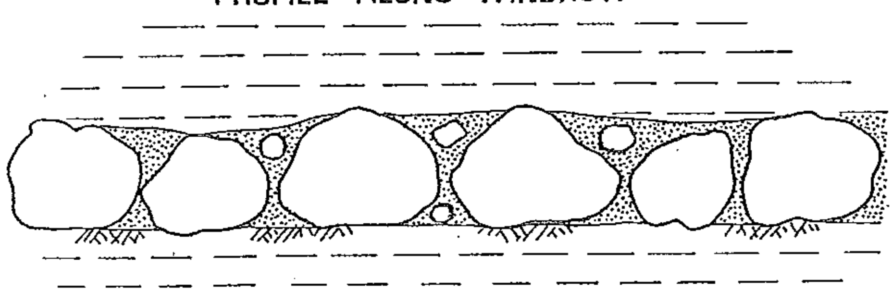
In addition to the wrapped gravel, outlet portion of the subdrain should be equipped with a minimum of 10 feet long perforated pipe connected to a nonperforated pipe having a minimum of 5 feet in length inside the wrapped gravel.

- SUBDRAIN INSTALLATION - Subdrain pipe shall be installed with perforations down or, at locations designated by the geotechnical consultant, shall be nonperforated pipe.
- SUBDRAIN TYPE - Subdrain type shall be ASTM C508 Asbestos Cement Pipe (ACP) or ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40 Acrylonitrile Butadiene Styrene (ABS) or ASTM D3034 SDR 23.5 or ASTM D1785, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe or approved equivalent.

ROCK DISPOSAL DETAIL



PROFILE ALONG WINDROW



Calculations

Surfical Slope Stability

Safety Factor = $(H(\beta) \cos(<)2 \tan(\Theta) + C) \;/\; (\gamma (H) \sin(<) \cos(<))$

<	Slope Angle	26.5	26.5	26.5
	radian	0.462278	0.462278	0.462278
H	Depth of saturation	3	4	4
β	Bouyant Weight of Soil	47.6	47.6	47.6
γ	Total wet Weight of Soil	110	110	110
Θ	Angle of internal friction	31.8	31.8	39
	radian	0.554733	0.554733	0.680333
c	Cohesion	145	145	220
	Safety Factor	1.63886	1.363672	1.95532
		greater then 1.5	less then 1.5	greater then 1.5
		Fill soils and loose top soils	Fill Soils and loose tope soils	Dense Natural sands

Fill Slope Stability

Factor of Safety = $(Ncf\ C) / H\gamma$

$\wedge cq = \gamma H \tan\Theta / C$

H	Height of Slope		30	40	45	50
<	Angle of Slope		26.5	26.5	26.5	26.5
	radians		0.462278	0.462278	0.462278	0.462278
γ	total wet weight of soil		110	110	110	110
Θ	Angle of internal friction		31.8	31.8	31.8	31.8
	radians		0.554733	0.554733	0.554733	0.554733
C	Cohesion		145	145	145	145
$\wedge cq$	$= \gamma H \tan\Theta / C$	=	14.10208	18.80277	21.15312	23.50346
Ncf	From Graph		40	52	58	60
	Factor of Safety		1.757576	1.713636	1.69899	1.581818

CUT Slope Stability Pauba Formation

Factor of Safety = (Ncf C) / H γ

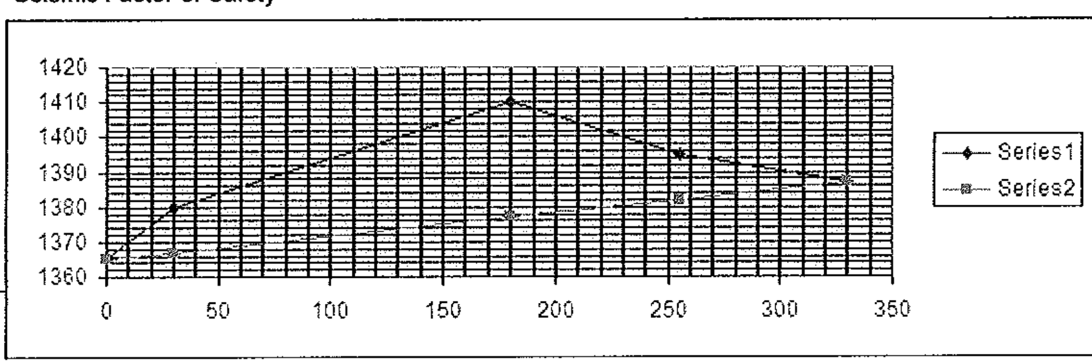
^cq = γH tanΘ / C

H	Height of Slope	20	40	60
<	Angle of Slope	26.5	26.5	26.5
	radians	0.462278	0.462278	0.462278
γ	total wet weight of soil	110	110	110
Θ	Angle of internal friction	40.3	40.3	40.3
	radians	0.703011	0.703011	0.703011
C	Cohesion	593	593	593
^cq	= γH tanΘ / C =	3.143992	6.287985	9.431977
Ncf	From Graph	15.5	22.5	30
	Factor of Safety	4.177955	3.032386	2.695455

Slope Stability Analysis

Project: Sand Creek
Section A-A' Siltstone bed at top of slope, Lot 5

Section No	1	2	3	4	5	
Section X coordinate	0	30	180	255	330	
Section Y coordinate te top	1365	1380	1410	1395	1387	
Section Y corrdinate bottom	1365	1367	1377	1382	1387	
Unit weight	115	115	115	115	115	
Cohesion	100	100	100	100	100	
angle of Friction	10	10	10	10	10	
Friction in radians radians =Q	0.174533	0.174533	0.174533	0.174533		
Section width	30	150	75	75		
Section area	195	3450	1725	487.5		
Section weight	22425	396750	198375	56062.5		
Section length	32.69557	153.5871	76.11833	75		
Section angle in radians	0.066568	0.066568	0.066568	0.066568		
Section angle in degrees	3.814075	3.814075	3.814075	3.814075		
W/cos&	22474.78	397630.7	198815.3	56186.95		
(Wcons&)tanQ	3962.91	70113.02	35056.51	9907.274		
CL + (Wcos&)tan Q	7232.466	85471.73	42668.34	17407.27		
(1+(tan& tanQ)/F1)=M&1	1.003562	1.003562	1.003562	1.003562		
(1+(tan& tanQ)/F2)=M&2	1.011755	1.011755	1.011755	1.011755		
(Cl+(Wcos&))/M&1	7206.795	85168.35	42516.89	17345.49		
Wsin&	1491.689	26391.42	13195.71	3729.222		
(CL+(Wcos&))/M&2	7148.436	84478.67	42172.6	17205.03		
.15Wcos&+Wsin&	4847.989	85772.11	42886.05	12119.97		
Sum of Static Resisting						152237.5
Sum of Static Driving						44808.04
Sum of Seismic Resisting						151004.7
Sum of Seismic Driving						145626.1
F1=Static Factor of Safety						3.397549
F2=Seismic Factor of Safety						1.036934

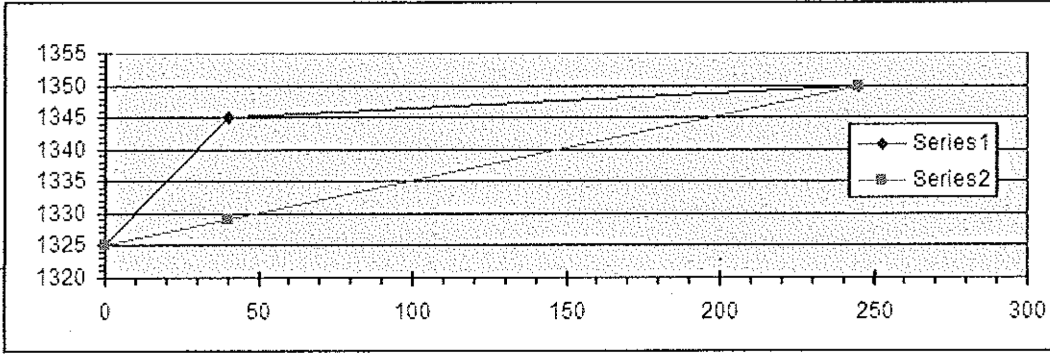


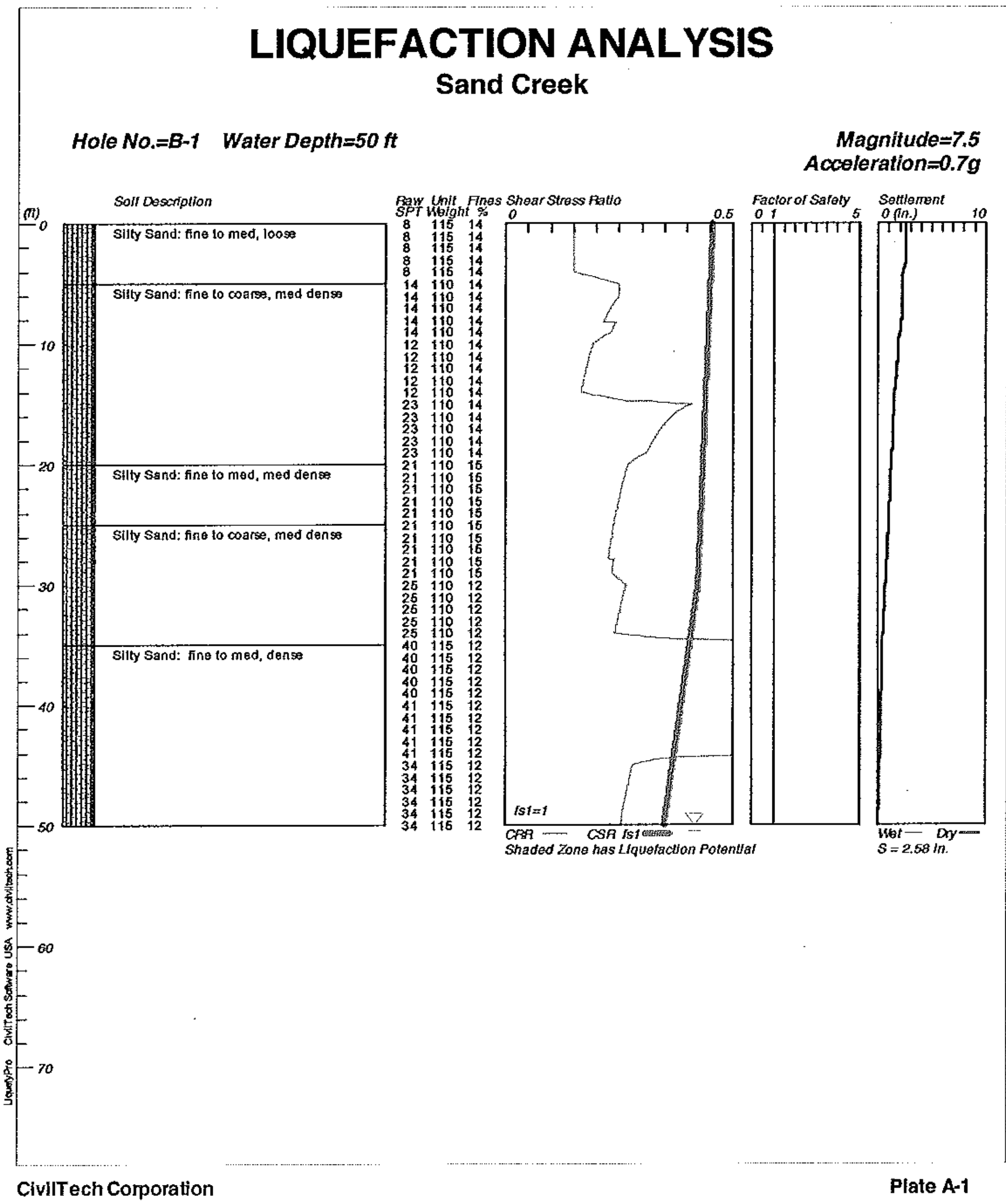
Slope Stability Analysis

Project: Sand Creek
Section C-C' Debris Flow, Lot 14

Section No	1	2	3
Section X coordinate	0	40	245
Section Y coordinate top	1325	1345	1350
Section Y corrdinate bottom	1325	1329	1350
Unit weight	115	115	
Cohesion	100	100	
angle of Friction	10	10	
Friction in radians =Q	0.174533	0.174533	
Section width	40	205	
Section area	320	1640	
Section weight	36800	188600	
Section length	43.08132	205	
Section angle in radians	0.099669	0.102083	
Section angle in degrees	5.710593	5.848922	
W/cos&	36983.54	189587	
(Wcons&)tanQ	6521.196	33429.3	
CL + (Wcos&)tan Q	10829.33	53929.3	
(1+(tan& tanQ)/F1)=M&1	1.006297	1.006451	
(1+(tan& tanQ)/F2)=M&2	1.01603	1.016421	
(Cl+(Wcos&))/M&1	10761.56	53583.63	
Wsin&	3661.737	19219.42	
(CL+(Wcos&))/M&2	10658.48	53058.05	
.15Wcos&+Wsin&	9154.342	47362.15	

Sum of Static Resisting	64345.19
Sum of Static Driving	22881.16
Sum of Seismic Resisting	63716.53
Sum of Seismic Driving	56516.49
F1=Static Factor of Safety	2.812147
F2=Seismic Factor of Safety	1.127397





LIQUEFACTION ANALYSIS
Sand Creek
Hole No.=B-1 Water Depth=50 ft
Magnitude=7.5
Acceleration=0.7g
Soil Description
(ft)
0
10
20
30
40
50
60
70
Silty Sand: fine to med, loose
Silty Sand: fine to coarse, med dense
Silty Sand: fine to med, med dense
Silty Sand: fine to coarse, med dense
Silty Sand: fine to med, dense
Raw SPT Unit Weight Fines %
8 115 14
8 115 14
8 115 14
8 115 14
8 115 14
14 110 14
14 110 14
14 110 14
14 110 14
14 110 14
12 110 14
12 110 14
12 110 14
12 110 14
12 110 14
23 110 14
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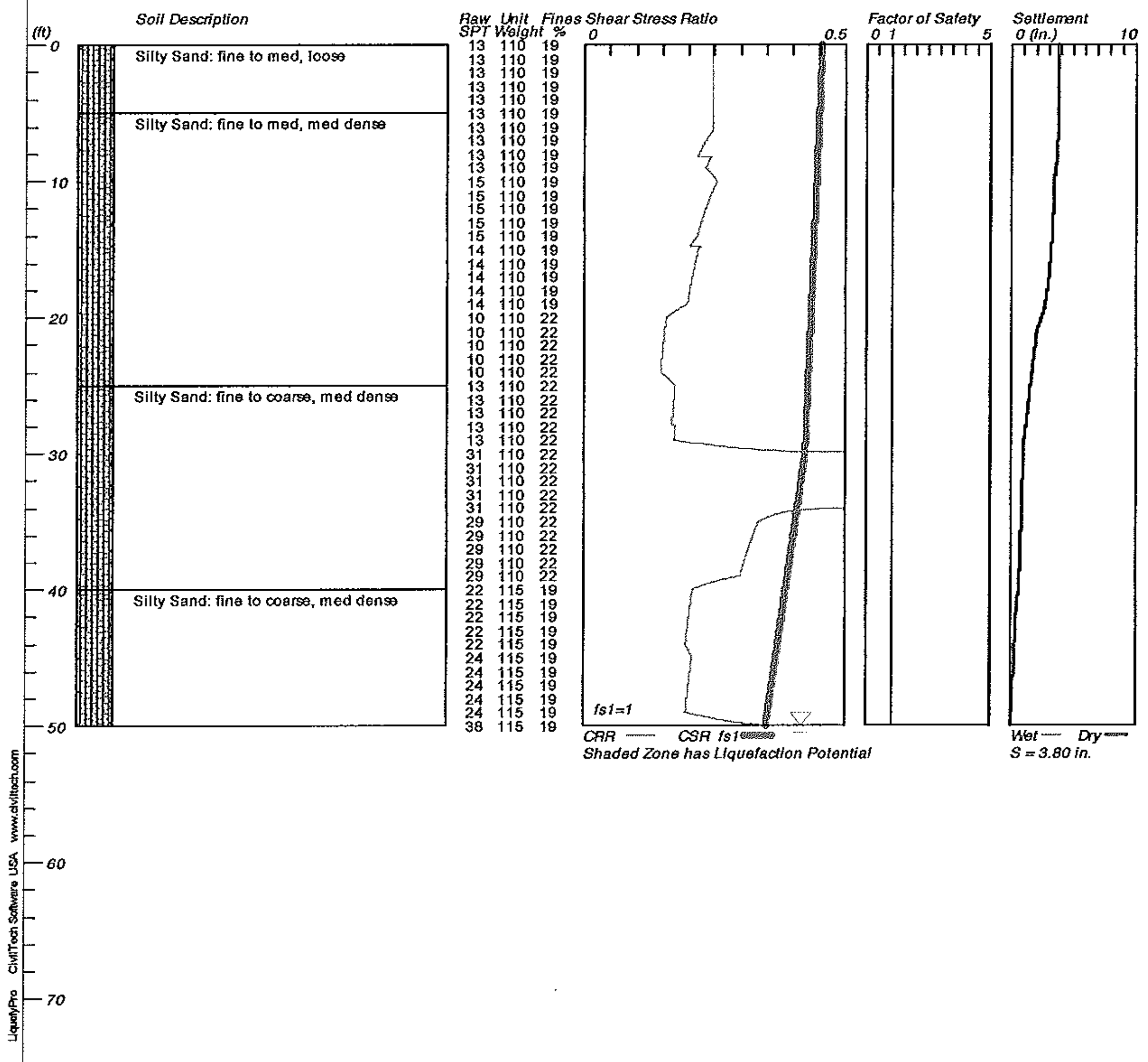
25 110 12
25 110 12
25 110 12
25 110 12
25 110 12
40 115 12
40 115 12
40 115 12
40 115 12
40 115 12
41 115 12
41 115 12
41 115 12
41 115 12
41 115 12
34 115 12
34 115 12
34 115 12
34 115 12
34 115 12
34 115 12
Shear Stress Ratio
0
0.5
fs1=1
CRR — CSR fs1 —
Shaded Zone has Liquefaction Potential
Factor of Safety
0 1 5
Settlement
0 (in.) 10
Wet — Dry —
S = 2.58 in.
LiquefyPro CivilTech Software USA www.civiltech.com
CivilTech Corporation
Plate A-1

LIQUEFACTION ANALYSIS
Sand Creek

Hole No.=B-5 Water Depth=50 ft

Magnitude=7.5
Acceleration=0.7g



CivilTech Corporation

Plate A-1

These specifications present standard recommendations for grading and earthwork.

No deviation from these specifications should be permitted unless specifically superseded in the geotechnical report of the project or by written communication signed by the Geotechnical Consultant. Evaluations performed by the Geotechnical Consultant during the course of grading may result in subsequent recommendations which could supersede these specifications or the recommendations of the geotechnical report.

1.0 GENERAL

1.1 The Geotechnical Consultant is the Owner's or Developer's representative on the project. For the purpose of these specifications, observations by the Geotechnical Consultant include observations by the Soils Engineer, Geotechnical Engineer, Engineering Geologist, and others employed by and responsible to the Geotechnical Consultant.

1.2 All clearing, site preparation, or earthwork performed on the project shall be conducted and directed by the Contractor under the **allowance or supervision** of the Geotechnical Consultant.

1.3 The Contractor should be responsible for the safety of the project and satisfactory completion of all grading. During grading, the Contractor shall remain accessible.

1.4 Prior to the commencement of grading, the Geotechnical Consultant shall be employed for the purpose of providing field, laboratory, and office services for conformance with the recommendations of the geotechnical report and these specifications. It will be necessary that the Geotechnical Consultant provide adequate testing and observations so that he may provide an opinion as to determine that the work was accomplished as specified. It shall be the responsibility of the Contractor to assist the Geotechnical Consultant and keep him apprized of work schedules and changes so that he may schedule his personnel accordingly.

1.5 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes, agency ordinances, these specifications, and the approved grading plans. If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as questionable soil, poor moisture condition, inadequate compaction, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant will be empowered to reject the work and recommend that construction be stopped until the conditions are rectified.

1.6 It is the Contractor's responsibility to provide safe access to the Geotechnical Consultant for testing and/or grading observation purposes. This may require the excavation of test pits and/or the relocation of grading equipment.

1.7 A final report shall be issued by the Geotechnical Consultant attesting to the Contractor's conformance with these specifications.

2.0 SITE PREPARATION

2.1 All vegetation and deleterious material shall be disposed of off-site. This removal shall be observed by the Geotechnical Consultant and concluded prior to fill placement.

2.2 Soil, alluvium, or bedrock materials determined by the Geotechnical Consultant as being unsuitable for placement in compacted fills shall be removed from the site or used in open areas as determined by the Geotechnical Consultant. Any material incorporated as a part of a compacted fill must be approved by the Geotechnical Consultant prior to fill placement.

2.3 After the ground surface to receive fill has been cleared, it shall be scarified, disced and/or bladed by the Contractor until it is uniform and free from ruts, hollows, hummocks, or other uneven features which may prevent uniform compaction.

The scarified ground surface shall then be brought to optimum moisture, mixed as required, and compacted as specified. If the scarified zone is greater than twelve inches in depth, the excess shall be removed and placed in lifts not to exceed six inches or less.

Prior to placing fill, the ground surface to receive fill shall be observed, tested, and approved by the Geotechnical Consultant.

2.4 Any underground structures or cavities such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipe lines, or others are to be removed or treated in a manner prescribed by the Geotechnical Consultant.

2.5 In cut-fill transition lots and where cut lots are partially in soil, colluvium or unweathered bedrock materials, in order to provide uniform bearing conditions, the bedrock portion of the lot extending a minimum of 5 feet outside of building lines shall be over excavated a minimum of 3 feet and replaced with compacted fill. Greater over excavation could be required as determined by Geotechnical Consultant. Typical details are attached.

3.0 COMPACTED FILLS

3.1 Material to be placed as fill shall be free of organic matter and other deleterious substances, and shall be approved by the Geotechnical Consultant. Soils of poor gradation, expansion, or strength characteristics shall be placed in areas designated by Geotechnical Consultant or shall be mixed with other soils to serve as satisfactory fill material, as directed by the Geotechnical Consultant.

3.2 Rock fragments less than six inches in diameter may be utilized in the fill, provided:

- They are not placed or nested in concentrated pockets.
- There is a sufficient amount of approved soil to surround the rocks.
- The distribution of rocks is supervised by the Geotechnical Consultant.

3.3 Rocks greater than twelve inches in diameter shall be taken off-site, or placed in accordance with the recommendations of the Geotechnical Consultant in areas designated as suitable for rock disposal. (A typical detail for Rock Disposal is attached.)

3.4 Material that is spongy, subject to decay, or otherwise considered unsuitable shall not be used in the compacted fill.

3.5 Representative samples of materials to be utilized as compacted fill shall be analyzed by the laboratory of the Geotechnical Consultant to determine their physical properties. If any material other than that previously tested is encountered during grading, the appropriate analysis of this material shall be conducted by the Geotechnical Consultant before being approved as fill material.

3.6 Material used in the compacting process shall be evenly spread, watered, processed, and compacted in thin lifts not to exceed six inches in thickness to obtain a uniformly dense layer. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Consultant.

3.7 If the moisture content or relative compaction varies from that required by the Geotechnical Consultant, the Contractor shall rework the fill until it is approved by the Geotechnical Consultant.

3.8 Each layer shall be compacted to at least 90 percent of the maximum density in compliance with the testing method specified by the controlling governmental agency or ASTM 1557-70, whichever applies.

If compaction to a lesser percentage is authorized by the controlling governmental agency because of a specific land use or expansive soil condition, the area to receive fill compacted to less than 90 percent shall either be delineated on the grading plan and/or appropriate reference made to the area in the geotechnical report.

3.9 All fills shall be keyed and benched through all topsoil, colluvium, alluvium, or creep material, into sound bedrock or firm material where the slope receiving fill exceeds a ratio of five horizontal to one vertical or in accordance with the recommendations of the Geotechnical Consultant.

3.10 The key for side hill fills shall be a minimum width of 15 feet within bedrock or firm materials, unless otherwise specified in the geotechnical report. (See detail attached.)

3.11 Sub-drainage devices shall be constructed in compliance with the ordinances of the controlling governmental agency, or with the recommendations of the Geotechnical Consultant. (Typical Canyon Sub-drain details are attached.)

3.12 The contractor will be required to obtain a minimum relative compaction of at least 90 percent out to the finish slope face of fill slopes, buttresses, and stabilization fills. This may be achieved by either over building the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment, or by any other procedure which produces the required compaction approved by the Geotechnical Consultant.

3.13 All fill slopes should be planted or protected from erosion by other methods specified in the Geotechnical report.

3.14 Fill-over-cut slopes shall be properly keyed through topsoil, colluvium or creep material into rock or firm materials, and the transition shall be stripped of all soil prior to placing fill. (See attached detail.)

4.0 CUT SLOPES

4.1 The Geotechnical Consultant shall inspect all cut slopes at vertical intervals exceeding five feet.

4.2 If any conditions not anticipated in the geotechnical report such as perched water, seepage, lenticular or confined strata of a potentially adverse nature, unfavorably inclined bedding, joints or fault planes encountered during grading, these conditions shall be analyzed by the Geotechnical Consultant, and recommendations shall be made to mitigate these problems. (Typical details for stabilization of a portion of a cut slope are attached.)

4.3 Cut slopes that face in the same direction as the prevailing drainage shall be protected from slope wash by a non-erodible interceptor swale placed at the top of the slope.

4.4 Unless otherwise specified in the geotechnical report, no cut slopes shall be excavated higher or steeper than that allowed by the ordinances of controlling governmental agencies.

4.5 Drainage terraces shall be constructed in compliance with the ordinances of controlling governmental agencies, or with the recommendations of the Geotechnical Consultant.

5.0 TRENCH BACKFILLS

5.1 Trench excavation shall be inspected prior to structure placement for competent bottom.

5.2 Trench excavations for utility pipes shall be backfilled under the supervision of the Geotechnical Consultant.

5.3 After the utility pipe has been laid, the space under and around the pipe shall be backfilled with clean sand or approved granular soil to a depth of at least one foot over the top of the pipe. The sand backfill shall be uniformly jetted into place before the controlled backfill is placed over the sand.

5.4 The on-site materials, or other soils approved by the Geotechnical Consultant, shall be watered and mixed, as necessary, prior to placement in lifts over the sand backfill.

5.5 The controlled backfill shall be compacted to at least 90 percent of the maximum laboratory density, as determined by the ASTM D1557-70 or the controlling governmental agency.

5.6 Field density tests and inspection of the backfill procedures shall be made by the Geotechnical Consultant during backfilling to see that proper moisture content and uniform compaction is being maintained. The contractor shall provide test holes and exploratory pits as required by the Geotechnical Consultant to enable sampling and testing.

6.0 GRADING CONTROL

6.1 Inspection of the fill placement shall be provided by the Geotechnical Consultant during the progress of grading.

6.2 In general, density tests should be made at intervals not exceeding two feet of fill height or every 500 cubic yards of fill placed. This criteria will vary depending on soil conditions and the size of the job. In any event, an adequate number of field density tests shall be made to verify that the required compaction is being achieved.

6.3 Density tests should also be made on the native surface material to receive fill, as required by the Geotechnical Consultant.

6.4 All clean-out, processed ground to received fill, key excavations, sub-drains, and rock disposals should be inspected and approved by the Geotechnical Consultant prior to placing any fill. It shall be the Contractor's responsibility to notify the Geotechnical Consultant when such areas will be ready for inspection.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Erosion control measures, when necessary, shall be provided by the Contractor during grading and prior to the completion and construction of permanent drainage controls.

7.2 Upon completion of grading and termination of inspections by the Geotechnical Consultant, no further filling or excavating, including that necessary for footings foundations, large tree wells, retaining walls, or other features shall be performed without the approval of the Geotechnical Consultant.

7.3 Care shall be taken by the Contractor during final grading to preserve any berms, drainage terraces, interceptor swales, or other devices of permanent nature on or adjacent to the property.