APPENDIX F

HYDROLOGY AND WATER QUALITY

GEOLOGIC
AND
SOIL ENGINEERING FEASIBILITY REPORT
FOR
PARAISO HOT SPRINGS SPA RESORT
MONTEREY COUNTY, CALIFORNIA

PROJECT LSW-0337-01

Prepared for

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DECEMBER 2004

LANDSET

December 31, 2004

File No.: LSW-0337

Mr. John M. Thompson
Thompson Holdings, L.L.C.
P.O. Box 2367
Horsham, PA 19044

SUBJECT: GEOLOGIC AND SOIL ENGINEERING FEASIBLITY REPORT
Paraiso Hot Springs Spa Resort
Paraiso Springs Road
Soledad Greenfield Area of Monterey County, California

Dear Mr. Thompson:

In accordance with your authorization, Landset Engineers, Inc has completed a geologic and soil engineering feasibility report for a proposed spa resort located west of the Soledad Greenfield area of Monterey County, California. This report presents the results of our field investigation, laboratory testing, along with our conclusions and recommendations for site development.

It is our opinion that the proposed spa resort is feasible from a soil engineering and geologic standpoint. However, portions of the site have a *high potential for liquefaction susceptibility*. We recommend that an additional site-specific supplemental liquefaction study be performed in accordance with the guidelines of the California Division of Mines & Geology, Special Publication 117.

The recommendations included in this report are preliminary and contingent upon the findings of the recommended supplemental liquefaction study. Additionally, it is recommended that design level soil engineering and engineering geologic investigation(s) should be performed once preliminary development plans have been completed and proposed land use, types of structures, and anticipated loads are known. The conclusions and recommendations included herein are based upon applicable standards at the time this report was prepared.

It has been a pleasure to be of service to you on this project. If you have any questions regarding the attached report, please contact the undersigned at (831) 443-6970

Respectfully submitted,
LandSet Engineers, Inc.

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TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

INTRODUCTION

This report summarizes the findings, conclusions, and recommendations for our geologic and soil engineering feasibility report for a proposed spa resort on an approximate 280-acre site located at Paraiso Hot Springs west of the Soledad/Greenfield area of Monterey County, California (see Vicinity Map, Figure 1).

We utilized the following plan during the course of the investigation:

Aerial Topo Map, Scale 1"=100', prepared by Bestor Engineers, Inc.

PURPOSE AND SCOPE OF SERVICES

Geologic Report. This report addresses the feasibility of the planned resort development from a geologic viewpoint, with emphasis on the potential for geologic/seismic-related hazards. Our studies included the following:

A. Research, review, and evaluation of data from published and unpublished geologic reports and maps pertaining to the site and vicinity. Most of the previously published geologic information on this area is preliminary in nature, and is based on reconnaissance techniques and extrapolation of data.
B. Examination and interpretation of 4 sets of stereo aerial photographs from 1949, 1956, 1997, & 2000, that cover the site and its vicinity. These photographs were scrutinized for site geology, terrain features characteristic of active fault zones and for landsliding features.
C. Geological site reconnaissance and mapping of the site to observe outcrops and identify those geologic features indicative of existing and potential geologic hazards.
D. Analysis of the data generated and preparation of a written report and maps presenting our findings, conclusions and recommendations addressing the following:

- Site geology
- Faulting
- Liquefaction Potential
- Landsliding
- Ground Shaking
- Erosion

Soil Engineering Feasibility Investigation. This soil engineering feasibility investigation has been prepared to explore surface and subsurface soil and groundwater conditions at the site, and provide preliminary soil-engineering criteria for construction of the project.

The conclusions and recommendations of this report were accomplished in general conformance with the standards noted, as modified by standard soil engineering practice in this area. Our scope of services included:

1. A visual site reconnaissance.
2. Review of available soil engineering data in our files pertinent to the site.
3. Exploration, sampling and classification of the surface and subsurface soils by means of drilling 29 exploratory borings.
4. Laboratory testing of selected soil samples collected from the exploratory borings and surface locations to determine their pertinent engineering and index properties.
5. Engineering analysis of the information collected based on the results of the field exploration including a laboratory testing program and review of published and unpublished studies in the general area of the site.
6. Preparation of this report summarizing our findings, soil engineering conclusions, and recommendations for site preparations, grading and compaction, foundations, utility trenches, slabs-on-grade, general site drainage, and erosion control.

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The site is located at approximately 36º19.878' N latitude, 121º22.059' W longitude in the southwest quarter of the northwest quadrant of the Paraiso Springs 7.5’ minute quadrangle in Monterey County, California. The site is sectionalized and is located in the southwest quarter of Sect. 30, T 18S, R 6E, and the southeast quarter of Sect. 25, T 18S, R 5E. Access to the site is gained via Paraiso Road. Surrounding land uses are agricultural and rural residential (Figure 1, Vicinity Map).

The site consists of a rectangular shaped parcel encompassing approximately 280 acres. The site is predominantly steep southwest and northeast facing slopes. Two northeast / southwest trending valleys occupy the approximate center of the site, Paraiso Springs Valley to the south, and Indian Valley to the north. The site is located between the crest of the Sierra De Salinas and the Salinas Valley (Figures 1 & 5). Existing site improvements include a barn, a “clubhouse”, many small shacks, and mobile homes. An active hot spring and associated spa and pools are also located on site. Many wells, operative and inoperative, are located on the site.

Vegetative cover on the 280-acre site consists of native grasses, weeds, trees, and chaparral in the bottoms of Paraiso Springs Valley and Indian Valley. The slopes to the south of Paraiso Springs Valley and Indian Valley are generally oak woodland. Slopes on the north side of Paraiso Springs Valley and Indian Valley are chaparral. Drainage of the site is by sheet flow to the drainages of Paraiso Springs Valley and Indian Valley. In the Paraiso Springs Valley drainage of site water also occurs through spring and seep discharge. These drainages are unnamed and flow to the east where they join the Arroyo Seco River. Drainage of the Arroyo Seco River is north to the Salinas River, which eventually discharges into the Monterey Bay.

We understand that the proposed site development will consist of the construction of a destination spa resort with residential structures, restaurants, and shops. Preliminary architectural drawings were available for our review at the time of this report. Other site improvements will consist of new access roads, sewage effluent disposal systems, underground utility and landscaping improvements (see Relative Geologic Hazards Map, Sheet 3).

FIELD EXPLORATION

The site was mapped in the field on August 10, 11, and 12, 2004 on the Aerial Topo Map prepared by Bestor Engineers, Inc. The field and aerial photograph mapping was then compiled on the Aerial Topo Map at a scale of 1”=200’ (Site Geologic Map, Sheet 1).

As part of our soil engineering feasibility report 29 exploratory borings were drilled on August 23, 24, 25, 2004. The approximate locations of the exploratory borings are shown on the Site Geologic Map, Sheet 1, located in the map pocket at the back of this report. The borings were drilled using a truck mounted drill rig equipped with an 8-inch outside diameter hollow stem hydraulic powered auger and a truck mounted drill rig with a 4-inch outside diameter solid stem hydraulic powered auger. The exploratory borings were drilled to depths ranging from 5.5 to 60.0 feet below the ground surface. A Certified Engineering Geologist and a staff geologist from our office logged the exploratory borings. Soils encountered in each test boring were visually classified in the field and a continuous log was recorded. Visual classifications were made in general accordance with the Unified Soil Classification System and ASTM D2487. Logs of the soil engineering borings can be found in Appendix A.

LABORATORY TESTING

Laboratory tests were performed to determine some of the physical and engineering characteristics of selected soil samples considered pertinent to the design of the project. The tests performed were selected on the basis of the probable design requirements as correlated to the site subsurface profile. A summary of the laboratory test results is presented in Appendix C. A brief generalized description of the tests performed is presented below.

- Moisture-Density Determinations: This test was conducted on samples taken with fiberglass liners to measure their in-situ moisture contents and dry unit weights. The test results are used to assess the distribution of subsurface pressures and to calculate degrees of in-situ relative compaction.

- Atterberg Limits: This test was performed on two disturbed bulk samples and four liner samples to determine their liquid limit and plastic limit index values. This test provides water content values for the sample’s liquid and plastic phases. This test aids in determining the expansive potential and other engineering characteristics of the soil.

- Grain Size Distribution (Gradation) Analysis: A grain size distribution analysis was performed on selected 2.5”, 1.0”, and bulk soil samples. The grain size distribution is used to determine the classification of the site soils. This information is used for foundation design analysis.

REGIONAL GEOLOGY

The site is situated on the east flank of the Sierra De Salinas on the west side of the Salinas Valley and is part of the Coast Ranges Geomorphic Province of California (Figure 2, Regional Geologic Map). The Coast Ranges Geomorphic Province consists of a series of mountain ranges paralleling the northwest-southeast structural orientation of the San Andreas fault, San Gregorio-Palo Colorado fault, Rinconada fault, Monterey Bay/Tularcitos fault, and other faults within the central coast of California (Figure 5, Regional Fault and Seismicity Map). These faults are characterized by a combination of strike-slip and reverse displacement and show horizontal displacements from tens to hundreds of miles. Several periods of continuous and semi-continuous strike-slip or “transform” movement throughout the late Cenozoic Era has occurred on the San Andreas and related fault systems causing compressional uplift of the mountains of the Coast Ranges Geomorphic Province. The region continues to be characterized by moderate to high rates of seismic and tectonic activity (Figure 5).

The San Andreas fault forms the boundary between the North American and Pacific plates. The site is located on the Pacific Plate on the southwest side of the San Andreas fault. The southwest side of the San Andreas fault is underlain by Pre Cretaceous Sierra De Salinas Schist and Cretaceous age Salinian Block granitic rocks with older Paleozoic Era (?) Sur Series metamorphic rocks that occur as roof pendants. These roof pendants predominantly consist of marble and dolomite (Compton, 1966). Overlying the granitic rocks of the Salinian Block is a series of folded and faulted Tertiary age (Oligocene to middle Miocene) sandstones, conglomerates, and volcanics (Dibblee, 1974).

During very late Tertiary (?) to mid Quaternary times, extensive alluvial and fluvial sediments were shed off of Tertiary uplands and deposited as extensive alluvial fans and the Paso Robles Formation, (Dibblee, 1974). These sediments unconformably overlie all older formations with which they are in contact. Holocene activity has consisted of continued tectonic uplift and down cutting and deposition of the local area streams, mass wasting of upland areas by landslides and erosion, and fault creep along the San Andreas and related fault systems. The geology of the site and its vicinity is depicted on the Geologic Vicinity Map, Figure 3.

REGIONAL FAULTING AND SEISMICITY

The closest faults that would most likely effect the site are the San Andreas, Rinconada, San Gregorio – Palo Colorado, and Monterey Bay Tularcitos faults (Figure 5).

San Andreas Fault

The San Andreas Fault is located about 30-km northeast of the site (Figure 5) and is the major seismic hazard in northern California. The San Andreas fault is a major right-lateral strike-slip fault that generally delineates the transform plate boundary between the North American and Pacific Plates. Trending to the northwest southeast, the San Andreas fault is nearly vertical as evidenced by the relatively straight outcrop pattern across topography of noticeable relief. Historic earthquakes on the San Andreas fault have caused extensive damage and very strong ground shaking in Monterey County. The 1906 (M_w~8.0) “San Francisco earthquake” ruptured a portion of the active San Andreas fault from approximately San Juan Bautista to Cape Mendocino, causing severe damage in parts of the Monterey-San Francisco Bay area. The earthquake occurred on April 18, 1906 and caused severe ground shaking and structural damage to buildings in Monterey and San Benito Counties (Lawson, 1908). The 1989 (M_w 7.1) Loma Preita earthquake also caused significant damage in the Monterey Bay area.

The San Andreas fault has been divided into several different segments that are characterized by varying slip rates, earthquake intensities, and earthquake recurrence intervals. The closest segment of the San Andreas fault to the site is the (Creeping Segment) at 30-km. The San Andreas fault Creeping Segment can expect a (M6.2) earthquake with a recurrence interval of approximately 61 years (Cao et al, 2003). The next closest segment is the (Santa Cruz Mtn. segment) at 56-km from the site. This segment can expect a (M7.0) with a recurrence interval of 218 years (WGCEP, 2002). Stronger earthquakes could be experienced at the site similar to the 1906 event with a maximum magnitude of (M7.9).

Rinconada Fault

The Rinconada Fault is a major structural feature along which granitic rocks of the Sierra de Salinas were uplifted to form the western border of the Salinas Valley and is located about 1.5-km. east of the site. The Rinconada fault in the vicinity of the site is within the Salinian Block and movement began during early Cenozoic time (Paleocene) and remained active to late Pleistocene time (Dibblee, 1976). The Rinconada fault is primarily a right lateral strike slip fault (Petersen et al, 1996) with a smaller component of vertical movement. Right lateral movement of the Rinconada Fault zone in the area of Paraiso Springs is illustrated by folded Tertiary sediments west of the fault (Dibblee, 1976). Here the Tertiary Monterey formation is extensively folded. Axis of the folds is east west near the fault where they are truncated. The younger Tertiary sediments of the Pancho Rico and Paso Robles formations on the west side of the fault do not show the extensive east-west oriented folds of the Monterey Formation. Orientations for these younger sediments are roughly a northwest strike with an easterly dip. Vertical displacement in the area of Paraiso Hot Springs is illustrated by the juxtaposition Quaternary alluvium with Pre-Tertiary granitic rocks. Vertical displacement in the Sierra de Salinas may be as much as 10,000 feet (Dibblee, 1976). Slip rate for the Rinconada fault is estimated at 1.0mm/yr. Maximum magnitude is expected to be (M7.5) (Cao et al, 2003) with a recurrence interval of 1,764 years (Petersen et al, 1996).

San Gregorio – Palo Colorado Fault

Like the San Andreas fault, the San Gregorio fault has been divided into several different segments that are characterized by varying slip rates, earthquake intensities, and earthquake recurrence intervals. The San Gregorio (Sur Region) is the closest segment, located offshore about 24-km southwest of the site and is classified as a Type B fault (CDMG, 1998). The San Gregorio (Sur region) is a northwest trending right lateral strike slip fault about 80 km long (Petersen et al, 1996). The San Gregorio fault is part of the San Andreas fault system and is expressed as a complex series of en echelon right lateral strike slip faults (San Gregorio, Palo Colorado, San Simeon, & Hosgri faults) in the offshore and nearshore environments. The San Gregorio and related faults are several hundred kilometers long extending from the Santa Barbara Channel in the south, to its juncture with the San Andreas fault near Bolinas Bay in the north. Strong evidence supports that the San Gregorio fault (Sur region) has been active during Holocene time (Greene et al, 1973). Slip rate for the San Gregorio fault (Sur region) is estimated at 3.0mm/yr. Maximum magnitude is expected to be (M7.0) with a recurrence interval of 411 years (Petersen et al, 1996).

Monterey Bay-Tularcitos Fault

Located about 12.6-km northwest of the site, the Monterey Bay-Tularcitos fault zone is a complex series of northwest trending reverse, right lateral, and oblique faults which include the Tularcitos, Chupines, and Navy faults (Petersen et al, 1996). The Monterey Bay-Tularcitos fault zone lies within a fault bounded wedge of granitic basement rocks belonging to the Salinian block and is bounded on the west by the San Gregorio fault and on the east by the San Andreas fault (McKittrick, 1987). The Monterey Bay-Tularcitos fault is 84 km. long (Petersen et al, 1996) and extends from Paloma Creek in upper Carmel Valley (Clark et al, 1997) to the offshore environment within the Monterey Bay. Post Miocene vertical displacement of the Tularcitos fault is about 380 m and 3.2km to as much as 16 km of right lateral displacement (Clark et al, 1997). Offsets of Holocene age colluvial and fluvial terrace deposits indicates that the Tularcitos fault is active (Clark et al, 1997). The Monterey Bay fault is the offshore extension of the Tularcitos fault and comprises a discontinuous series of en echelon faults in the inner Monterey Bay between Monterey and Santa Cruz (Greene et al, 1973). The Monterey Bay fault zone displaces late Tertiary and Pleistocene sediments and in a few locations appears to cut Holocene sediments (Greene et al, 1973). Slip rate for the Monterey Bay-Tularcitos fault is estimated at 0.5mm/yr. Maximum magnitude is expected to be (M7.1) with a recurrence interval of 2,841 years (Petersen et al, 1996).

SITE GEOLOGY

Previous Work

Previous published mapping of the site and its vicinity has been performed by Durham, 1970, Dibblee, 1974, and Tinsley, 1975. Durham, 1970 mapped the site at a scale of 1:24,000. Durham maps the sloped upland areas of the site as Miocene Tierra Redonda Formation (Tt). The upper elevations of the northwest portion of the site are mapped as Pre Tertiary Basement complex (pt). The low lying valley portions of the site, Paraiso Springs Valley and Indian Valley are mapped as Pleistocene Fanglomerate (Qf). An unnamed fault is mapped by Durham trending northeast across the northwest corner of the site. The fault juxtaposes Tertiary Tierra Redonda Formation and Pre Tertiary Basement.

Dibblee, 1974 maps the site at a scale of 1:62,500. Dibblee maps the upland sloped areas of the site as Miocene Unnamed Red Beds (Trb). The upper elevations of the northwest corner of the site are mapped as Mesozoic or older Schist (ms). Also mapped in the northwest corner of the site is an unnamed fault juxtaposing schist and Unnamed Red Beds. The fault is buried by Quaternary Older Fan Gravels (Qog) at the northern central border of the site. South of the unnamed fault a large Quaternary landslide (Qls) is mapped. The low lying valley portions of the site, Paraiso Springs Valley and Indian Valley are mapped as Quaternary Older Alluvium (Qoa). In the center of the site Dibblee maps a small outcrop of Mesozoic basement rock (gdx). Dibblee also proposes the possible existence of subsidiary fractures related to the Rinconada fault under Paraiso Hot Springs (Dibblee, 1976). Dibblee proposes that these fractures may be the conduit by which rising hot water from the Rinconada Fault is sent westward to Paraiso Springs.

Tinsley, 1975 mapped the site at a scale of 1:62,500. Tinsley's mapping focused on Quaternary geology. Mapping of pre-quaternary geology is identical to Dibblee, 1974. Tinsley's mapping differs from Dibblee, 1974 in the mapping of the low-lying valley floor sediments. Tinsley maps the northern and southern borders of Paraiso Springs and Indian valleys as Pleistocene Chualar alluvial fan surfaces (Qch). The central portion of these valleys is mapped as Holocene Arroyo

Seco alluvial fan surface (Qas). The quaternary deposits in the upper elevations of the northwestern portion of the site are mapped as Pleistocene Placentia alluvial fan surfaces (Qp). Tinsley's map also shows a large Quaternary landslide in the southwestern area of the site that is congruent with Dibblee, 1974.

Geology for this report was mapped in the field on August 10, 11, and 12, 2004. Field mapping was done on a base topographic map at a scale of 1"=200'. During our investigation, mapping performed by Dibblee, 1974 was found to be accurate. Changes made by our investigation include mapping the Tertiary Unnamed Red Beds (Trb) as Tertiary Tierra Redonda Formation (Tt), and mapping many areas showing landslides and debris flows. As part of our geologic mapping we examined and interpreted four sets of stereo aerial photographs, taken in 1949, 1956, 1997, and 2000 covering the site and its vicinity. These photographs were scrutinized for site geology, terrain features characteristic of fault and landslide features. We also reviewed two water well logs drilled on the site in December 1976 & July 1992 (Appendix B). Based on the above referenced techniques, it is our opinion that the geology as mapped by Dibblee, 1974 is the most accurate published map. However, variations between the published mapping and the actual site geology exist, see Site Geologic Map, Sheet 1, and Geologic Cross Sections, Sheet 2, located in the map pocket at the back of this report. Description of the site geology is as follows:

Site Geologic Model

The right-lateral strike-slip Rinconada fault is the dominant and controlling structural feature of the western Salinas Valley (Figures 2 and 3) and is located approximately 1.5-km. east of the site. The Rinconada has an estimated slip rate of 1.0 mm/yr and a maximum magnitude earthquake of 7.5 (Cao et al, 2003). An unnamed fault likely related to the Rinconada is located on site. This fault trends northeast southwest across the northwestern corner of the site. According to Dibblee, 1974 this fault has shows no evidence of significant offset since the Miocene. Maximum magnitude, slip rate, and the recurrence interval are unknown for this fault. The structure of the Tertiary deposits on site is that of a northwest southeast trending openly folded anticline (See Sheet 2, Geologic Cross Sections). Quaternary deposits on site are relatively flat lying.

It has been proposed by Dibblee, 1976 that the hot water of Paraiso Springs may rise from the Rinconada fault in the east along fractures under the site. During our investigation no evidence for fracturing or faulting in the area of the hot springs was noted. However the subsurface structure of the unnamed fault is not known. This fault may provide the conduit for which the hot water is transferred. Minor slickensided fractures that are roughly parallel with the unknown fault were noted in the Tierra Redonda Formation (Sheet 1, note 4 and 5). The presence of fractures under the site cannot be denied nor confirmed with the data available. In the approximate center of the site an outcrop of granitic basement rock (Kgd) has been mapped (Sheet 1). This unit was also encountered at 10.5 feet below the ground surface in boring B-15, see Sheet 1 and appendix A. The presence of this basement rock at shallow depths could also contribute the geothermal gradient of the area and be responsible for the hot springs at the site. Description of the site stratigraphic section is as follows.

(Hf) Fill (Holocene): Man made fill deposits consisting of unconsolidated to semi-consolidated sand, silt, clay, and gravel. Fill deposits are found in many areas of the site where previous grading has occurred.

(Qyls) Landslide Deposits (Holocene): Recent landslide deposits, mostly occurring in the steeper slopes of the Tierra Redonda Formation (Tt). Deposits consist of unconsolidated sand silt and clay. These deposits are found flanking the site drainages where steep slopes are present.

(Qydf) Debris Flow (Holocene): Recent debris flow deposits, mostly occurring in the Tierra Redonda Formation (Tt). Deposits consist of unconsolidated sand silt and clay. These deposits are found flanking the site drainages where steep slopes are present.

(Qodf) Older Debris Flow (Holocene): Older debris flow deposits, mostly occurring in the Tierra Redonda Formation (Tt). Deposits consist of unconsolidated sand, silt, and clay. These deposits are found flanking the site drainages where steep slopes are present.

(Qal 1) Alluvium (Holocene): Unconsolidated to semiconsolidated sand, silt, gravel, and cobbles. Qal 1 is found in the upper reaches of Paraiso Springs and Indian Valleys and is coarser grained and younger than alluvial deposits to the east (Qal 2).

(Qal 2) Alluvium (Holocene): Unconsolidated sand, silt, and trace gravel. Qal 2 is found in the eastern portions of Paraiso Springs and Indian Valleys. Qal 2 is finer grained and older than alluvial deposits to the west

(Qols) Older Landslide (Pleistocene): Older landslide deposits consisting of unconsolidated to semi-consolidated boulders and cobbles supported by a sand and clay matrix. Clasts are of Sierra De Salinas Schist (ms) and granitic (Kgd) provenance. Located in the southwest corner of the site the slide buries Tierra Redonda deposits on the existing road

(Qoa) Older Alluvium (Pleistocene): older alluvial deposits consisting of unconsolidated to semi-consolidated cobbles and boulders. Older alluvial deposits are located in upper elevations of the northwest quarter of the site.

(Tt) Tierra Redonda Formation (Miocene): Marine sandstone, conglomerate and some mudstone. Deposits consist of slightly cemented fine to coarse grained, subangular to subrounded sand with subrounded to subangular fine to coarse gravels up to 6 inches in diameter. Sands and gravel clasts are composed of reworked granitic basement rock and Sierra De Salinas Schist. Deposits of Tierra Redonda are found flanking the site on the north and south sides.

(Kgd) Granitic Basement Rock (Cretaceous): Hornblende granodiorite with phenocrysts of feldspar. Kgd crops out in the central portion of the site.

(ms) Sierra De Salinas Schist (Pre-Cretaceous): Biotite schist of the Salinian Block. This unit is found in the upper elevations of the northwest corner of the site, west of the unnamed fault.

Landsliding

Landsliding on site consists of the debris avalanche and small rock slump type failures concentrated in the Tierra Redonda Formation (Tt), with one large debris slide off of the Sierra De Salinas Schist (ms). Slope failures are found on the steep northern slopes of Indian Valley, the steep southern slopes of Paraiso Springs Valley, and the northwestern slope of the western extent of Paraiso Springs Valley (Sheet 1).

Slope failures along the northern slope of Indian valley are of the debris avalanche (Qydf &Qodf) and small rock slump (Qyls) type, as classified by Varnes, 1978. The slides mapped were found during aerial photo review and during field mapping. Relative ages of slope failures were given based on geomorphic evidence. Young debris avalanche failures (Qydf) are expressed as elongate, shallow failures that expose unvegetated bedrock. Older debris flow avalanche failures (Qodf) are also expressed as elongate, shallow failures, but show regrowth of vegetation and softening of geomorphic features. Recent rock slump failures (Qyls) are expressed as lobate failures with rotated, intact blocks. These failures are shallow and lack regrowth of vegetation in the scarp areas.

Landsliding on the southern slopes of Paraiso Springs Valley consists entirely of the debris avalanche type (Qydf &Qodf). Relative ages of the slides were given using the criteria outlined above. Failures in this area are more extensive than those of Indian Valley in width and depth. The younger debris avalanches (Qydf) mapped are recent failures from March of 1995 (locality 1 and 6, Sheet 1). These events were rapid, and occurred on steep vegetated slopes after heavy rains for multiple days. Deposits on the valley floor were approximately 0.5 to 1.0 foot of mud and sand.

A large, old (Pleistocene) debris slide (Qols) is mapped in the southwestern portion of the site. This slide is approximately 800 feet wide and a minimum of 100 feet thick. The slide buries the Tierra Redonda Formation and the unnamed fault that crosses the northwestern corner of the site

(Sheet 1). The slide debris is made up of breciated gravels and cobbles in a sand and clay matrix. Lithology of the gravels and cobbles is granitic basement (Kgd) and Sierra De Salinas Schist (ms).

For purposes of zoning for our relative geologic hazard map, areas with identified landsliding were given the designation of zone 4 (High Geologic Hazard Potential). The steep slopes surrounding the areas of landsliding that do not show evidence of slope failure was also designated zone 4. These areas were classified as zone 4 due to similar earth materials and slope gradients.

SUBSURFACE CONDITIONS

A total of 29 exploratory borings were drilled on site. Subsurface constituents were fairly uniform and consistent with the published geologic mapping. Eleven different geologic units were encountered on site, all with varying subsurface conditions. To generalize, the site soil conditions of the upland areas are composed of bedrock and landslide deposits, while the valley areas are underlain by unconsolidated to semiconsolidated alluvium. The proposed development area is predominantly underlain by alluvium composed of unconsolidated to semiconsolidated sand, silt and clay with minor gravels and cobbles. Subsurface conditions are shown in the boring logs found in Appendix A at the back of this report.

GROUNDWATER

The Paraiso Springs Valley has a long history of ground water use. Native Californians were the first to utilize this resource, hence the name of Indian Valley given to the drainage to the north of Paraiso Springs Valley. The Spaniards and early Californians also took advantage of the groundwater resources of the area. In the southeast corner of Paraiso Springs Valley the Mission Soledad had its vineyard. The mission eventually sold the property. After the sale, the site was used for its hot spring mineral baths circa 1880's.

Numerous wells and hot springs are located on site. The Main Well is 104 feet deep and currently in use for domestic water, pumping at a rate of 20-30 gallons per minute (Geoconsultants, 2004). The Fluoride well is 640 feet deep and pumps at a rate of 200-300 gallons per minute, but is not used for domestic water (Geoconsultants, 2004). The Soda Springs well is currently being used for hot water. This well is 37 feet deep and produces 30-40 gallons per minute at +/- 115° F (Geosolutions, 1998).

The abundant groundwater resource of this valley was verified by our investigation. Of the 15 borings drilled in Paraiso Springs Valley, 10 borings encountered groundwater (See Table 1 & Sheet 1). Depths to ground water ranged from 11.0 to 55.0 feet below the ground surface. Depths to ground water and temperatures can be found in Table 1. Ground water in the area of the current hot springs was found to be 11.0 to 18.5 feet below the ground surface. The borings west of the current hot springs encounter ground water at greater depths the farther west they were drilled. Depth to ground water increases from 18.5 feet below the ground surface just west of the current hot springs in B-11 to 55.0 feet below the ground surface in B-19. All borings that encountered ground water were drilled in Quaternary alluvium, Qal 2. A slight to moderate sulfur odor was noted in some of the borings and was noted in the boring logs. Hydrophilic vegetation was also noted in the area east of the Great Lawn. The presence of this type of vegetation is indicative of springs and shallow ground water. Ground water was not found in borings outside of the Paraiso Springs Valley or in any other geologic unit.

TABLE 1

Ground Water Depth & Temperature

Boring	Initial Depth to Ground Water	Depth to Ground Water After 30m	Temperature F°
1	18.5´	6.5´	73.4
3	15.0´	19.0´	73.0
5	21.0´	11.5´	79.0
7	11.0´	8.0´	--
9	12.0´	7.0´	80.9
11	18.5´	18.2´	94.1
13	12.0´	9.7´	95.0
17	31.5´	41.3	95.7
19	55.0´	58.3´	95.0
23	14.0´	5.5´	73.0

Local groundwater levels can fluctuate over time depending on but not limited to factors such as seasonal rainfall, site elevation, groundwater withdrawal, and construction activities at neighboring sites. The influence of these time dependent factors could not be assessed at the time of our investigation.

SITE SOIL CLASSIFICATION

Because of the variability of geologic materials found on the site, multiple soil classifications could be applied. The ridges and slopes underlain by Tierra Redonda Formation (Tt) could be classified as soil type S_C, Very Dense Soil and Soft Rock. Alluvium in Indian Valley and alluvium west of locality 1 (Sheet 1) could be classified as S_C / $S_{D,}$ Very Dense Soil and Soft Rock/Stiff Soil Profile. In the alluvium east of locality 1 high groundwater conditions and low blow counts were encountered. These soils are given soil type $S_{E,}$ Soft Soil Profile. A majority of the development of the site is proposed to occur in the area east of locality in soil type S_E. For this reason we are designating the soil type for the site as S_E as defined by the guidelines in the

2001 edition of the California Building Code (CBC). As per Chapter 16, Section 1636.2 The Soft Soil Profile (S_E) is classified as having an average shear wave velocity of less than 180 m/sec.

GEOLOGIC AND SOIL ENGINEERING CONCLUSIONS

Seismic Hazards: The site is located in the seismically active Monterey Bay region of the Coast Ranges Geomorphic Province (Figure 5). The closest earthquake fault zone is the San Andreas fault, located 30-km to the northeast. The California Division of Mines and Geology has classified the San Andreas fault (Creeping segment) as a Type A Fault for purposes of the 2001 CBC (CDMG, 1998). The San Andreas fault Creeping segment can expect a (M6.2) earthquake with an approximate 61 year recurrence interval (Cao et al, 2003). Stronger earthquakes could be experienced at the site similar to the 1906 event with a maximum magnitude of (M7.9) with a recurrence interval of 210 years (Petersen et al, 1996).

Surface Fault Rupture: The site is not located within an Earthquake Fault Zone as established in accordance with the Alquist-Priolo Earthquake Fault Zoning Act of 1972. However a fault of unknown activity has been mapped on site. The northwestern portion of the site where the fault is mapped has been designated Zone 4F for our Relative Geologic Hazard Map (Sheet 3). This area has moderate potential for surface fault rupture. The remaining portion of site has low potential for surface fault rupture.

Historical Earthquakes: During recent historic times moderate to large earthquakes have caused significant damage to man made structures in the greater Monterey Bay area. These include the following:

1857 San Andreas Fault: A large quake occurred on the San Andreas fault, rupturing from Parkfield south to Wrightwood, on January 9, 1857. The quake had an estimated magnitude of 7.8. Very severe shocks were felt in Sacramento and a cabin was knocked down in the Cholame area (Rosenberg, 2001).

1881 Parkfield: On February 2, 1881 a 5.6 magnitude quake occurred in the Parkfield area knocking down several adobe structures and chimneys. Springs and cracks were also noted in the area of the quake (Rosenberg, 2001).

1901 Parkfield: A magnitude 5.8 struck the Parkfield area on March 2, 1901. Again many chimneys were damaged and cracks in the ground were noted. A small tsunami also occurred in the Monterey Bay. (Rosenberg, 2001)

1906 California: The 1906 (M_w~8.0) "San Francisco earthquake", which ruptured a portion of the active San Andreas fault from approximately San Juan Bautista to Cape Mendocino, caused severe damage in parts of the Monterey-San Francisco Bay area and throughout California. The earthquake occurred on April 18, 1906 and caused severe ground shaking, ground settlement, liquefaction, and structural damage to buildings in Monterey, Santa Cruz, and San Benito Counties (Lawson, 1908). The most significant earthquake effects in the area of the site and vicinity were the sinking of the Salinas River bed in the areas of King City and San Ardo. (Rosenberg, 2001). Ground water flow changes were also common. At Paraiso Springs the temperature and flow of water had been decreasing for "some time" before the quake (Lawson, 1908). After the quake the temperature and flow of the springs returned too its previous values (Lawson, 1908).

1922 Parkfield: The March 10, 1922 earthquake that struck the Parkfield area was a magnitude 6.1. It caused ground cracks six to twelve inches in width and a quarter-mile long in the Chalome Valley (Rosenberg, 2001). Chimneys were knocked down and some housed suffered structural damage. An oil pipeline was also damaged in the area.

1926 Monterey Bay Doublet: On October 22, 1926 two magnitude 6.1 earthquakes an hour apart occurred in southern Monterey Bay. Numerous buildings experienced damage and cracking on the Monterey Peninsula and in Salinas. It is postulated that the earthquakes occurred on either the San Gregorio fault or Monterey Bay fault zone (Rosenberg, 2001).

1934 Parkfield: A magnitude 6.1 earthquake again struck the Parkfield area on June 7, 1934. Again this quake caused fracturing of the ground surface and broke the oil pipeline in the area. Chimneys and houses were also damaged in the area (Rosenberg, 2001).

1938 Stonewall Canyon: On September 27, 1938 a magnitude 5.0 quake occurred in the Stonewall Canyon area northeast of Soledad. Details of the damage caused by this quake are unknown. This is the closest quake of magnitude 5.0 or greater to the site at approximately 17-km away.

1989 Loma Prieta: The October 17, 1989 (M_w 7.1) Loma Prieta earthquake, which is believed to have occurred on an oblique-slip blind thrust closely associated with the San Andreas fault, also caused significant damage in the San Francisco and Monterey Bay areas. It was the largest earthquake to strike this region of California since the California earthquake of 1906. The effects of this earthquake was felt over an area of 400,000 square miles and resulted in 74 deaths, 3,757 injuries, 12,000 homeless, and over $6 billion in property damage (Plafker & Galloway, 1989). In Monterey County 19 homes were destroyed, 341 homes damaged, two deaths and 14 people injured, and causing approximately $118 million in damages (Rosenberg, 2001). The southern Salinas Valley suffered little damage as a result of this quake. The liquefaction experienced in the 1906 quake was absent during this event. The explanation given by Rosenberg, 2001 for the differences in liquefaction occurrence is differences in ground water table at the time of rupture. Groundwater was likely higher in 1906 as they had a wet winter, and the 1989 quake occurred after several years of drought.

As part of our historical earthquake research, we performed a database search of the Northern California Earthquake Data Center catalog for earthquakes with magnitudes greater than 5.0 within an approximate 100km radius of the site for the years between 1910 to 2004. The database research indicated a total of 87 events within our search parameters. The December 22, 2003 Paso Robles earthquake and the September 28, 2004 Parkfield earthquake were within the search radius. The closest earthquake was the Stonewall Canyon earthquake of 1938.

Ground Shaking: The 1906 (M_w~8.0) “San Francisco earthquake”, which ruptured a portion of the active San Andreas fault from approximately San Juan Bautista to Cape Mendocino, caused severe damage in parts of the Monterey-San Francisco Bay area. Its epicenter was located directly west of the Golden Gate, approximately 183 kilometers northwest of the site. The earthquake occurred on April 18, 1906 and caused severe ground shaking and structural damage to buildings in Monterey and San Benito Counties (Lawson, 1908). The 1989 (M_w 7.1) Loma Prieta earthquake, which is believed to have occurred on an oblique-slip blind thrust closely associated with the San Andreas fault, also caused significant damage in Monterey County. The epicenter of this event was located in the Forest of Nicene Marks State Park, approximately 80 kilometers northwest of the site. Strong ground shaking associated with major earthquakes along the San Andreas and related faults will undoubtedly occur at the site in the future. The State of California estimates the peak ground acceleration with a 10 percent probability of being exceeded in a 50-year period in the vicinity of the site to be >0.35 to 0.45g (Petersen et al, 1996)

<u>Seismic Design Parameters:</u> As previously stated we have classified the soil profile as Soft Soil Profile (S_E) as defined in the guidelines in the 2001 CBC, Section 1636.2 (average shear wave velocity for the upper 30 meters is less than 180 m./sec.). We have determined the appropriate seismic coefficients to be used for the design of the structure according to the 2001 CBC.

TABLE 2
Near Source Factors & Seismic Coefficients

Seismic Source	**Fault Type**	**Distance**	N_a	N_v	C_a	C_V
Rinconada Fault	B	1.5 km E	1.3	1.6	0.47	1.54

<u>Liquefaction, Lateral Spreading, and Dynamic Compaction:</u> Liquefaction is the transformation of soil from a solid to a liquid state as a consequence of increased pore-water pressures, usually in response to strong ground shaking, such as those generated during a seismic event (earthquake). Liquefaction is most commonly associated with Holocene age deposits where the groundwater is less than 30 feet below the surface and the anticipated peak ground acceleration (PGA) having a 10% probability of being exceeded in 50 years is greater than 0.2g (Arulmoli et. al., 1999). Liquefaction most often occurs in Holocene age loose saturated silts, and saturated poorly graded fine-grained sands. However, some cohesive clay soils can be subject to strength loss even under relatively minor strains. All but two borings, B-17 and B-19, that encountered ground water meet the above stated criteria of a PGA higher than a 0.2 and ground water at less than 30 feet below the ground surface. Data collected from exploratory borings were used to evaluate the liquefaction potential of the site using the “Liquefy 2”computer program developed by Thomas F. Blake. Each boring which encountered ground water, Borings 1, 3, 5, 7, 9, 11, 13, 17, 23, was evaluated using a peak ground acceleration of 0.47g, and a maximum magnitude earthquake of 7.5. Of the nine borings evaluated, only boring B-23 has a factor of safety greater than 1.0 for the entire depth of the boring. Therefore it is our opinion that the potential for liquefaction at the site is high. As a result we are recommending a supplemental liquefaction study be conducted in the areas where high ground water was encountered (Zone 3L) to quantify the hazards associated with soil settlement due to liquefaction.

Dynamic compaction occurs when loose, unsaturated soils densify in response to ground shaking during a seismic event. Because loose soils were encountered on the site, it is our opinion that the potential for dynamic compaction is high in areas designated as Zone 3L.

Ridge-Top Shattering: Ridge-top shattering was well documented after the 1971 San Fernando earthquake and also occurred during the 1989 Loma Prieta earthquake in the Santa Cruz Mountains. The phenomenon occurs most commonly on the crests of sharp ridges, where seismic shaking energy is concentrated as in the chimney of a building. Shattering can effect both soil and the underlying bedrock and gives the appearance of plowed ground (Barrows, 1975; Kahle, 1975). The site lacks sharp ridgelines typical of ridge-top shattering failures, therefor the potential for ridge-top shattering is considered to be low.

Landsliding and Slope Stability: The steep slopes underlain by the Tierra Redonda Formation that flank Paraiso Springs Valley and Indian Valley are very prone to slope failure. Numerous debris avalanches and debris slides of varying ages are present on these slopes. All steep slopes of the Tierra Redonda have been given the designation Zone 4D or 4S, major geologic hazard potential for debris flow and sliding, on our Relative Geologic Hazards Map (Sheet 3).

Flood Hazards: According to the National Flood Insurance Program map Panel Number 060195 0350 D (FEMA, 1984) the site is not located within a flood zone. However flooding of the site near the current hot spring did occur in March of 1995. This flood was the result of channeling the drainage into a culvert of insufficient diameter. Brush, rocks and other stream debris clogged the culvert and caused the drainage to overflow (Sheet 1, Locality 2). The flood that resulted caused significant damage to the road and pools below. To help prevent future incidences like the 1995 flood, on site stream channels may need to be enlarged. On site stream channels will also need to be cleared and maintained. Culverts and bridges should be designed to not cause restrictions to flow in the stream channel.

Erosion: The site soils and earth materials are erodible. Stringent erosion control measures should be implemented to provide surficial stability of existing and proposed graded cut/fill slopes.

Soil Expansion: Expansive soils experience volumetric changes with changes in moisture content, swelling with increases in moisture content and shrinking with decreasing moisture content. These volumetric changes that the soil undergoes in this cyclic pattern can cause distress resulting in damage to concrete slabs and foundations. The Atterberg limits tests performed on a near surface soil samples resulted in plasticity indexes of 9 to 23. These values indicate that the near surface soil (upper 5-feet) typically has a low expansion potential. No special measures are required to mitigate soil expansion.

GEOLOGIC CONSTRAINTS & PROPOSED DEVELOPMENT

One of the purposes of this report was to evaluate the site geologic constraints and develop a relative geologic hazard assessment, within the framework of the proposed development. For the purposes of land use planning, the term geologic hazard indicates a naturally occurring surface or subsurface constraint caused by existing site geologic conditions. Potential risks can usually be assessed and mitigated to an acceptable level by analyzing these constraints.

Preparing a relative geologic hazards map involves interpreting site topography, soil and rock type, groundwater conditions and geologic structure. In order to provide a useful framework for project planners, we have prepared a map depicting the relative geologic hazards (Sheet 3). This map is a result of the interpretation and compellation of our findings from site geologic mapping, subsurface exploration, aerial photographic review, and literature review.

The relative geologic hazards map (Sheet 3) has been divided into for zones of relative geologic risk from low (Area 1) to high (Area 4). These zones have been further subdivided into areas of specific hazards related to potential risk for faulting (F), liquefaction (L), debris flow (D) and landsliding (S). The project planners must understand that the geologic hazards map should be utilized as a guideline for planning purposes, and ***is not*** a substitute for the recommended design level site specific investigations. While solid or dashed lines delineate the hazard areas, the actual boundaries between the hazard areas are gradational. The following presents an overview of the relative geologic hazards for the areas of proposed site development, and their potential mitigative measures.

Area 1 – Low Geologic Hazard Potential

Proposed development within this area includes; the Estate Lots, northern portion of the Paraiso Institute, the majority of the Hillside Village Condominiums, western portion of the Casitas area, northern portion of the Teahouse Complex and western portion of the Sports Center. No special mitigative grading or foundation measures are required for site development in this area. Building foundations may consist of either conventional cast-in-place footings or pier and grade beam foundations depending on slope gradients. A site-specific design level soil engineering investigation is recommended once the actual building locations and preliminary grading plans have been completed. This hazard area associated with an “ordinary level of risk”. (See Appendix D)

Area 2D – Minor Geologic Hazard Potential – Debris Flow

Proposed development within this area includes the western portion of the Sports Center. Mitigation measures to protect development in this area should include adequate design of site storm drain facilities for post-development runoff, and debris flow walls and basins in the upstream drainages. Building foundations may consist of conventional cast-in-place footings. A site-specific design level engineering geologic and soil engineering investigation is recommended once the actual building locations and preliminary grading plans have been completed. This hazard area associated with an “ordinary level of risk”. (See Appendix D)

Area 2S – Minor Geologic Hazard Potential - Landslide

Proposed development within this area includes the northwestern portion of the Hillside Village Condominiums. Mitigation measures to protect development in this area should include appropriate grading techniques & methodology and adequate design of site drainage facilities for post-development runoff. Building foundations should consist drilled pier and grade beam foundations. A site-specific design level engineering geologic and soil engineering investigation is recommended once the actual building locations and preliminary grading plans have been completed. This hazard area associated with an “ordinary level of risk”. (See Appendix D)

Area 3L – Moderate Geologic Hazard – Liquefaction Potential

Proposed development within this area includes the Biolarium, Living Machine, Nursery, Winery, Day Spa, Hamlet Town Square, Hotel, Conference Center and eastern portion of the Casitas. Mitigation measures to protect development in this area could include structural strengthening of buildings to resist predicted ground settlements (if small), placement of a sufficiently thick layer of engineered fill to resist predicted ground settlement, utilization of post tension or mat slab foundations, or a combination of the above noted measures. A site-specific supplemental liquefaction investigation prepared in accordance with CDMG Special Publication 117 should be performed prior to the completion of preliminary grading plans. This hazard area associated with an “ordinary level of risk”. (See Appendix D)

Area 3D – Moderate Geologic Hazard – Debris Flow Potential

Proposed development within this area includes the southern portion of the Casitas and Teahouse areas. Mitigation measures to protect development in this area should include appropriate grading techniques & methodology and adequate design of site drainage facilities for post-development runoff. Debris flow basins and diversion structures are recommended to protect future development from debris flow source areas. Building foundations may consist of conventional cast-in-place footings. A site-specific design level engineering geologic and soil engineering investigation is recommended once the actual building locations and preliminary grading plans have been completed. This hazard area associated with an “ordinary level of risk”. (See Appendix D)

Area 3S – Moderate Geologic Hazard – Landslide Potential

Proposed development within this area includes the southwestern portion of the Hillside Village Condominiums. Mitigation measures to protect development in this area should include appropriate grading techniques & methodology and adequate design of site drainage facilities for post-development runoff. Building foundations should consist drilled pier and grade beam foundations. A site-specific design level engineering geologic and soil engineering investigation is recommended once the actual building locations and preliminary grading plans have been completed. This hazard area associated with an “ordinary level of risk”. (See Appendix D)

Area 3DS – Moderate Geologic Hazard – Debris Flow and Landslide Potential

Proposed development within this area includes the north-central portion of the Hillside Village Condominiums. Mitigation measures to protect development in this area should include appropriate grading techniques & methodology and adequate design of site drainage facilities for post-development runoff. Debris flow basins and diversion structures are recommended to protect future development from debris flow source areas. Building foundations should consist of drilled pier and grade beam foundations. A site-specific design level engineering geologic and soil engineering investigation is recommended once the actual building locations and preliminary grading plans have been completed. This hazard area associated with an “ordinary level of risk”. (See Appendix D)

RECOMMENDATIONS

The following recommendations are drawn from the data acquired and evaluated during this investigation for the proposed project.

Geologic

In our opinion, the site is suitable for the proposed development provided that the recommendations contained herein are strictly adhered to and implemented in the design and construction. These recommendations have been prepared assuming that Landset Engineers, Inc. will be commissioned to review proposed site development and grading plans prior to construction and provide design level engineering geologic recommendations. Soil and groundwater conditions can deviate from the conditions encountered in the exploratory borings, if significant variations in the subsurface conditions are encountered during construction, it may be necessary for Landset Engineers, Inc. to review the recommendations presented herein, and recommend adjustments as necessary.

1. An additional site-specific supplemental liquefaction study should be performed for proposed development located in Zone 3L. The supplemental liquefaction study should be performed in accordance with the guidelines contained within the California Division of Mines & Geology Special Publication 117, as adopted by the State Mining and Geology Board in accordance with the State of California Seismic Hazards Mapping Act of 1990. It is recommended that the supplemental liquefaction study should include cone penetrometer test (CPT) borings and additional laboratory testing in order to more accurately characterize the site subsurface conditions and estimate potential ground settlements as a result of liquefaction.

2. Prior to construction, the location of proposed areas to be developed including building envelopes, roadways, drainage, utilities, and leachfield improvements should be reviewed by the project geologist for proposed development located in geologic hazard zones 2, 3 and 4.

The purpose of this review is to provided additional engineering geologic design level criteria verify setbacks from slopes, landslides and other identified geologic hazards.

3. Structures designed for human occupancy shall be designed according to the current edition of the CBC. Structures should be designed for a mean peak horizontal ground acceleration of 0.47g.

4. The project geologist **must** review and approve all project grading plans prior to submittal to the governing jurisdiction. The purpose of this review is to examine the slopes for overall stability and to provide additional recommendations if site conditions differ from those identified during the course of this investigation.

Soil Engineering

In our opinion, the site is suitable from a soil engineering standpoint for the proposed development provided that the recommendations contained herein are implemented in the design and construction. The following preliminary recommendations are presented as guidelines to be used by project planners and designers for the soil engineering aspects of the project design and construction. These recommendations have been prepared assuming that Landset Engineers, Inc. will be commissioned to perform additional design level investigations, review proposed grading and foundation plans before construction, and to observe, test and advise during earthwork and foundation construction. Soil and groundwater conditions can deviate from the conditions encountered at the boring locations. If significant variations in the subsurface conditions are encountered during construction, it may be necessary for Landset Engineers, Inc. to review the recommendations presented herein, and recommend adjustments as necessary.

Site Preparation and Grading

1. The soil engineer should be notified **at least ten (10) working days prior to any site clearing or grading** so that the work in the field can be coordinated with the grading contractor, and arrangements for testing and observation services can be made. The recommendations contained in this report are based on the assumption that Landset Engineers, Inc. will perform the required testing and observation services during grading and construction. It is the owner's responsibility to make the necessary arrangements for these required services.

2. Prior to grading, construction areas should be cleared of obstructions, buried structures & utilities, and other deleterious materials. Site clearing should be observed by a field representative of Landset Engineers, Inc. Voids created by removal of material as described above should be called to the attention of the soil engineer. No fill should be placed unless a representative of this firm has observed the underlying soil.

3. Following site clearing, the upper 1 to 4-feet of native soil should be overexcavated from the building areas. The actual depth of subexcavation should be determined by additional design level soil engineering investigations. Building areas are defined as the soils within and extending a minimum of 5 feet beyond the foundation perimeters and structural fill areas.

4. The soils exposed by overexcavation should be scarified 8 inches; moisture conditioned to above optimum moisture content, and compacted to at least 90% of maximum dry density. Where referenced in this report, percent relative compaction and optimum moisture content shall be based on ASTM test D1557-91. Areas to receive structural fill outside the building pad should be scarified and recompacted in a similar manner.

5. In order to limit the potential for differential settlement of conventional footings, foundations should not be supported on both fill and cut. Therefore, we recommend that the cut side of the building area should be overexcavated (undercut). The proposed grading within the building area should be designed so that no more than 5 feet of differential fill thickness exists below foundations. The portion of the building foundations bearing on cut should be undercut at least 3 feet below the proposed building pad so that the entire foundation is bearing on a uniform layer of compacted fill. Deeper overexcavation may be necessary in order to satisfy the differential fill thickness recommendations. The use of post-tensioned slabs may reduce or eliminate the need to undercut cut/fill pads

6. If structural fill is to be placed on slopes steeper than 6:1 (horizontal to vertical), keyways should be established at the toe of the proposed fill slopes. The keyways should have minimum widths of 10-feet and should be sloped approximately 2% back into the hillsides. The keyways and subsequent upslope benches should penetrate into sufficiently stable material at determined by the soil engineer at the time of grading.

7. If structural fill is to be placed on slopes steeper than 10:1, the slopes should be benched. The benches should have a minimum width of 10-feet and should be sloped approximately 2% back into the hillsides. The soil engineer will determine the depth, scarification, and recompaction of the bench bottoms at the time of grading.

8. If fill over cut slopes are to be constructed, keyways should be established at the cut/fill daylight lines. The keyways should have minimum widths of 10-feet and should be sloped approximately 2% back into the hillsides. The keyways and subsequent upslope benches should penetrate into sufficiently stable material as determined by the soil engineer at the time of grading.

9. The soil engineer should also observe keyways and benches to assess the need for subsurface drains (subdrains). Subdrains in other areas may also be recommended depending on the grading plan and site conditions observed at the time of grading.

10. Fill slopes should be constructed at a maximum finished slope inclination of 2:1 (horizontal to vertical). Fill slopes should be overfilled and trimmed back to competent material. Further compaction of exposed fill slope faces using sheepsfoot rollers or tracked equipment may be recommended by the soil engineer. Cut slopes should be constructed at an inclination of 2:1.

11. Fill, material should be placed in thin lifts, moisture conditioned to a level above optimum moisture content, and compacted to a minimum of 90 percent of maximum dry density. Prior to compaction, the soil should be cleaned of any rock, debris, and irreducible material larger than 3-inches in diameter.

12. Fill material should consist of non-expansive Select Structural Fill. Select Structural Fill is defined herein as a native or import fill material which, when properly compacted, will support foundations, pavements, and other fills without detrimental settlement or expansion. Select Structural Fill is specified as follows:

Select Structural Fill

- ❈ Clean native soil may be utilized, but import fill shall have a Plasticity Index of less than 12;
- ❈ Be free of debris, vegetation, and other deleterious material;
- ❈ Have a maximum particle size of 3-inches in diameter;
- ❈ Contain no more than 15% by weight of rocks larger than 21/2-inches in diameter;
- ❈ Have sufficient binder to allow foundation and unshored excavation stand without caving;
- ❈ Prior to delivery to the site, a representative sample of proposed import should be provided to Landset Engineers, Inc. for laboratory evaluation.

13. In areas to be paved, the upper 12-inches of subgrade soils and all aggregate base should be compacted to a minimum of 95 percent of maximum dry density. Aggregate base and subgrade should be firm and unyielding when proofrolled by heavy rubber-tired equipment prior to paving.

Foundations

14. The buildings may be supported by conventional continuous and spread (pad) footings, drilled pier & grade beam, or by post-tensioned slab foundations (see Geologic Constraints and Proposed Development section of this report for recommended foundation type).

Conventional Footings

15. The buildings may be supported by conventional continuous and spread (pad) footings supported on recompacted soil. Footings should have minimum depths of 12-inches below lowest adjacent grade for single story structures, and 18-inches below lowest adjacent grade for two story structures, and 24-inches below lowest adjacent grade for three story structures. For the above conditions, the footings for a proposed structure may be designed for an allowable bearing pressure range of 1,000 to 3,000ft^2 for dead plus live loads. Footings should be reinforced as directed by the architect/structural engineer.

16. Post construction total and differential settlements of foundations are expected to be about ½ to 1½-inch from static loading. Estimated foundation movements due to seismically induced settlement as a result of earthquakes could be higher.

17. Footing excavations should be observed by a representative of this firm prior to placement of formwork or reinforcement. Concrete should be placed only in foundation excavations that have been kept moist, and contain no loose or soft soil debris.

18. Footings located adjacent to other footings or utility trenches should have their bearing surfaces founded below an imaginary 1:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trenches.

Pier & Grade Beam Foundations

19. Drilled friction and/or end bearing pier and grade beam foundations should penetrate through any engineered fill and/or topsoil and bear entirely into the dense native bedrock materials.

20. Foundation piers should be 12 to 18-inches in diameter and should be spaced apart at least 3 pier diameters, center to center. These cast-in-place concrete piers should be reinforced as directed by the project architect/structural engineer.

21. The piers should penetrate through any fill or topsoil, and a minimum of 5 feet into bedrock material as verified by a representative of this firm at the time of drilling. Overall piers depths should be at least 8 to 10-feet below lowest adjacent grade.

22. For the above conditions, the piers for a proposed structure may be designed for an allowable skin-friction range of 250 to 500 psf. for pier lengths in bedrock for dead plus live loading. This value may be increased by one-third when considering temporary additional short-term wind or seismic loading. The support from end bearing of the piers should be neglected. Due to possible disturbance during drilling, skin friction on the upper 2-feet of the piers should be discounted in the calculations. Piers should be

structurally connected to grade beams designed to transfer imposed loads to the foundation piers.

23. For calculating resistance to lateral loading, a passive resistance equal to an equivalent fluid weight range of 250 to 350 pcf. can be used (ultimate value). For pier foundations, this lateral resistance can be used over two times the cross sectional area of the pier. Only competent bedrock and engineered structural fill may be utilized in calculating lateral passive resistance. Additionally, the upper 2-feet of the pier should be ignored in providing lateral passive resistance.

24. Post construction total and differential settlements of foundations are expected to be about ½-inch from static loading. Estimated foundation movements due to seismically induced settlement as a result of earthquakes could be higher.

25. Perimeter foundation piers and piers adjacent to structural concrete slabs-on-grade should be laterally restrained by concrete grade beams penetrating a minimum of 12-inches below lowest adjacent grade. Grade beams between interior piers are not considered necessary. Grade beams should be reinforced as directed by the project architect/structural engineer.

Post-Tensioned Slab Foundations

26. Post-tensioned slabs may be utilized to resist differential settlement of the fill material and/or potentially liquefiable soils. Post-tensioned slabs should be designed in accordance with the 2001 edition of the California Building Code and the latest design recommendations by the Post-Tensioning Institute utilizing the following design criteria:

27. For the above conditions, the post-tensioned slabs may be designed for an allowable bearing pressure range of 1,000 to 3,000 pounds per square foot for dead plus live loads. A qualified structural engineer should design post-tensioned slabs.

28. A minimum of 4 inches of clean sand should be provided beneath the post-tensioned slabs. The building pad subgrade should be pre-moistened to a level at or slightly above optimum moisture content prior to the placement of the clean sand cushion. Clean sand is defined as a sand (ASTM D 2488-93) of which less than 3 percent passes the No. 200 sieve.

29. To minimize floor dampness, such as where moisture sensitive floorings will be present, a membrane vapor barrier should be placed at the midsection of the clean sand cushion. The membrane vapor barrier should be a minimum 10 mil in thickness, and care should be taken to properly lap and seal the vapor barrier, particularly around utilities.

30. To limit the potential for subsurface moisture to enter the underlying sand cushion, the perimeters of the post-tensioned slabs should be thickened to penetrate below the bottom of the sand cushion layer.

31. Post-tensioned slabs should be constructed and maintained in accordance with the latest procedures as specified by the Post-Tensioning Institute. Plumbing through the slabs, utility connections, exterior flatwork, and drainage systems should be designed to accommodate the specified differential settlement conditions as determined by additional design level investigations.

Conventional Slabs-on-Grade and Exterior Flatwork

32. For buildings utilizing conventional footings, interior slabs-on-grade should have a thickness of 4 to 6-inches. It should be noted that the project structural engineer might require thicker slab sections to provide the necessary support for the anticipated structural loads. Conventional concrete slabs-on-grade should be reinforced with steel as specified by the structural engineer.

33. To minimize floor dampness, such as where moisture sensitive floorings will be present, a section of capillary break material at least 4-inches thick covered with a membrane

vapor barrier should be placed between the floor slab and the compacted soil subgrade. The capillary break should consist of a clean, free draining material such as ½ to ¾-inch drainrock with not more than 10 percent of the material passing a No. 4 sieve. The drainrock should be free of sharp edges that might damage the membrane vapor barrier. The membrane vapor barrier should be a minimum 10 mil in thickness, and care should be taken to properly lap and seal the vapor barrier, particularly around utilities. The sand cushion should be lightly moistened immediately prior to concrete placement.

34. Exterior concrete flatwork such as driveways, patios and sidewalks should be designed to act independently of building foundations. Exterior flatwork should be constructed on compacted soil subgrade moisture conditioned to over optimum moisture content. Reinforcement and joint spacing should be at the direction of the architect/structural engineer.

Utility Trenches

35. On-site soils should be properly shored and braced during construction to prevent sloughing and caving of trench sidewalls. The contractor should comply with the Cal/OSHA and local safety requirements and codes dealing with excavations and trenches.

36. A select non-corrosive, granular, material should be used as bedding and shading immediately around underground utility pipes and conduits. Native soils may be used for trench backfill above the select material.

37. Trench backfill in landscaped or unimproved areas should be compacted to a minimum of 85 percent of maximum dry density. Trench backfill beneath asphalt and concrete pavements should be compacted to a minimum of 95 percent of maximum dry density. Trench backfill in other areas should be compacted to a minimum of 90 percent of maximum dry density.

38. The bottoms of utility trenches that are parallel to foundations should not extend below an imaginary plane sloping downward at a 1:1 (horizontal to vertical) angle from the bottom outside edges of foundations.

Site Drainage

39. The site soils are highly erodible and a drainage & erosion control plan is essential to the project. Fluctuations of moisture contents are a major consideration, both before and after construction. Site runoff will be substantially increased due to the large paved and surfaced areas. A comprehensive drainage & erosion control plan is essential to the long-term sustainability of the project.

40. Surface drainage should provide for positive drainage so that runoff is not permitted to pond adjacent to foundations, concrete slabs-on-grade, and pavements. Surface drainage should be directed away from site improvements at a minimum 2 percent grade for a minimum distance of 5-feet. Surface drainage facilities should be armored or hard-scaped to limit erosion potential. If this is not practicable due to the terrain or other site features, swales with improved surfaces should be provided to divert drainage away from improvements.

41. Roof gutters should be utilized around the building eaves. Roof gutters should be connected to downspouts, which in turn should be connected to pipes leading to the site storm drain system. Runoff from downspouts, planter drains and other improvements should discharge in a non-erosive manner away from site improvements in accordance with the requirements of the governing agencies.

42. The migration of water or spread of root systems below foundations, slabs, or pavements may cause differential movement and subsequent damage. Landscaping runoff collection facilities should be incorporated in the project design.

43. Cut-off drainage swales should be constructed at the top of all cut and fill slopes. These drainage swales should be of adequate size to collect surface runoff and flow to an approved point of discharge in a non-erosive manner. Proper drainage and re-vegetation of graded slopes is essential to ensure stability.

QUALITY CONTROL

The conclusions and recommendations contained in this geologic report and soil engineering feasibility investigation are preliminary in nature. We recommend that Landset Engineers, Inc. be retained to review preliminary plans once they are available. Additionally, we should provide final engineering geologic, grading, foundation, and retaining wall design criteria based on a site specific design level investigations once the proposed site usage, construction type, locations and anticipated loads are known. These services are beyond the scope of this investigation.

The following items should be performed, reviewed, tested, or observed by this firm:

- Design level engineering geologic and soil engineering investigation(s)
- Final grading and foundation plans
- Site stripping and clearing
- Overexcavation
- Scarification and recompaction
- Fill placement and compaction
- Foundation excavations
- Underground utility backfill and compaction.
- Compaction of subgrade and Class 2 A.B. in areas to be paved.

If Landset Engineers, Inc. is not retained to provide design level engineering geologic services, design level soil engineering services, or construction observation and compaction testing, we shall not be responsible for the interpretation of the information by others or any consequences arising therefrom.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The preliminary recommendations contained in this report are based, in part, on certain plans, information, and data that has been provided to us. Any changes in those plans, information, and data will render our recommendations invalid unless we are commissioned to review the changes and to make any necessary modifications and/or additions to our recommendations. The criteria in this report are considered preliminary until such time as they are modified or verified by the engineering geologist or soil engineer in the field during construction. No representation, warranty, or guarantee is either expressed or implied. This report is intended for the exclusive use by the client and the client's architect/engineer. Application beyond the stated intent is strictly at the user's risk.

The recommendations of this report are based upon the assumption that the soil/rock conditions do not deviate from those disclosed in the borings or geologic maps. If any variations or undesirable conditions are encountered during construction, Landset Engineers, Inc. should be notified so that supplemental recommendations can be given.

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 called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to ensure that the Contractor and Subcontractors carry out such recommendations. The conclusions and recommendations contained herein are professional opinions derived in accordance with current and local standards of professional practice.

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 due to natural processes or to 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 in part, by changes outside of our control. Therefore, this report should not be relied upon after a period of three years, without being reviewed by Landset Engineers, Inc. from the date of issuance of this report.

This report does not address issues in the domain of the contractor such as, but not limited to, loss of volume due to stripping of the site, shrinkage of fill soils during compaction, excavatability, and construction methods. The scope of our services did not include any determination or evaluation of soil corrosion potential, environmental assessment of wetlands, radioisotopes, hydrocarbons, hazardous or toxic materials, or other chemical properties in the soil, surface water, groundwater or air, on or below or around the site.

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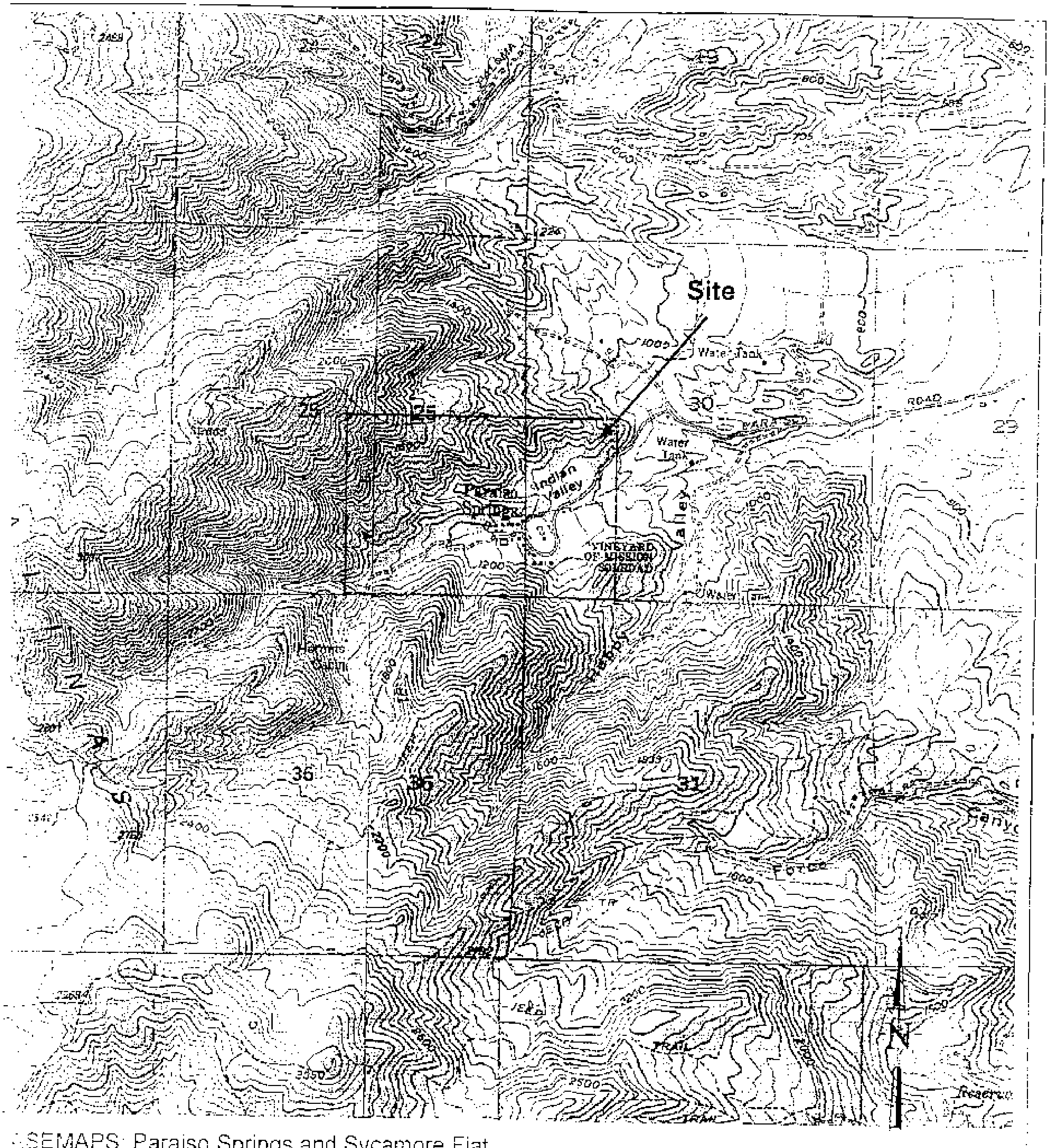
FIGURES



:SEMAPS: Paraiso Springs and Sycamore Flat
U.S.G.S Topographic Quadrangle Map

0 2000 ft

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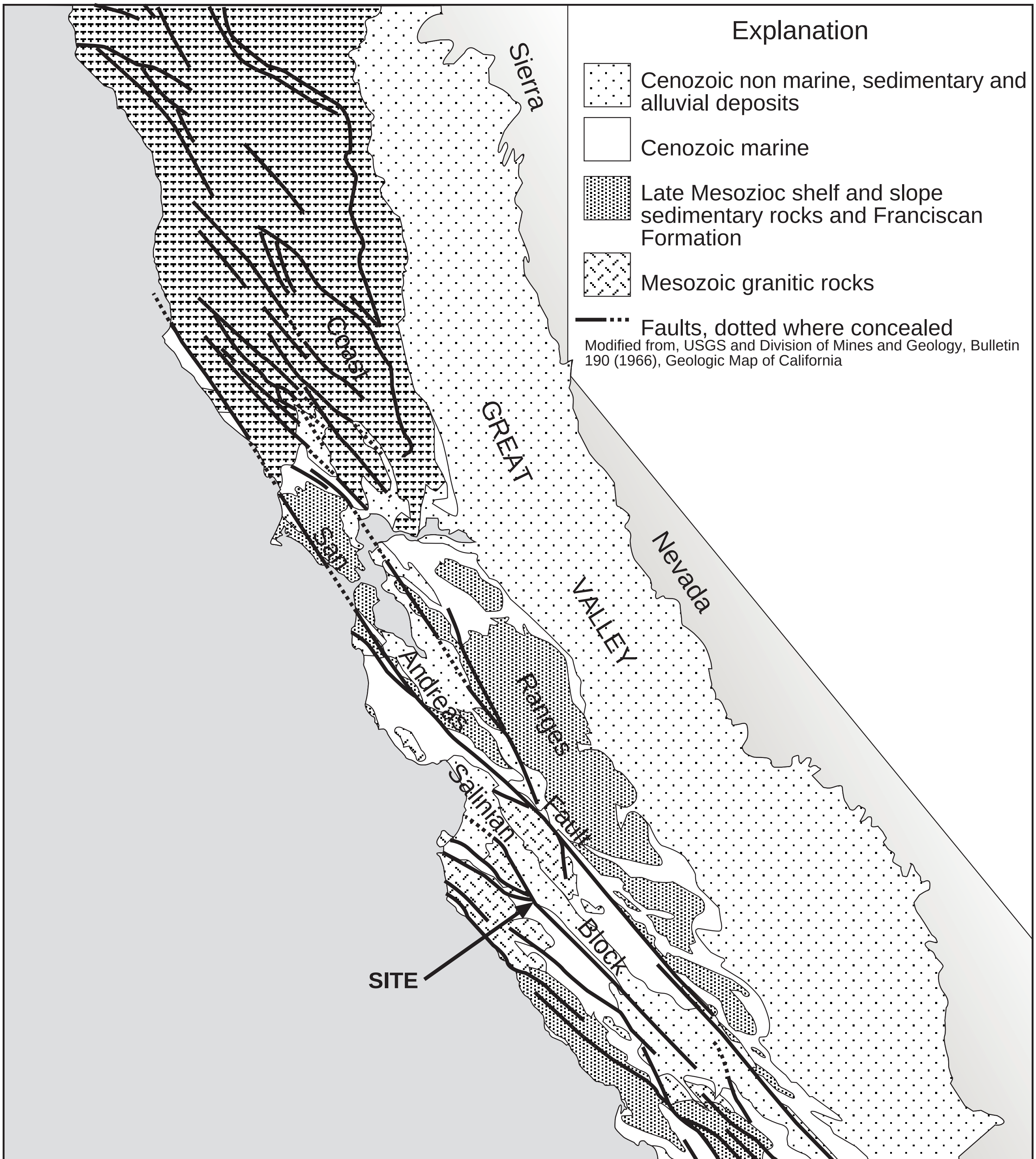
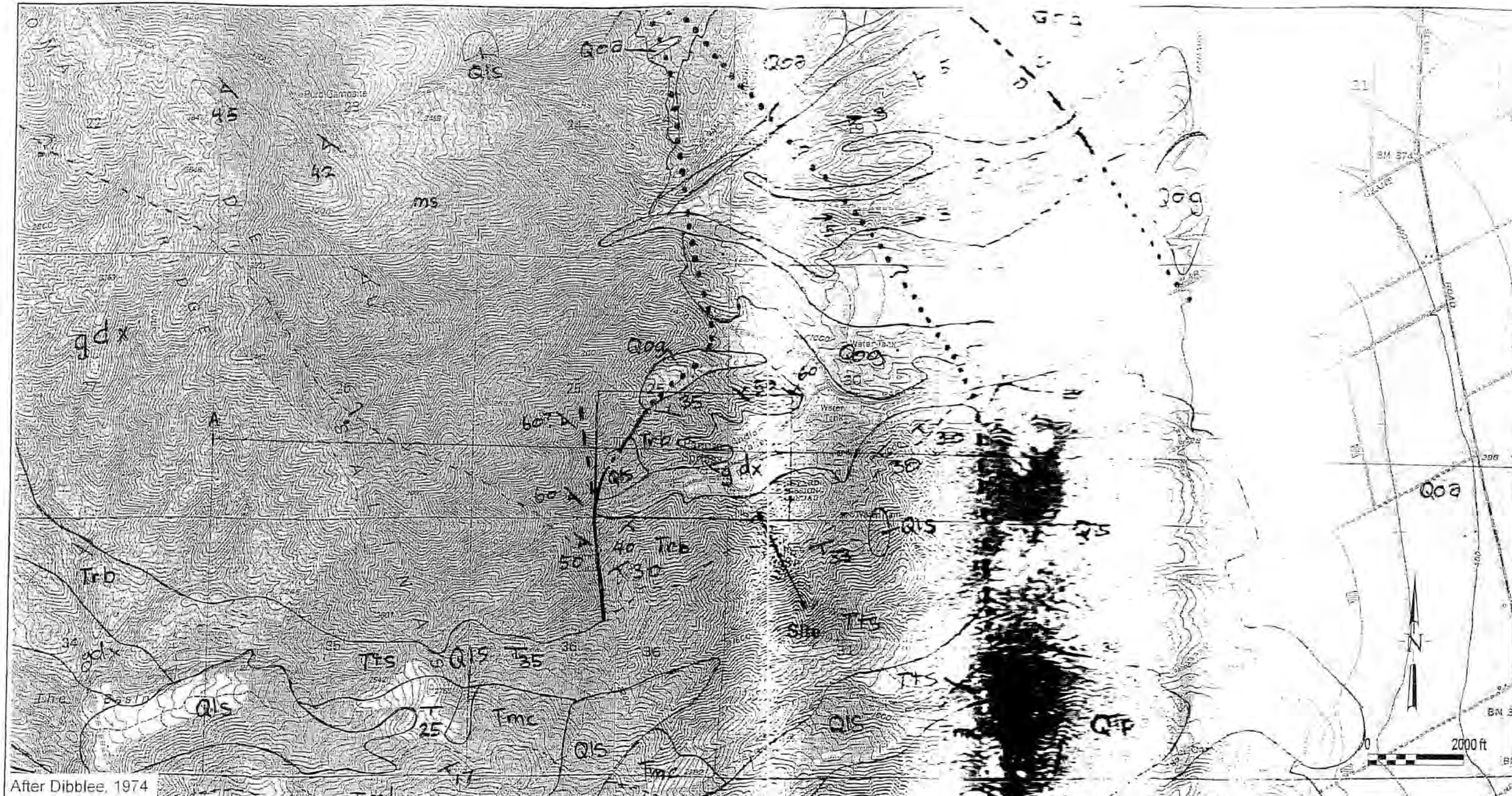
ENGINEERS, INC.

	Vicinity Map	FIGURE
	Paraiso Hot Springs	1
	Paraiso Springs Road	PROJECT
	Greenfield/Soledad Area	LSW-0337-01
	Monterey County, CA	



LANDSET ENGINEERS, INC. ENGINEERING - LANDPLANNING SURVEYING - ENVIRONMENTAL CONSULTING 520-B Crazy Horse Canyon Road, Salinas, CA 93907	**Regional Geologic Map** Paraiso Hot Springs Paraiso Springs Road Greenfield/Soledad Area Monterey County, CA	**FIGURE** **2** **PROJECT** **LSW-0337-01**



After Dibblee, 1974

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520-B Crazy Horse Canyon Road, Salinas, CA 93907

Geologic Vicinity Map
Paraiso Hot Springs
Paraiso Springs Road
Greenfield/Soledad Area
Monterey County, CA

FIGURE
3

PROJECT
LSW-0337-01

Qg Qs
Qa

Surficial deposits
Qg, river gravel and sand
Qs, dune sand
Qa, alluvium

Qls

Landslide debris

Qoa

Older alluvium

Qog

Older fan gravel

UNCONFORMITY

QTp
QTc

Paso Robles Formation
QTp, pebble conglomerate, sand and clay
QTc, clay

LOCAL UNCONFORMITY

Tus

Unnamed marine sandstone
buff white, fine grained

Tpo

Pancho Rico Formation (marine)
diatomaceous mudstone, shale and fine sandstone

Holocene
Pleistocene
Quaternary
Pliocene
Tertiary

Tm
Tml
Tmc

Monterey shale (m...
Tm, siliceous shale (Upper Miocene...
Tml, semisiliceous sh... foram. shale and ... carbonate beds
Tmc, clay shale (M. M...

Tts

Marine sandsto... (light gray, M. & L.M...

Trb

Unnamed red be...
(non marine red sa... conglom., M. & L.M...

UNCONFORMITY

ga
gr | grd | gqd | bq...

Granitic rocks
ga, aplite, alaskite ...
gr, quartz monzon...
grd, granodiorite
gqd, granodiorite-...
bqd biotite quartz... massive to gneis...
gdx, hornblende gr... with phenocryst...

msc | ms ...

Metamorphic roc...
msc, schist
ms, schist-gnei...
m, marble

Miocene
Oligocene? & Miocene
Tertiary
Mesozoic
Mesozoic or older

...tional
...cated

...ertain
...cealed

...de
...ate
...movement
...ates
...fault plane

syncline

...showing
...plunge

...of strata

...slope of
...deposition

...foliation

...downward
...landslide

...test hole
...or gas

...locality

After Dibblee, 1974

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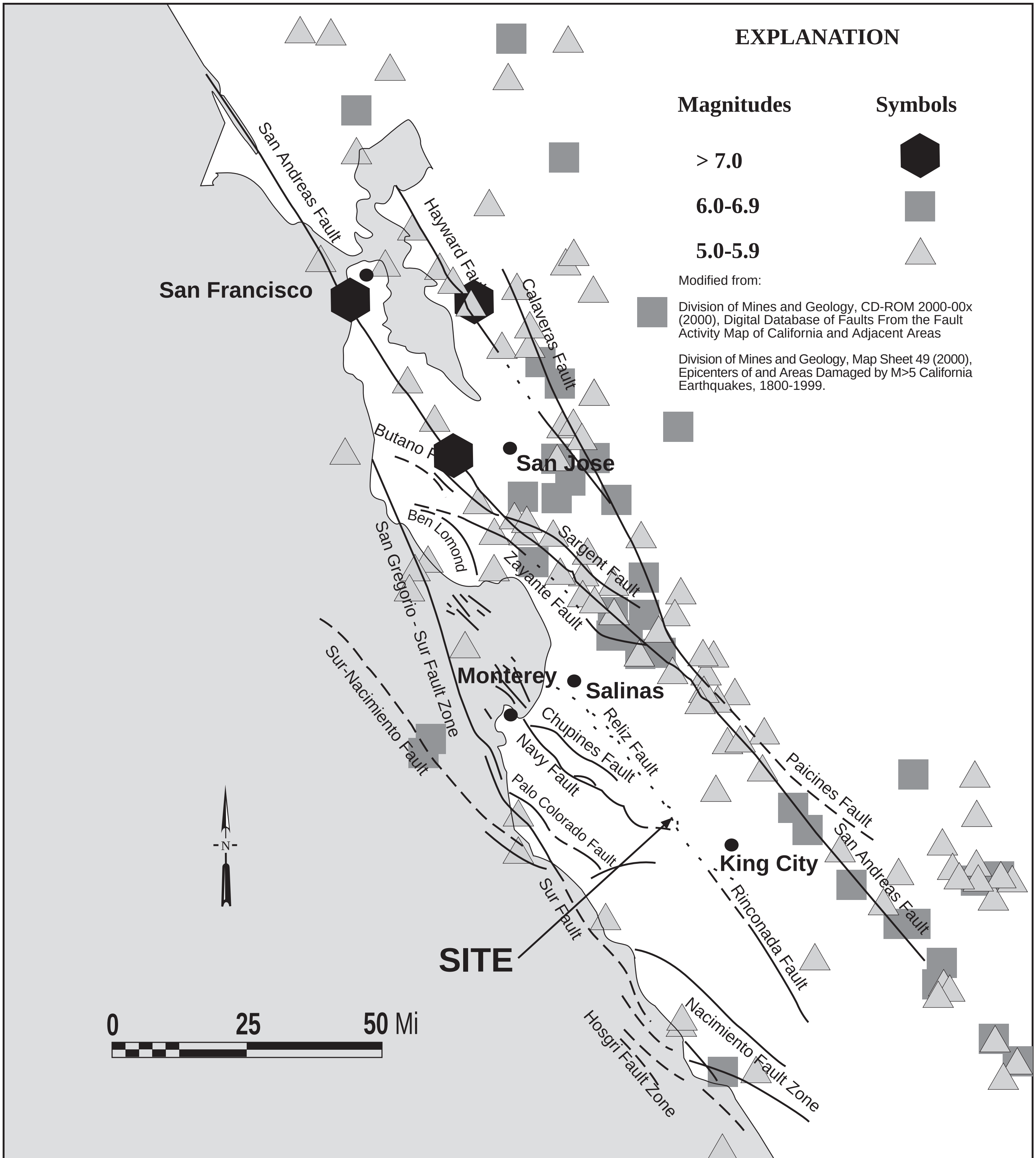
SURVEYING - ENVIRONMENTAL CONSULTING
520-B Crazy Horse Canyon Road, Salinas, CA 93907

Explanati... to Geologic Vicin...
...raiso Hot Springs
...aiso Springs Roa...
...nfield/Soledad Are...
...nterey County, CA

FIGURE
4

PROJECT
LSW-0337-01



LANDSET ENGINEERS, INC. ENGINEERING - LANDPLANNING SURVEYING - ENVIRONMENTAL CONSULTING 520-B Crazy Horse Canyon Road, Salinas, CA 93907	**Regional Fault and Seismicity Map** Paraiso Hot Springs Paraiso Springs Road Greenfield/Soledad Area Monterey County, CA	FIGURE 5 PROJECT LSW-0337-01

APPENDIX A

Unified Soil Classification System
Key to Logs of Borings
Soil Terminology
Exploratory Boring Logs B-1 through B-29

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	Well-graded gravels, gravel-sand mixtures, little or no fines
				GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	More than 50 % of coarse fraction retained on No. 4 sieve.	GRAVELS WITH FINES		GM	Silty gravel, gravel-sand-silt mixtures
				GC	Clayey gravels, gravel-sand-clay mixtures
More than 50 % of material is larger than No. 200 sieve size	SAND AND SANDY SOILS	CLEAN SAND (Little or no fines)		SW	Well graded sands, gravelly sands little or no fines
				SP	Poorly-graded sands, gravelly sands, little or no fines
	More than 50 % of coarse fraction passing No. 4 sieve.	SAND WITH FINES (Appreciable amount of fines)		SM	Silty sands, sand-silt mixtures.
				SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
				OL	Organic silts and organic silty clay of low plasticity
More than 50 % of material is smaller than No. 200 sieve size		LIQUID LIMIT GREATER THAN 50		MH	Inorganic silty, micaceous or diatomaceous fine sand or silty soils
				CH	Inorganic clays of high plasticity, fat clays
				OH	Organic clays of medium to high plasticity, organic silts
	HIGHLY ORGANIC SOILS			PT	Peat, humus, swamp soils with high organic contents
	VARIOUS SOILS AND MAN MADE MATERIALS				Fill materials
	MAN MADE MATERIALS				Asphalt and concrete

KEY TO LOG OF BORINGS

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)

Shelby Sampler
Thin walled, 3" diameter, 3 ft long, hydraulically advanced.

Modified California Sampler
3" diam. split-barrel sampler with brass liners driven by a 140 lb hammer with a drop of 30"

Standard Penetration Test (SPT) Sampler
2" diam. split-barrel sampler driven by a 140 lb hammer with a drop of 30"

Bulk Sample
Loose soil removed for testing.

California Sampler
2.5" diam. split-barrel sampler with brass liners driven by a 140 lb hammer with a drop of 30".
Shaded area denotes sample taken.

Hand Sampler (2.5" diam. driven by hand).

Continuous Core Sampler
94 mm Christianson Sampler

Groundwater encountered during drilling

Groundwater after drilling

Seepage

75 Approximate blows per foot

Solid line denotes soil or lithologic change

Dashed line denotes gradational or approximate soil or lithologic change

Heavy line denotes termination of boring

N.R. = No sample recovered
D.S. = Disturbed sample

SOIL TERMINOLOGY

SOIL TYPES (Ref. 1)

Boulders:	Particles of rock that will not pass a 12 inch screen
Cobbles:	Particles of rock that will pass a 12 inch screen, but not a 3 inch sieve
Gravel:	Particles of rock that will pass a 3 inch sieve, but not a No 4 sieve
Sand:	Particles that will pass a No. 4 sieve, but not a No. 200 sieve
Silt:	Soil that will pass a No. 200 sieve, that is non-plastic or very slightly plastic, and that exhibits little or no strength when dry
Clay:	Soil that will pass a No. 200 sieve, that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength when dry

MOISTURE AND DENSITY

Moisture Condition:	An observational term; dry, slightly moist, moist, very moist, saturated
Moisture Content:	The weight of water in a sample divided by the weight of dry soil in the soil sample, expressed as a percentage
Dry Density:	The pounds of dry soil in a cubic foot of soil

DESCRIPTORS OF CONSISTENCY (Ref. 3)

Liquid Limit:	The water content at which a No. 40 soil is on the boundary between exhibiting liquid and plastic characteristics. The consistency feels like soft butter.
Plastic Limit:	The water content at which a No. 40 soil is on the boundary between exhibiting plastic and semi-solid characteristics. The consistency feels like stiff putty
Plasticity Index:	The difference between the liquid limit and the plastic limit, i.e. the range in water contents over which the soil is in a plastic state.

MEASURES OF CONSISTENCY OF COHESIVE SOILS (CLAYS) (Refs. 2 & 3)

Very soft	N=0-1 *	C=0-250 psf	Squeezes between fingers
Soft	N=2-4	C=250-500 psf	Easily molded by finger pressure
Medium Stiff	N=5-8	C=500-1000 psf	Molded by strong finger pressure
Stiff	N=9-15	C=1000-2000 psf	Dented by strong finger pressure
Very Stiff	N=16-30	C=2000-4000 psf	Dented slightly by finger pressure
Hard	N>30	C>4000 psf	Dented slightly by a pencil point

* N = Blows per foot in the Standard Penetration Test. In cohesive soils, with the 3" diameter sampler, 140 pound weight, divide the blow count by 1.2 to get N (Ref. 4).

MEASURES OF RELATIVE DENSITY OF GRANULAR SOILS (GRAVELS, SANDS AND SILTS) (Refs. 2 & 3)

Very Loose	N=0-4 **	RD=0-30	Easily push a 1/2" reinforcing rod by hand
Loose	N=5-10	RD=30-50	Push a 1/2" reinforcing rod by hand
Medium Dense	N=11-30	RD=50-70	Easily drive a 1/2" reinforcing rod
Dense	N=31-50	RD=70-90	Drive a 1/2" reinforcing rod 1 foot
Very Dense	N>50	RD=90-100	Drive a 1/2" reinforcing rod a few inches

** N = Blows per foot in the Standard Penetration Test. In granular soils, with the 3" diameter sampler, 140 pound weight, divide the blow count by 2 to get N (Ref. 4). RD = Relative Density

Ref. 1 ASTM Designation: D 2487-93, Standard Classification of Soils for Engineering Purposes (Unified Soils Classification System)

Ref. 2 Terzaghi, Karl, and Peck, Ralph B., Soil Mechanics in Engineering Practice, John Wiley & Sons, New York, 2nd Ed., 1967, pp. 30, 341, 347

Ref. 3 Sowers, George F., Introductory Soil Mechanics and Foundations: Geotechnical Engineering, Macmillan Publishing Company, New York, 4th Ed., 1979, pp. 80-81 and 312.

Ref. 4 Lowe, John III, and Zaccheo, Phillip F., Subsurface Explorations and Sampling, Chapter 1 in "Foundation Engineering Handbook," Hsai-Yang Fang, Editor, Van Nostrand Reinhold Company, New York, 2nd Ed., 1991, p. 39

EXPLORATORY BORING LOG

No. B-1 pg 1 of 2

PROJECT: Paraiso Hot Springs — DATE DRILLED: 23-Aug-04 — FILE No. LSW-0337-01

DRILLER: Exploration Geoservice — DRILLING METHOD: B-56 — LOGGED BY: IMS

BORING DIAMETER: 8" HS — BORING DEPTH: 45.0' — GROUNDWATER DEPTH: 6.5'

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1	BULK A				**Qal 2: Alluvium (Holocene)** Dark yellowish brown clayey SAND, medium dense, dry to slightly moist well graded	SC		
2								
3								
4	1-1		32	>4.5	Dense, slightly moist		6.9	[illegible]
5								
6								
7	1-2		19	2.5	Medium dense slightly moist, increase fines content		4.1	123.2
8								
9								
10					Mottled light gray and brownish orange well grade SAND with clay, medium dense, very moist	SW		
11								
12	1-3		18	3.0			8.2	[illegible]
13								
14								
15								
16								
17	1-4		18		Occasional interbeds of discontinuous coarse grained angular poorly graded sand and clay		12.3	
18								
19								
20								
21								
22	1-5		13		Medium dense, saturated		[illegible]	
23								
24								
25					Color changed to light grayish olive, trace fines			
26								
27	1-6		22				[illegible]	

EXPLORATORY BORING LOG

No. B-1 pg 2 of 2

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	23-Aug-04
FILE No.	LSW-0337-01	DRILLER:	Exploration Geoservice
DRILLING METHOD:	B-56	LOGGED BY:	BP
BORING DIAMETER:	8" HS	BORING DEPTH:	45.0'
GROUNDWATER DEPTH:	6.5'		

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
27					Light grayish olive well graded SAND, medium dense saturated, trace fines	SW		
28								
29								
30								
31								
32	1-7		14				[illegible].6	
33								
34					Color changed to moderate gray			
35								
36	1-8		16				[illegible]	
37								
38								
39–40	1-9		5		Loose		18.0	
41								
42								
43								
44–45	1-10		17		Medium dense		[illegible].6	

TD @ 45.0'
GROUNDWATER ENCOUNTERED @ 18.0'
GROUNDWATER ENCOUNTERED @ 6.5'
30 MINUTES AFTER DRILLING

Water temperature 73.4 F
Slight sulfur odor

EXPLORATORY BORING LOG

No. B-2

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	23-Aug-04	FILE No.	LSW-0337-01
DRILLER:	California Geotech	DRILLING METHOD:	B-24	LOGGED BY:	IMS
BORING DIAMETER:	4" SS	BORING DEPTH:	21.5'	GROUNDWATER DEPTH:	N/A

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.C.S.C. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Qal 2: Alluvium (Holocene)** Dark reddish brown clayey SAND, dense dry 10-20% fines, fine to coarse sand	SC		
2								
3	2-1		46	>4.5			2.3	109.6
4								
5								
6	2-2		50/5"		Color change to pinkish brown		1.8	113.1
7								
8					Color change to dark reddish brown			
9								
10								
11	2-3		77		Driller added water, trace fine angular gravel, fines clayey		1.9	
12								
13								
14								
15					Drilling hard			
16	2-4		85				5.4	
17								
18								
19								
20								
21	2-5		25		Sample is moist		10.4	
22					**TD @ 21.5'** **NO GROUNDWATER ENCOUNTERED**			
23								
24								
25								
26								
27								

EXPLORATORY BORING LOG

No. B-3 pg 1 of 2

PROJECT: Paraiso Hot Springs — DATE DRILLED: 23-Aug-04 — FILE No. LSW-0337-01

DRILLER: Exploration Geoservices — DRILLING METHOD: B-56 — LOGGED BY: BP

BORING DIAMETER: 8" HS — BORING DEPTH: 30.0' — GROUNDWATER DEPTH: 15.0'

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Qal 2: Alluvium (Holocene)** Yellowish brown silty SAND, loose, slightly moist, well graded, trace gravel	SM		
2								
3	3-1		10	3.25			[illegible]4	104.0
4								
5								
6	3-2		17	>4.5	Medium dense, slightly moist to moist		6.2	112.2
7								
8								
9								
10					Light yellowish brown well graded SAND, medium dense, moist to very moist	SW		
11	3-3		17	1.25			9.1	106.3
12								
13								
14								
15								
16	3-4		9		Loose, saturated		15.3	
17								
18								
19					Color change to dark gray, slightly clayey, very loose to loose, saturated			
20								
21	3-5		6				[illegible]3	
22								
23					Common stiff clay interbeds			
24								
25					Dark yellowish brown clayey SAND, medium dense, saturated trace gravel	SC		
26	3-6		15				[illegible]	
27								

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EXPLORATORY BORING LOG

No. B-3 pg 2 of 2

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	23-Aug-04	FILE No.	LSW-0337-01
DRILLER:	Exploration Geoservices	DRILLING METHOD:	B-56	LOGGED BY:	BP
BORING DIAMETER: 8" HS	BORING DEPTH: 30.0'	GROUNDWATER DEPTH:	15.0'		

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
26					Dark gray well graded SAND with clay medium dense saturated. trace of gravel	SW		
29								
	3-7		15				18.0	
30								

TD @ 30.0'
GROUNDWATER ENCOUNTERED @ 15.0'
GROUNDWATER ENCOUNTERED @ 19.0'
30 MINUTES AFTER DRILLING

Water Temp 73.0 F
No Odor

EXPLORATORY BORING LOG

No. B-4

PROJECT: Paraiso Hot Springs
DRILLER: California Geotech
BORING DIAMETER: 4" SS
BORING DEPTH: 21.5'
DATE DRILLED: 23-Aug-04
DRILLING METHOD: B-24
GROUNDWATER DEPTH: N/A
FILE No. LSW-0337-01
LOGGED BY: IMS

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture % dry weight	Dry Density (pcf)
0	BULK				**Qal 2: Alluvium (Holocene)**			
1	C				Dark reddish brown clayey SAND, medium dense, dry 10-20% fines, fine to coarse sand	SC		
2								
3	4-1		23	1.5			[illegible]	107.7
4								
5								
6	4-2		89	>4.5			3.4	115.8
7								
8								
9								
10					Color change to pinkish brown			
11	4-3		50/6	>4.5			[illegible]	115.1
12								
13								
14								
15					Pinkish brown clayey sand with gravel dense, dry 10-20% fines, fine to coarse sand, 10-20% fine gravel			
16	4-4		43				3.5	
17								
18								
19								
20								
21	4-5		56				[illegible]	
22					**TD @ 21.5'**			
23					**NO GROUNDWATER ENCOUNTERED**			
24								
25								
26								
27								

EXPLORATORY BORING LOG

No. B-5 pg 1 of 2

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	23-Aug-04	FILE No.	LSW-0337-01
DRILLER:	Exploration Geoservices	DRILLING METHOD:	B-56	LOGGED BY:	BP
BORING DIAMETER: 8" HS		BORING DEPTH:	40.0'	GROUNDWATER DEPTH:	11.5'

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1	BULK B				**Qal 2: Alluvium (Holocene):** Yellowish brown well graded SAND, medium dense, dry, 5-15% fines	SW		
2								
3	S-1		30	>4.5			2.6	126.2
4								
5								
6	S-2		20	>4.5	Very moist		[illegible]	118.8
7								
8								
9					Dark yellowish brown clayey SAND, medium dense, very moist well graded	SC		
10								
11	S-3		22	2.25			14.0	112.[illegible]
12								
13								
14								
15								
16	S-4		25	[illegible]			[illegible]	115.1
17								
18								
19								
20	S-5		9				14.[illegible]	
21					Light olive well graded SAND, loose, saturated	SW		
22								
23								
24								
25					Light olive sandy lean CLAY, stiff, very moist	CL		
26	S-6		10				[illegible]	
27								

EXPLORATORY BORING LOG				No.	B-5 pg 2 of 2
PROJECT:	Paraiso Hot Springs	DATE DRILLED:	23-Aug-04	FILE No.	LSW-0337-01
DRILLER:	Exploration Geoservice	DRILLING METHOD:	B-56	LOGGED BY:	BP
BORING DIAMETER: 8" HS		BORING DEPTH: 40.0'		GROUNDWATER DEPTH:	11.5'

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
27					Light olive sandy lean CLAY, stiff, very moist, well graded sand fraction	CL		
28								
29								
30					Light olive poorly graded SAND, medium dense, saturated,	SP		
31	S-7		16		very fine to medium grained		14.8	
32								
33								
34								
35								
36	S-8		36		Common discontinuous clay lenses		[illegible]	
37					Dense			
38								
39	S-9		31				17.8	
40								

TD @ 40.0'
GROUNDWATER ENCOUNTERED @ 21.0'
GROUNDWATER ENCOUNTERED @ 11.5'
30 MINUTES AFTER DRILLING

Water Temp 79.0 F No Odor

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EXPLORATORY BORING LOG

No. B-6

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	23-Aug-04	FILE No.	LSW-0337-01
DRILLER:	California Geotech	DRILLING METHOD:	B-24	LOGGED BY:	IMS
BORING DIAMETER:	4" SS	BORING DEPTH:	21.5'	GROUNDWATER DEPTH:	N/A

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture % dry weight	Dry Density (pcf)
0								
					Qal 2: Alluvium (Holocene):	SC		
1					Reddish brown (5YR4/3) clayey SAND, very dense dry 10 20% fines, fine to coarse sand			
2								
3	6-1		52				3.0	113.6
4								
5								
6	6-2		50/6"				3.0	116.2
7								
8								
9								
10								
11	6-3		35				3.1	
12								
13								
14								
15					Driller added water, drilling difficult			
16	6-4		35		Fine to medium sand, trace coarse sand		3.5	
17								
18								
19								
20								
21	6-5		45				3.9	
22					**TD @ 21.5'** **NO GROUNDWATER ENCOUNTERED**			
23								
24								
25								
26								

EXPLORATORY BORING LOG

No. B-7pg 1 of 2

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	23-Aug-04	FILE No.	LSW-0337-01
DRILLER:	Exploration Geoservice	DRILLING METHOD:	B-56	LOGGED BY:	BP
BORING DIAMETER:	8" HS	BORING DEPTH:	55.0'	GROUNDWATER DEPTH:	8.0'

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
					Qal 2: Alluvium (Holocene)	SM		
1					Dark yellowish brown silty SAND, loose, moist, well graded			
2								
3	7-1		14				[illegible]	[illegible]
4								
					Light yellowish brown well graded SAND, loose to medium dense	SW		
5								
6	7-2		15				[illegible]	105.6
7								
8								▼
9								
10					Dark gray silty SAND, loose, very moist, well graded, slight odor, saturated @ 11.0'	SM		
11	7-3		9				24.0	[illegible]
12								
					Dark gray well graded SAND, loose, saturated	SW		
13								
14								
15								
16	7-4		6				[illegible]	
17								
18								
19					Color change to light gray			
20								
21	7-5		6				[illegible]	
22								
23								
24								
25								
26	7-6		[illegible]		Loose to medium dense			
27							[illegible]	
28					Light gray well graded sand, medium dense, saturated			

EXPLORATORY BORING LOG

No. B-7 pg 2 of 2

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	23-Aug-04	FILE No.	LSW-0337-01
DRILLER:	Exploration Geoservice	DRILLING METHOD:	B-56	LOGGED BY:	IMS
BORING DIAMETER:	8" HS	BORING DEPTH:	55.0'	GROUNDWATER DEPTH:	8.0'

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
29						SW		
30								
31	7-7		26		Common thin stiff clay lenses		[illegible]	
32								
33								
34								
35								
36	7-8		[illegible]		Medium dense		38.6	
37								
38								
39								
40								
41	7-9		11				27.5	
42								
43								
44								
45								
46	7-10		13				[illegible]	
47								
48								
49								
50	7-11		28				[illegible]	
51								
52								
53								
54	7-12		43		Dense		[illegible]	
55								

TD @ 55.0'
GROUNDWATER ENCOUNTERED @ 11.0'
GROUNDWATER ENCOUNTERED @ 8.0'
30 MINUTES AFTER DRILLING

EXPLORATORY BORING LOG

No. B-8

PROJECT: Paraiso Hot Springs	DATE DRILLED: 23-Aug-04	FILE No. LSW-0337-01	
DRILLER: California Geotech	DRILLING METHOD: B-24	LOGGED BY: IMS	
BORING DIAMETER: 4" SS	BORING DEPTH: 21.5'	GROUNDWATER DEPTH: N/A	

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture % dry weight	Dry Density (pcf)
0								
	BULK				**Qal 2: Alluvium (Holocene):**	SP		
1	D				Yellowish brown (10YR5/3) SAND with clay and gravel,			
					dry, <10% fines, fine to coarse sand fine to medium gravel			
2								
3								
	B-1		28				1.4	112.3
4								
5								
					Color change to reddish brown (10YR4/3), fines, 10-20%			
6								
	B-2		70				[illegible]	[illegible]
7								
8								
9								
10								
11								
	B-3		49				[illegible]	
12								
13								
14								
15								
16								
	B-4		65		Drilling difficult		[illegible]	
17								
18								
19					Cobbles			
20								
21								
	B-5		[illegible]				[illegible]	
22					TD @ 21.5'			
					NO GROUNDWATER ENCOUNTERED			
23								
24								
25								
26								
27								

EXPLORATORY BORING LOG

No. B-9 pg 1 of 2

PROJECT: Paraiso Hot Springs — DATE DRILLED: 23-Aug-04 — FILE No. LSW-0337-01

DRILLER: Exploration Geoservice — DRILLING METHOD: B-56 — LOGGED BY: BP

BORING DIAMETER: 8" HS — BORING DEPTH: 30.0' — GROUNDWATER DEPTH: 7.0'

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
	BULK				**HF: Fill (Holocene):**	SC		
1	C				Dark yellowish brown clayey SAND loose, slightly moist to moist, well graded			
2								
3					**Qal 2: Alluvium (Holocene):**	SM		
	9-1		13		Yellowish gray silty SAND, loose, slightly moist well graded		2.5	84.1
4								
5								
6								
	9-2		10				11.2	108.7
7								
8								
9					Color change to yellowish brown very moist to saturated			
10								
11								
	9-3		14				17.2	109.1
12								
					Dark gray clayey SAND, loose saturated, moderate odor	SC		
13								
14								
15								
16								
	9-4		8				16.8	100.3
17								
18								
19								
					Orange brown well graded SAND, loose, saturated	SW		
20								
21								
	9-5		13				16.5	106.7
22								
23								
24								
25								
26								
	9-6		12				19.8	
27								

EXPLORATORY BORING LOG

No. B-9 pg 2 of 2

PROJECT: Paraiso Hot Springs	DATE DRILLED: 23-Aug-04	FILE No. LSW-0337-01	
DRILLER: Exploration Geoservice	DRILLING METHOD: B-56	LOGGED BY: BP	
BORING DIAMETER: 8" HS	BORING DEPTH: 30.0'	GROUNDWATER DEPTH: 7.0'	

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
28					Orange brown well graded SAND, loose saturated	SW		
29	9-7		2		Color change to dark gray, very loose		[illegible]	
30								

TD @ 30.0'

GROUNDWATER ENCOUNTERED @ 12.0'
GROUNDWATER ENCOUNTERED @ 7.0'
15 MINUTES AFTER DRILLING

Water Temp 80.9 F No Odor

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EXPLORATORY BORING LOG

No. B-10

PROJECT: Paraiso Hot Springs
DRILLER: California Geotech
BORING DIAMETER: 4" SS
BORING DEPTH: 10.5'
DATE DRILLED: 24-Aug-04
DRILLING METHOD: B-24
GROUNDWATER DEPTH: N/A
FILE No. LSW-0337-01
LOGGED BY: IMS

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil-Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Qal 2: Alluvium (Holocene)** Reddish brown (5YR4/4) Clayey SAND very dense dry 15-25% fines, fine to coarse sand	SC		
2								
3	10-1		50/3"				2.1	119.4
4								
5					**Tt: Tierra Redonda Fm (Miocene)** Light yellowish brown (10YR6/4) SANDSTONE, very dense, moist, slightly weathered, friable, plastic when wet, rock hardness firm, fracture and bedding unknown, grain size fine to coarse sandy, subrounded to subangular, abundant biotite, slightly cemented			
6	10-2		50/3"				9.1	112.7
7								
8								
9								
10	10-3		50/6"				6.5	
11					**TD @ 10.5'** **NO GROUNDWATER ENCOUNTERED**			
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								

EXPLORATORY BORING LOG

No. B-11 pg 1 of 2

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	24-Aug-04
FILE No.	LSW-0337-01	DRILLER:	Exploration Geoservice
DRILLING METHOD:		LOGGED BY:	BP
BORING DIAMETER:	8" HS	BORING DEPTH:	46.5'
GROUNDWATER DEPTH:	18.2'		

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Qal 2: Alluvium (Holocene)** Yellowish brown silty SAND, medium dense, dry, well graded	SM		
2								
3	11-1		20				1.3	
4								
5								
6	11-2		30	>4.5			3.0	95.9
7								
8								
9								
10								
11	11-3		15	>4.5	Color change to light yellowish gray, loose to medium dense, slightly moist increase fines		6.6	104.9
12								
13								
14								
15								
16	11-4		11		Very moist Dark gray poorly graded SAND medium dense, very moist	SP	20.9	
17								
18								
19					Brown gray poorly graded SAND medium dense, very moist	SW		
20								
21	11-5		12				13.9	
22								
23								
24								
25								
26	11-6		15		Common thin silty sand and clayey sand interbeds		11.3	
27								

EXPLORATORY BORING LOG

No. B-11 pg 2 of 2

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	24-Aug-04	FILE No.	LSW-0337-01
DRILLER:	Exploration Geoservice	DRILLING METHOD:	B-56	LOGGED BY:	BP
BORING DIAMETER:	8" HS	BORING DEPTH:	46.5'	GROUNDWATER DEPTH:	18.2'

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
28						SM		
29					Color change to greenish gray, medium dense, saturated			
30								
31	11-7		29				14.0	
32								
33								
34					Grayish brown silty SAND medium dense, saturated, trace pea gravel	SM		
35								
36	11-8		27				18.9	
37								
38								
39								
40								
41	11-9		23				[illegible]	
42								
43					Color change to dark gray, medium dense to dense			
45								
46	11-10		33				[illegible]	
47								

TD @ 46.5'
GROUNDWATER ENCOUNTERED @ 18.5'
DURING DRILLING
GROUNDWATER ENCOUNTERED @ 18.2'
30 MINUTES AFTER DRILLING

Water Temp 94.1 F
No Odor

EXPLORATORY BORING LOG

No. B-12

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	24-Aug-04	FILE No.	LSW-0337-01
DRILLER:	California Geotech	DRILLING METHOD:	B-24	LOGGED BY:	IMS
BORING DIAMETER:	4"SS	BORING DEPTH:	15.25'	GROUNDWATER DEPTH:	N/A

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Soil:** Reddish brown for first 2" then becomes brown (10YR5/3) clayey SAND, very dense, moist, 15-25% fines, fine to coarse sand	SC		
2	12-1		50/3"				[illegible]	[illegible]
3								
4								
5					**Tt: Tierra Redonda Fm (Miocene)** Light yellowish brown (2.5Y6/4) SANDSTONE, very dense, dry, slightly weathered, friable, plastic when wet, firm rock hardness, fracture spacing and bedding unknown, slightly to moderately cemented			
6	12-2		85/9"				[illegible]	
7								
8								
9								
10								
11	12-3		97/9"				[illegible]	
12								
13								
14								
15	12-4		50/3"				[illegible]	
16					**TD @ 15.25'** **NO GROUNDWATER ENCOUNTERED**			
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								

EXPLORATORY BORING LOG

No. B-13 pg 1 of 2

PROJECT: Paraiso Hot Springs | DATE DRILLED: 24-Aug-04 | FILE No. LSW-0337-01

DRILLER: Exploration Geoservice | DRILLING METHOD: B-56 | LOGGED BY: BP

BORING DIAMETER: 8" HS | BORING DEPTH: 50.0' | GROUNDWATER DEPTH: 9.7'

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Qal 2: Alluvium (Holocene)** Light yellowish brown well graded SAND, medium dense, dry	SW		
2								
3								
4	13-1		25	>4.5			3.1	105.2
5								
6								
7	13-2		19		Dry		3.4	102.6
8								
9								
10					Color change to orange brown, very moist			
11								
12	13-3		16		Saturated @ 11.5'		16.7	101.6
13								
14					Grayish olive silty SAND with clay, loose, saturated, common thin well graded sand, silt and clay interbeds, slight odor	SM		
15								
16								
17	13-4		8				20.7	
18								
19								
20								
21								
22	13-5		5				27.7	
23								
24					Color change to dark gray, moderate odor			
25								
26								
27	13-6		7				[illegible]	
28								

EXPLORATORY BORING LOG

No. B-13 pg 2 of 2

PROJECT: Paraiso Hot Springs	DATE DRILLED: 24-Aug-04	FILE No. LSW-0337-01	
DRILLER: Exploration Geoservice	DRILLING METHOD: B-56	LOGGED BY: BP	
BORING DIAMETER: 8" HS	BORING DEPTH: 50.0'	GROUNDWATER DEPTH: 9.7'	

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
29								
30					Dark gray silty SAND, with clay loose to medium dense, saturated trace gravels	SM		
31	N/R		11					
32								
33								
34								
35								
36	13-7		13				18.3	
37								
38								
39								
40								
41	13-8		22		Dense		21.9	
42								
43								
44					Occasional poorly graded, very coarse grained sand interbeds			
45								
46	13-9		16				16.9	
47								
48								
49								
50	13-10		12				11.8	

TD @ 50.0'
GROUNDWATER ENCOUNTERED @ 12.0'
WHILE DRILLING
GROUNDWATER ENCOUNTERED @ 9.7'
45 MINUTES AFTER DRILLING

Water Temp 95 F
Slight Odor

EXPLORATORY BORING LOG

No. B-14

PROJECT: Paraiso Hot Springs	**DATE DRILLED:** 24-Aug-04	**FILE No.** LSW-0337-01	
DRILLER: California Geotech	**DRILLING METHOD:** B-24	**LOGGED BY:** IMS	
BORING DIAMETER: 4"SS	**BORING DEPTH:** 26.5'	**GROUNDWATER DEPTH:** N/A	

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Soil:** Reddish brown (5YR4/4) clayey SAND, very dense, dry, 15-25% fines, fine to coarse sand	SC		
2								
3	14-1		50/6				6.7	126.9
4					**Tt: Tierra Redonda Fm (Miocene)** Yellowish brown (10YR5/4) SANDSTONE, very dense, dry, slightly weathered, friable, plastic when wet, rock hardness firm, fracture and bedding unknown, grain size fine to coarse, subrounded to subangular, slightly cemented			
5								
6	14-2		50/6"				1.9	
7								
8								
9					Sand is clean, no cementing			
10								
11	14-3		45				1.9	
12								
13								
14								
15					Fines down to 10-20%			
16	14-4		68				6.0	
17								
18								
19								
20								
21	14-5		97/11"				[illegible]	
22								
23								
24								
25								
26	14-6		86				[illegible]	

TD @ 26.5'
NO GROUNDWATER ENCOUNTERED

EXPLORATORY BORING LOG

No. B-15

PROJECT: Paraiso Hot Springs	DATE DRILLED: 24-Aug-04	FILE No. LSW-0337-01	
DRILLER: Exploration Geoservice	DRILLING METHOD: B-56	LOGGED BY: BP	
BORING DIAMETER: 8" HS	BORING DEPTH: 18.75'	GROUNDWATER DEPTH: N/A	

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Qal 2: Alluvium (Holocene)** Light orange brown well graded SAND, medium dense, slightly moist	SW		
2								
3								
	N/R		16					
4								
5								
6	15-1			3.0	Grayish brown SILT, stiff, moist	ML	[illegible]	93.3
	15-2		18	3.0	Dusky yellowish brown organic SILT, stiff, very moist	MH	[illegible]	75.8
7					Light gray silty SAND, medium dense, very moist, 40-45% fines	SM		
8								
9								
10								
11					**Kgd: Granite (Cretaceous):**			
	15-3		50	>4.5	Red, dense		[illegible]	100.5
12								
13								
14								
15								
	15-4		50/2		Color change to gray		0.6	
16								
17								
18								
	15-5		50/3				3.5	
19					**TD @ 18.75'** **NO GROUNDWATER ENCOUNTERED**			
20								
21								
22								
23								
24								
25								
26								

EXPLORATORY BORING LOG

No. B-16

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	24-Aug-04	FILE No.	LSW-0337-01
DRILLER:	California Geotech	DRILLING METHOD:	B-24	LOGGED BY:	IMS
BORING DIAMETER:	4" SS	BORING DEPTH:	16.5'	GROUNDWATER DEPTH:	N/A

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Soil:** Reddish brown (5YR4/4) clayey SAND very dense, dry, 10-20% fines, fine to coarse sand	SC		
2	16-1		50-6"	>4.5			4.6	119.7
3								
4								
5	16-2		50/6"		**Tt: Tierra Redonda Fm (Miocene)** Yellowish brown (10YR5/4) SANDSTONE, very dense dry slightly weathered, friable, plastic when wet, firm rock hardness fracture and bedding unknown, fine to coarse sand slightly cemented		[illegible]	
6								
7								
8								
9								
10					Drilling becomes difficult			
11	16-3		50-6				[illegible]	
12								
13								
14								
15								
16	16-4		79				[illegible]	
17					TD @ 16.5' NO GROUNDWATER ENCOUNTERED			
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								

EXPLORATORY BORING LOG

No. B-17 pg 1 of 2

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	24-Aug-04	FILE No.	LSW-0337-01
DRILLER:	Exploration Geoservice	DRILLING METHOD:	B-56	LOGGED BY:	BP
BORING DIAMETER:	8" HS	BORING DEPTH:	50.0'	GROUNDWATER DEPTH:	31.5'

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
	BULK				**Qal 2: alluvium (Holocene)**	SM		
1	F				Pale reddish brown silty SAND, dense, dry, very fine to fine grained			
2								
3								
	17-1		43	>4.5	Slightly moist		3.8	112.6
4								
5								
6								
	17-2		33		Medium dense, well graded		20.5	[illegible]
7								
8								
					Light yellowish brown well graded SAND, medium dense slightly moist, rare gravels	SW		
9								
10								
11								
	17-3		23				[illegible]	[illegible]
12								
13								
14								
15								
16								
	17-4		35				[illegible]	
17								
18								
19								
20								
21								
	17-5		15				[illegible]	
22								
23								
24								
25								
26								
	17-6		41		Abundant gravels from 24.0 to 26.0		[illegible]	
27								
					Dense			
28								

EXPLORATORY BORING LOG

No. B-17 pg 2 of 2

PROJECT: Paraiso Hot Springs | DATE DRILLED: 24-Aug-04 | FILE No. LSW-0337-01

DRILLER: Exploration Geoservice | DRILLING METHOD: B-56 | LOGGED BY: BP

BORING DIAMETER: 8" HS | BORING DEPTH: 50.0' | GROUNDWATER DEPTH: 31.5'

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
29					Light yellowish brown well graded SAND, dense, slightly moist trace of gravel	SW		
30								
31	17-7		45				2.5	
32								
33								
34								
35								
36	17-8		51		Very dense, saturated		[illegible]	
37								
38								
39					Color change to olive gray, medium dense, saturated, very slight odor			
40								
41	17-9		28				14.5	
42								
43								
44								
45								
46	17-10		24				[illegible]	
47								
48								
49								
50	17-11		18				[illegible]	

TD @ 50.0'
GROUNDWATER ENCOUNTERED @ 31.5'
DURING DRILLING
GROUNDWATER ENCOUNTERED @ 41.3'
10 MINUTES AFTER DRILLING

Water Temp 95.7 F No odor

EXPLORATORY BORING LOG

PROJECT: Paraiso Hot Springs	**DATE DRILLED:** 24-Aug-04	**No.** B-18	
DRILLER: California Geotech	**DRILLING METHOD:** B-24	**FILE No.** LSW-0337-01	
BORING DIAMETER: 4"SS	**BORING DEPTH:** 11.0'	**GROUNDWATER DEPTH:** N/A	**LOGGED BY:** IMS

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Soil:** Reddish brown (5YR4/4) clayey SAND, very dense dry, 10-20% fines, fine to coarse sand	SC		
2								
3	18-1		50/5				6.1	97.5
4								
5					**Tt: Tierra Redonda Fm (Miocene)** Yellowish brown (10YR5/4) SANDSTONE, very dense dry, slightly weathered, friable, plastic when wet, firm rock hardness fractured and bedding unknown, fine to coarse sand, slightly cemented			
6	18-2		50/5				2.5	
7								
8								
9								
10								
11	18-3		50/4				1.2	
12					**TD @ 11.0'** **NO GROUNDWATER ENCOUNTERED**			
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								

EXPLORATORY BORING LOG

No. B-19 pg 1 of 3

PROJECT: Paraiso Hot Springs	DATE DRILLED: 24-Aug-04	FILE No. LSW-0337-01	
DRILLER: Exploration Geoservice	DRILLING METHOD: B-56	LOGGED BY: BP	
BORING DIAMETER: 8" HS	BORING DEPTH: 60.0'	GROUNDWATER DEPTH: 55.0'	

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Qal 2: Alluvium (Holocene)** Yellowish brown silty SAND, loose, dry, well graded	GM		
2								
3	19-1		9				2.4	103.5
4								
5								
6	19-2		22	3.0	Medium dense, dry, common 1/2" diameter angular granitic gravels		2.9	100.8
7								
8								
9								
10								
11	N/R		19					
12								
13								
14								
15								
16	19-3		13	3.25	Loose, slightly moist		[illegible]	101.2
17								
18								
19								
20								
21	19-4		16		Medium dense		[illegible]	
22								
23								
24								
25								
26	19-5		13				4.1	

EXPLORATORY BORING LOG

No. B-19 pg 2 of 3

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	24-Aug-04	FILE No.	LSW-0337-01
DRILLER:	Exploration Geoservice	DRILLING METHOD:	B-56	LOGGED BY:	BP
BORING DIAMETER:	8" HS	BORING DEPTH:	60.0'	GROUNDWATER DEPTH:	55.0'

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
27					Yellowish brown silty SAND medium dense moist, well graded occasional subangular to angular granitic gravels	SM		
28								
29								
30								
31	19-6		13				4.1	
32								
33								
34								
35								
36	19-7		25				3.5	
37								
38								
39								
40								
41	19-8		24				3.8	
42								
43								
44								
45								
46								
	19-9		18				3.0	
47								
48								
49								
50								
51								
	19-10		37				3.9	
52								
53								

EXPLORATORY BORING LOG

No. 19 pg 3 of 3

PROJECT: Paraiso Hot Springs	DATE DRILLED: 24-Aug-04	FILE No. LSW-0337-01	
DRILLER: Exploration Geoservice	DRILLING METHOD: B-56	LOGGED BY: BP	
BORING DIAMETER: 8" HS	BORING DEPTH: 60.0'	GROUNDWATER DEPTH: 55.0'	

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
54								
55								▼
56	19-11		17				[illegible]	
57								
58								▼
59								
	19-12		11				10.0	
60								

TD @ 60.0
GROUNDWATER ENCOUNTERED @ 55.0'
WHILE DRILLING
GROUNDWATER ENCOUNTERED @ 58.3'
30 MINUTES AFTER DRILLING

Water Temp 95 F

EXPLORATORY BORING LOG

No. B-20

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	24-Aug-04	FILE No.	LSW-0337-01
DRILLER:	California Geotech	DRILLING METHOD:	B-24	LOGGED BY:	IMS
BORING DIAMETER:	4" SS	BORING DEPTH:	16.5'	GROUNDWATER DEPTH:	N/A

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Soil:** Yellowish brown (10YR5/4). SAND with silt, very dense dry <10% fines. fine to coarse angular sand no mica's	SW		
2	20-1		50/3				[illegible]	[illegible]
3								
4								
5								
6	20-2		50/5		Mica's in sample		4.[illegible]	
7								
8								
9								
10					Color change to reddish brown fines increases to 15-25%			
11	20-3		50/5		Drilling hard and slow		4.[illegible]	
12								
13					**Kgd: Granite (Cretaceous)** Reddish brown granite extremely weathered. friable. firm rock hardness. grain size fine to coarse			
14								
15								
16	20-4		82				3.[illegible]	
17					TD @ 16.5' NO GROUNDWATER ENCOUNTERED			
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								

EXPLORATORY BORING LOG

No. B-21

PROJECT: Paraiso Hot Springs | DATE DRILLED: 24-Aug-04 | FILE No. LSW-0337-01

DRILLER: Exploration Geoservice | DRILLING METHOD: B-56 | LOGGED BY: BP

BORING DIAMETER: 8" HS | BORING DEPTH: 24.0' | GROUNDWATER DEPTH: N/A

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Qal 2: Alluvium (Holocene)** Light yellowish brown silty SAND, medium dense, dry well graded			
2								
3	21-1		16				1.4	142.6
4								
5								
6	21-2		34				1.5	
7								
8								
9								
10								
11	21-3		51		**Tt: Tierra Redonda Fm (Micocene)** Grayish red arkosic sandstone dense, slightly moist, friable weakly indurated, severely weathered		6.1	113.9
12								
13								
14								
15								
16	21-4		39				5.0	
17								
18								
19								
20								
21	21-5		39		Moderately weathered		3.6	
22								
23								
24			50.5		Slightly weathered, very dense		2.4	
25								
26								
27								

TD @ 24.0'

NO GROUNDWATER ENCOUNTERED

EXPLORATORY BORING LOG

No. B-22

PROJECT: Paraiso Hot Springs	DATE DRILLED: 24-Aug-04	FILE No. LSW-0337-01	
DRILLER: California Geotech	DRILLING METHOD: B-24	LOGGED BY: IMS	
BORING DIAMETER: 4" SS	BORING DEPTH: 10.5'	GROUNDWATER DEPTH: N/A	

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
					Tt: Tierra Redonda Fm (Miocene)			
1					Yellowish brown (10YR5/4) SANDSTONE, very dense slightly moist, slightly weathered, friable, plastic when wet firm, rock hardness, fracture and bedding unknown, fine to coarse sand, slightly cemented			
2								
3	22-1		50/5	>4.5			[illegible]	[illegible]
4								
5								
6	22-2		50/6				[illegible]	
7								
8								
9								
10	22-3		50/5				3.2	
11					**TD @ 10.5'**			
					NO GROUNDWATER ENCOUNTERED			
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								

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EXPLORATORY BORING LOG

No. B-23 pg 1 of 2

PROJECT: Paraiso Hot Springs | DATE DRILLED: 25-Aug-04 | FILE No. LSW-0337-01
DRILLER: Exploration Geoservice | DRILLING METHOD: B-56 | LOGGED BY: BP
BORING DIAMETER: 8" HS | BORING DEPTH: 39.5' | GROUNDWATER DEPTH: 14.0'

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.C.S.C. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Hf: fill (Holocene)** Grayish brown silty SAND, medium dense, slightly moist	SM		
2								
3	23-1		34	>4.5			9.8	[illegible]
4								
5								
6					Color change to reddish brown			
7	23-2		26	>4.5			7.9	[illegible]
8								
9					**Qal 2: Alluvium (Hollocene)** Dark olive brown silty SAND, medium dense moist	SM		
10								
11	23-3		34				[illegible]	[illegible]
12								
13								
14								
15					Color change to reddish brown, loose, saturated very fine to medium grained			
16	23-4		8				19.4	
17								
18								
19								
20								
21	23-5		11		Loose to medium cense		[illegible]	
22								
23								
24					Reddish brown CLAY stiff, very moist	CL		
25								
26	23-6		14				[illegible]	

EXPLORATORY BORING LOG

No. B-23 pg 2 of 2

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	25-Aug-04	FILE No.	LSW-0337-01
DRILLER:	Exploration Geoservice	DRILLING METHOD:	B-56	LOGGED BY:	BP
BORING DIAMETER:	8" HS	BORING DEPTH:	39.5'	GROUNDWATER DEPTH:	14.0'

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
27					Reddish brown CLAY, hard very moist	CL		
28								
29								
30								
31	23-7		32					
32								
33					Tt: Tierra Redonda Fm			
34								
35								
36	23-8		65					
37								
38								
39			50/5					
40								

TD @ 39.5'
GROUNDWATER ENCOUNTERED @ 14.0'
DURING DRILLING
GROUNDWATER ENCOUNTERED @ 5.5'
30 MINUTES AFTER DRILLING

Water temp 73 F No Odor

EXPLORATORY BORING LOG

No. B-24

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	25-Aug-04	FILE No.	LSW-0337-01
DRILLER:	California Geotech	DRILLING METHOD:	B-24	LOGGED BY:	IMS
BORING DIAMETER:	4"SS	BORING DEPTH:	21.5'	GROUNDWATER DEPTH:	

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Qal 1: Alluvium (Holocene)** Dark yellowish brown (10YR4/4) SAND with gravel and silt, medium dense, dry, <10% fines, fine to coarse angular sand, fine to medium angular gravel, clast entirely of granitic provenance			
2								
3								
	24-1		40				1.8	
4								
5								
6								
	24-2		27				1.0	
7								
8								
9					Boring caved to 8' @ 10' Cleared out to 9' before sampling Sand clear			
10								
11								
	24-3		42				1.6	
12								
13								
14					Caved to 13.5'			
15								
16								
	24-4		36				1.5	
17								
18					Drilling becomes difficult @ 17' - fine gravel abundant in cuttings			
19								
20								
21								
	24-5		29		Trace monterey clasts in sample		2.1	
22					TD @ 21.5' NO GROUNDWATER ENCOUNTERED			
23								
24								
25								
26								
27								

EXPLORATORY BORING LOG

No. B-25

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	25-Aug-04
DRILLER:	California Geotech	DRILLING METHOD:	B-24
BORING DIAMETER:	4"SS	BORING DEPTH:	21.5'
FILE No.	LSW-0337-01	LOGGED BY:	IMS
GROUNDWATER DEPTH:	N/A		

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Qal 1: Alluvium (Holocene)** Brown (10YR4/3) SAND with gravel medium dense, dry, fine to coarse sand, angular, 20-30% fine to coarse angular gravel, clasts are granite and schist	SW		
2								
3	25-1		38	4.0			2.5	102.9
4								
5					Gravel encountered			
6	25-2		15				1.5	
7								
8								
9								
10					Gravel encountered			
11	25-3		51				0.9	
12								
13								
14					Gravel encountered			
15								
16	25-4		56				2.0	
17								
18								
19					Drilling very difficult, cobbles			
20								
21	25-5		58				2.0	
22					**TD @ 21.5'** **NO GROUNDWATER ENCOUNTERED**			
23								
24								
25								
26								
27								

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EXPLORATORY BORING LOG

No. B-26

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	25-Aug-04	FILE No.	LSW-0337-01
DRILLER:	California Geotech	DRILLING METHOD:	B-24	LOGGED BY:	IMS
BORING DIAMETER:	4"SS	BORING DEPTH:	19.5'	GROUNDWATER DEPTH:	N/A

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Qal 2: Alluvium (Holocene)** Yellowish brown (10YR5/4). SAND with gravel, dense dry to coarse sand fine to medium gravel 20-30% angular to sub rounded mostly fine, mix of Tt and Qal	SP		
2								
3								
	26-1		66				1.6	103.7
4								
5								
6								
	26-2		36				0.9	
7								
8								
9								
10					**Tt: Tierra Redonda Fm (Miocene)** Brown (5YR3/2) SANDSTONE, very dense, dry, 20-30% fines, fine to coarse sand mostly fine to medium, mica and plag rich			
11								
	26-3		61				2.7	
12								
13								
14								
15					Drilling difficult			
16								
	26-4		64		Color change to dark brown (7.5YR4/2) fine content decreases to 10-20%		0.7	
17								
18								
19					Drilling very difficult - refusal			
	26-5		63				1.8	
20					TD @ 19.5' NO GROUNDWATER ENCOUNTERED			
21								
22								
23								
24								
25								
26								
27								

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EXPLORATORY BORING LOG

No. B-27

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	25-Aug-04	FILE No.	LSW-0337-01
DRILLER:	California Geotech	DRILLING METHOD:	B-24	LOGGED BY:	IMS
BORING DIAMETER:	4" SS	BORING DEPTH:	6.5'	GROUNDWATER DEPTH:	N/A

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	USCS Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
1					**Soil:** Dark brown (10YR4/1) clayey SAND, very dense,	SC		
2					dry, 25-35% fines, fine to medium sand			
3	27-1		76/11"	>4.5			[illegible]	[illegible]
4					**Tt: Tierra Redonda Fm (Miocene)** Light gray (210YR7/1) SANDSTONE, very dense, dry, slightly			
5					weathered to fresh, cemented, weak firm to moderately hard rock hardness, very closely fractured			
6	27-2		70				[illegible]	
7					**TD @ 6.5'** **NO GROUNDWATER ENCOUNTERED**			
8								
9								
10								
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								

EXPLORATORY BORING LOG

No. B-28

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	25-Aug-04	FILE No.	LSW-0337-01
DRILLER:	California Geotech	DRILLING METHOD:	B-24	LOGGED BY:	IMS
BORING DIAMETER:	4" SS	BORING DEPTH:	5.5'	GROUNDWATER DEPTH:	N/A

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Class	Moisture (% dry weight)	Dry Density (pcf)
0								
					Soil:	SC		
1					Dark brown (10YR4/1) clayey SAND, very dense dry, 25-35% fines, fine to medium sand			
2								
3	28-1		50/6	3.25			[illegible]	112.4
					Tt: Tierra Redonda Fm (Miocene)			
4					Light gray (10YR7/1) SANDSTONE, very dense, dry, slightly weathered to fresh cemented, weak, firm to moderately hard rock hardness, very closely fractured			
5								
	28-2	II	50/5				[illegible]	
6					**TD @ 5.5'**			
					NO GROUNDWATER ENCOUNTERED			
7								
8								
9								
10								
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								

EXPLORATORY BORING LOG

No. B-29

PROJECT:	Paraiso Hot Springs	DATE DRILLED:	24-Aug-04	FILE No.	LSW-0337-01
DRILLER:	California Geotech	DRILLING METHOD:	B-24	LOGGED BY:	IMS
BORING DIAMETER:	4" SS	BORING DEPTH:	6.5'	GROUNDWATER DEPTH:	N/A

Depth (ft)	Sample	Graphic Log	Blows per foot	Pocket Pen (tsf)	Description	U.S.C.S. Soil Group	Moisture (% dry weight)	Dry Density (pcf)
0								
					Soil:	SC		
1					Reddish brown (5YR4/3) clayey SAND, very dense			
					dry, 20-30% fine fine to medium sand			
2								
3	29-1		53.5	>4.5			[illegible]	[illegible]
4								
5					Tt: **Tierra Redonda Fm (Miocene)**			
					Reddish brown SANDSTONE, moderately weathered			
6					slightly cemented, weak firm to moderately hard, fracture			
	29-2		67		unknown		[illegible]	
7					**TD @ 6.5'**			
					NO GROUNDWATER ENCOUNTERED			
8								
9								
10								
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								

APPENDIX B

Water Well Drillers Reports

DEPARTMENT OF WATER RESOURCES

WATER WELL DRILLERS REPORT

Paradise Springs

Paradise Springs Rd.

Wheeler Co. WY 83960

LOCATION OF WELL:

Paradise Springs Rd

WELL LOG:

109

0 - 5' Top soil

15' - 80' Sand

80' - 95' Brown Sand

95' - 105' Rock

105'-109' Rock

TYPE OF WORK (check):

PROPOSED USE (check):

EQUIPMENT:

Rotary

Cable

Other

CASING INSTALLED:

If gravel packed

Diam.	Gage or Wall	Diameter of Drill	From ft.	To ft.
8"	8 Ga.	22"	0	104

1/4 Dec

PERFORATIONS OR SCREEN:

104	4	5	3/8 Std Length

60

WELL DRILLER'S STATEMENT

This well was drilled under my supervision and this report is true to the best of my knowledge and belief.

Wheeler Co.

FLOURIDE WELL

Paraiso Inc.

Paraiso Springs

WELL LOCATION

Paraiso Springs

Soledad

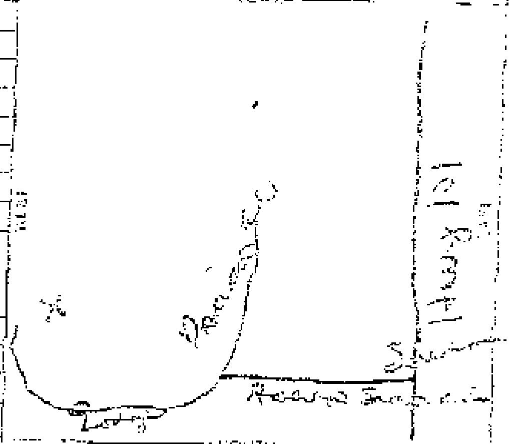
Monterey

Township Range Section

Latitude NORTH Longitude

LOCATION SKETCH

NORTH



Illustrate or Describe Distance of Well from Landmarks such as Roads, Buildings, Fences, Rivers, etc.
PLEASE BE ACCURATE & COMPLETE.

DRILLING METHOD Rotary

WATER LEVEL & YIELD OF COMPLETED WELL

DEPTH OF STATIC WATER LEVEL ______ (Ft.) & DATE MEASURED

ESTIMATED YIELD* ______ (GPM) & TEST TYPE

TEST LENGTH ______ (Hrs.) TOTAL DRAWDOWN ______

*May not be representative of a well's long-term yield.

CASING(S)

INTERNAL DIAMETER

GAUGE OR WALL THICKNESS

SLOT SIZE IF ANY

DEPTH FROM SURFACE

ANNULAR MATERIAL

A 1 1 5 7 9

WELL LOCATION

Page ___ Parcel ___

Township ___ Range ___ Section ___

Latitude ___ DEG. MIN. SEC. ___ NORTH ___ Longitude ___

LOCATION SKETCH

NORTH

WEST

EAST

SOUTH

Illustrate or Describe Distance of Well from Landmarks such as Roads, Buildings, Fences, Rivers, etc. PLEASE BE ACCURATE & COMPLETE

DRILLING METHOD ___ FLUID ___

WATER LEVEL & YIELD OF COMPLETED WELL

DEPTH OF STATIC WATER LEVEL ___ (Ft.) & DATE MEASURED ___

ESTIMATED YIELD * ___ (GPM) & TEST TYPE ___

TEST LENGTH ___ (Hrs.) TOTAL DRAWDOWN ___ (Ft.)

* May not be representative of a well's long-term yield.

TOTAL DEPTH OF BORING ___ (Feet)

TOTAL DEPTH OF COMPLETED WELL ___ (Feet)

	BORE-HOLE DIA. (Inches)	CASING (S): TYPE (✓)	MATERIAL / GRADE	INTERNAL DIAMETER (Inches)	GAUGE OR WALL THICKNESS	SLOT SIZE IF ANY (Inches)
[illegible]	10	[illegible]	[illegible]	5		
[illegible]	10	[illegible]	[illegible]	5		
[illegible]	10	[illegible]	[illegible]	5		
[illegible]	10	[illegible]	[illegible]	5		
[illegible]	10	[illegible]	[illegible]	5		
[illegible]	10	[illegible]	[illegible]	5		

DEPTH FROM SURFACE

CERTIFICATION STATEMENT

APPENDIX C

Laboratory Test Results

Table C-1
Summary of Laboratory Test Results

Sample No.	Depth (ft.)	Dry Density (pcf)	Water Content (%)	Pocket Penetrometer (tsf)	Swell (%)	Moisture Increase (%B)	Angle of Internal Friction	Unit Cohesion (pcf)
1-1	3.0-3.5	117.8	5.9	>4.5				
1-2	6.0-6.5	123.2	4.1	2.5				
1-3	11.0-11.5	117.7	8.8	3.0				
1-4	15.0-16.5		12.3					
1-5	20.0-21.5		14.1					
1-6	25.0-26.5		15.2					
1-7	31.0-32.5		17.8					
1-8	35.0-36.5		20.0					
1-9	38.5-40.0		18.0					
1-10	43.5-45.0		20.8					
2-1	3.0-3.5	109.5	2.3	>4.5				
2-2	5.5-6.0	113.1	1.8	>4.5				
2-3	10.0-11.5		1.8					
2-4	15.0-16.5		5.4					
2-5	20.0-21.5		10.4					
3-1	3.0-3.5	104.0	7.4	3.25				
3-2	6.0-6.5	112.2	6.2	>4.5				
3-3	11.0-11.5	106.3	9.1	1.25				
3-4	15.0-16.5		18.3					
3-5	20.0-21.0		19.3					
3-6	25.0-26.5		14.2					
3-7	28.5-30.0		18.0					
4-1	3.0-3.5	107.7	3.1	1.5				
4-2	6.0-6.5	118.6	3.4	>4.5				
4-3	10.5-11.0	115.1	3.2	>4.5				
4-4	15.0-16.5		3.3					
4-5	20.0-21.5		2.3					

Table C-1 Continued
Summary of Laboratory Test Results

Sample No.	Depth (ft.)	Dry Density (pcf)	Water Content (%)	Pocket Penetrometer (tsf)	Swell (%)	Moisture Increase (%B)	Angle of Internal Friction	Unit Cohesion (pcf)
5-1	3.0-3.5	106.2	2.6	>4.5				
5-2	6.0-6.5	118.8	15.1	>4.5				
5-3	11.0-11.5	112.2	14.0	2.25				
5-4	16.0-16.5	115.1	12.4	2.0				
5-5	19.5-21.0		14.0					
5-6	25.0-26.5		17.8					
5-7	30.0-31.5		16.8					
5-8	35.0-36.5		17.0					
5-9	38.5-40.0		17.8					
6-1	3.0-3.5	113.6	3.0					
6-2	5.5-6.0	116.2	3.2					
6-3	10.0-11.5		3.1					
6-4	15.0-16.5		3.5					
6-5	20.0-21.5		2.9					
7-1	3.0-3.5	108.7	7.7					
7-2	6.0-6.5	105.6	7.4					
7-3	11.0-11.5	93.0	24.0					
7-4	15.0-16.5		28.3					
7-5	20.0-21.5		17.7					
7-6	25.0-26.5		17.2					
7-7	30.0-31.5		17.2					
7-8	35.0-36.5		38.6					
7-9	40.0-41.5		23.5					
7-10	45.0-46.5		19.0					
7-11	48.5-50.0		17.9					
7-12	53.0-54.5		14.2					
8-1	3.0-3.5	112.3	1.4					

Table C-1 Continued
Summary of Laboratory Test Results

Sample No.	Depth (ft.)	Dry Density (pcf)	Water Content (%)	Pocket Penetrometer (tsf)	Swell (%)	Moisture Increase (%B)	Angle of Internal Friction	Unit Cohesion (pcf)
8-2	6.0-6.5	116.9	1.0					
8-3	10.0-11.5		0.9					
8-4	15.0-16.5		1.6					
8-5	20.0-21.5		1.2					
9-1	3.0-3.5	84.1	8.3					
9-2	6.0-6.5	108.7	15.2					
9-3	11.0-11.5	109.1	17.2					
9-4	16.0-16.5	100.3	16.8					
9-5	21.0-21.5	106.7	16.5					
9-6	25.0-26.5		19.6					
9-7	28.5-30.0		18.1					
10-1	2.5-3.0	119.4	8.1	>4.5				
10-2	5.5-6.0	112.7	9.1	>4.5				
10-3	9.5-10.5		0.6					
11-1	3.0-3.5		1.3					
11-2	6.0-6.5	95.5	3.0	>4.5				
11-3	11.0-11.5	104.6	6.6	>4.5				
11-4	15.0-16.5		20.9					
11-5	20.0-21.5		13.8					
11-6	25.0-26.5		11.8					
11-7	30.0-31.5		14.0					
11-8	35.0-36.5		18.9					
11-9	40.0-41.5		17.9					
11-10	45.0-46.5		19.1					
12-1	2.0-2.5	88.5	8.3					
12-2	5.0-6.5		2.0					

Table C-1 Continued
Summary of Laboratory Test Results

Sample No.	Depth (ft.)	Dry Density (pcf)	Water Content (%)	Pocket Penetrometer (tsf)	Swell (%)	Moisture Increase (%B)	Angle of Internal Friction	Unit Cohesion (pcf)
12-3	10.0-11.5		2.7					
12-4	15.0-15.5		2.3					
13-1	3.0-3.5	105.2	3.1	>4.5				
13-2	6.0-6.5	102.6	3.4					
13-3	11.0-11.5	101.6	16.7					
13-4	15.0-16.5		20.7					
13-5	20.0-21.5		27.7					
13-6	25.0-26.5		17.6					
13-7	35.0-36.5		19.3					
13-8	40.0-41.5		21.9					
13-9	45.0-46.5		18.9					
13-10	48.5-50.0		11.8					
14-1	2.5-3.0	125.9	5.7					
14-2	5.0-6.0		2.9					
14-3	10.0-11.5		1.9					
14-4	15.0-16.5		6.0					
14-5	20.0-21.5		1.9					
14-6	25.0-26.5		2.7					
15-1	5.5-6.0	93.3	11.4	3.0				
15-2	6.0-6.5	76.8	33.9	3.0				
15-3	11.0-11.5	109.5	10.0	>4.5				
15-4	15.0-15.5		0.6					
15-5	18.0-18.7		3.5					
16-1	2.0-2.5	119.7	4.8	>4.5				
16-2	5.0-5.5		1.3					
16-3	10.0-11.0		5.4					

Table C-1 Continued
Summary of Laboratory Test Results

Sample No.	Depth (ft.)	Dry Density (pcf)	Water Content (%)	Pocket Penetrometer (tsf)	Swell (%)	Moisture Increase (%B)	Angle of Internal Friction	Unit Cohesion (pcf)
16-4	15.0-16.5		3.2					
17-1	3.0-3.5	112.8	3.8	>4.5				
17-2	6.0-6.5	91.2	20.8					
17-3	11.0-11.5	101.1	1.3					
17-4	16.0-16.5		1.3					
17-5	20.0-21.5		2.1					
17-6	25.0-26.5		1.8					
17-7	30.0-31.5		2.5					
17-8	35.0-36.5		9.9					
17-9	40.0-41.5		14.8					
17-10	45.0-46.5		17.9					
17-11	48.5-50.0		12.6					
18-1	2.0-2.5	97.5	5.1					
18-2	5.0-5.5		2.3					
18-3	10.5-11.0		1.2					
19-1	3.0-3.5	103.3	2.4					
19-2	6.0-6.5	100.8	2.9	3.0				
19-3	16.0-16.5	101.2	3.1	3.25				
19-4	20.0-21.5		1.8					
19-5	25.0-26.5		4.7					
19-6	30.0-31.5		4.1					
19-7	35.0-36.5		3.5					
19-8	40.0-41.5		3.8					
19-9	45.0-46.5		3.0					
19-10	50.0-51.5		2.9					
19-11	55.0-56.5		8.9					
19-12	58.5-60.0		10.0					

Table C-1 Continued
Summary of Laboratory Test Results

Sample No.	Depth (ft.)	Dry Density (pcf)	Water Content (%)	Pocket Penetrometer (tsf)	Swell (%)	Moisture Increase (%B)	Angle of Internal Friction	Unit Cohesion (pcf)
20-1	2.0-2.5	111.7	5.9					
20-2	5.0-6.0		4.3					
20-3	10.0-11.0		4.0					
20-4	15.0-16.5		3.8					
21-1	3.0-3.5	146.6	1.4					
21-2	6.0-6.5		1.3					
21-3	11.0-11.5	113.9	6.1	>4.5				
21-4	15.0-16.5		5.0					
21-5	20.0-21.5		3.6					
21-6	23.5-24.0		2.4					
22-1	2.5-3.0	118.0	6.1	>4.5				
22-2	5.0-6.0		2.0					
22-3	10-10.5		3.2					
23-1	3.0-3.5	107.3	9.8	>4.5				
23-2	6.0-6.5	115.0	7.8	>4.5				
23-3	11.0-11.5	117.1	11.5					
23-4	15.0-16.5		19.4					
23-5	20.0-21.5		11.9					
23-6	25.0-26.5		12.7					
24-1	3.0-3.5		1.8					
24-2	5.0-6.5		2.0					
24-3	10.0-11.5		1.6					
24-4	15.0-16.5		1.5					
24-5	20.0-21.5		2.1					
25-1	3.0-3.5	102.9	2.5	4.0				

Table C-1 Continued
Summary of Laboratory Test Results

Sample No.	Depth (ft.)	Dry Density (pcf)	Water Content (%)	Pocket Penetrometer (tsf)	Swell (%)	Moisture Increase (%B)	Angle of Internal Friction	Unit Cohesion (pcf)
25-2	5.0-6.5		1.8					
25-3	10.0-11.5		0.8					
25-4	15.0-16.5		2.0					
25-5	20.0-21.5		2.0					
26-1	3.0-3.5	103.7	1.5					
26-2	5.0-6.5		0.9					
26-3	10.0-11.5		2.7					
26-4	15.0-16.5		2.7					
26-5	18.0-19.5		1.8					
27-1	2.5-3.0	107.3	7.4	>4.5				
27-2	5.0-6.5		3.1					
28-1	2.5-3.0	112.4	2.7					
28-2	5.0-5.5		2.7					
29-1	2.5-3.0	105.2	5.5					
29-2	5.0-6.5		7.6					

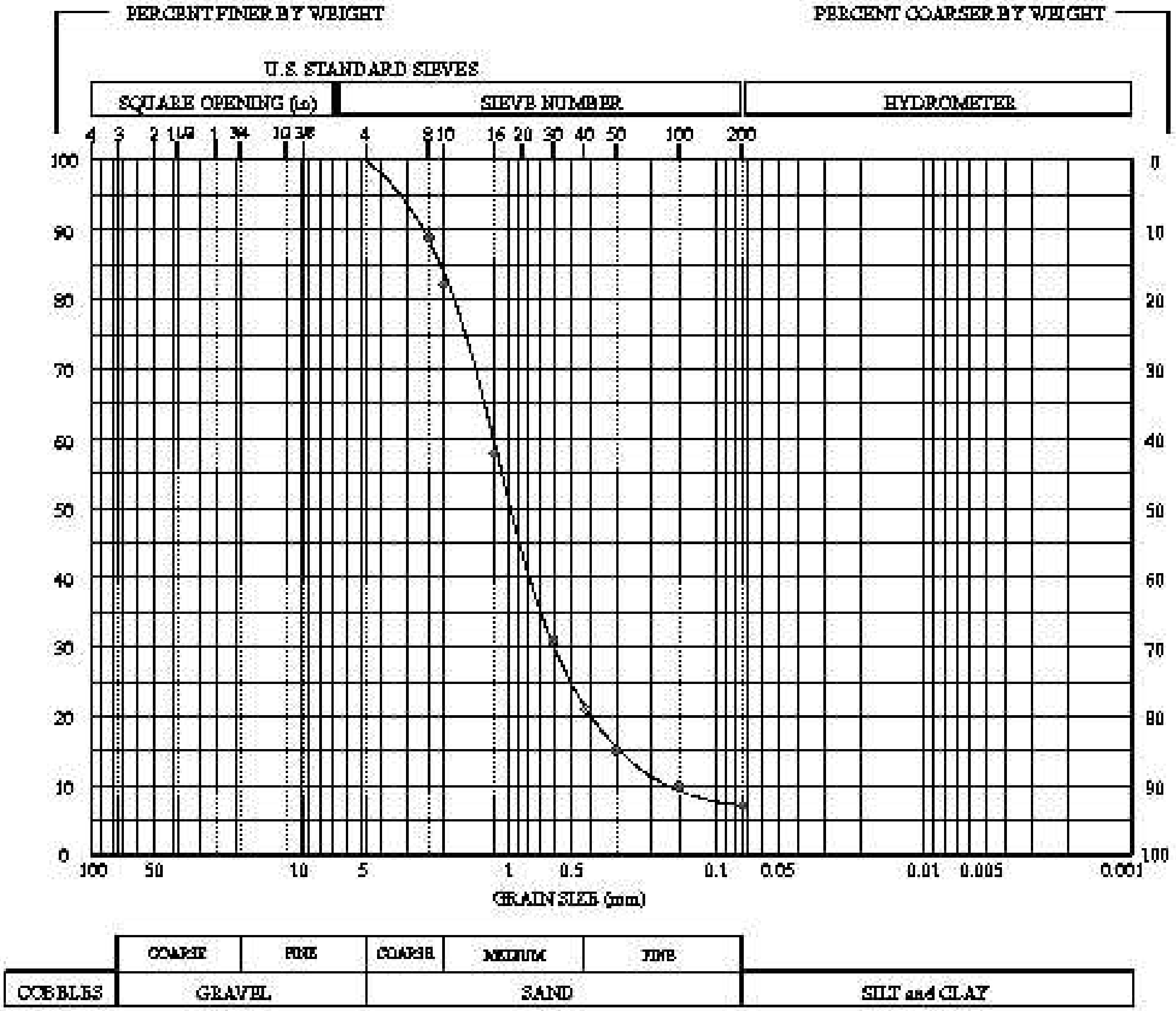
Summary of Atterberg Limits Test Results

Sample No.	Depth (ft.)	Liquid Limit	Plastic Limit	Plasticity Index
5-6	25.0-26.5	14	25	11
9-4	16.0-16.5	27	18	9
23-6	25.0-26.5	36	13	23
28-1	2.5-3.0	19	33	14
Bulk A	0.0-5.0	27	18	9
Bulk G	0.0-5.0	27	15	12

C7

GRAIN SIZE TEST RESULTS

PROJECT NAME	Pariaso Hot Springs					**PROJECT No.**	LSW-0337-01
DRILL HOLE No.	B-1	**DEPTH (ft)**	38.0 - 39.5'	**SAMPLE**	1-9	**DATE OF TEST**	9/13/04
DESCRIPTION OF SOIL	grey SAND W/CLAY: loose, saturated, 5-15% fines, fine to coarse sand						



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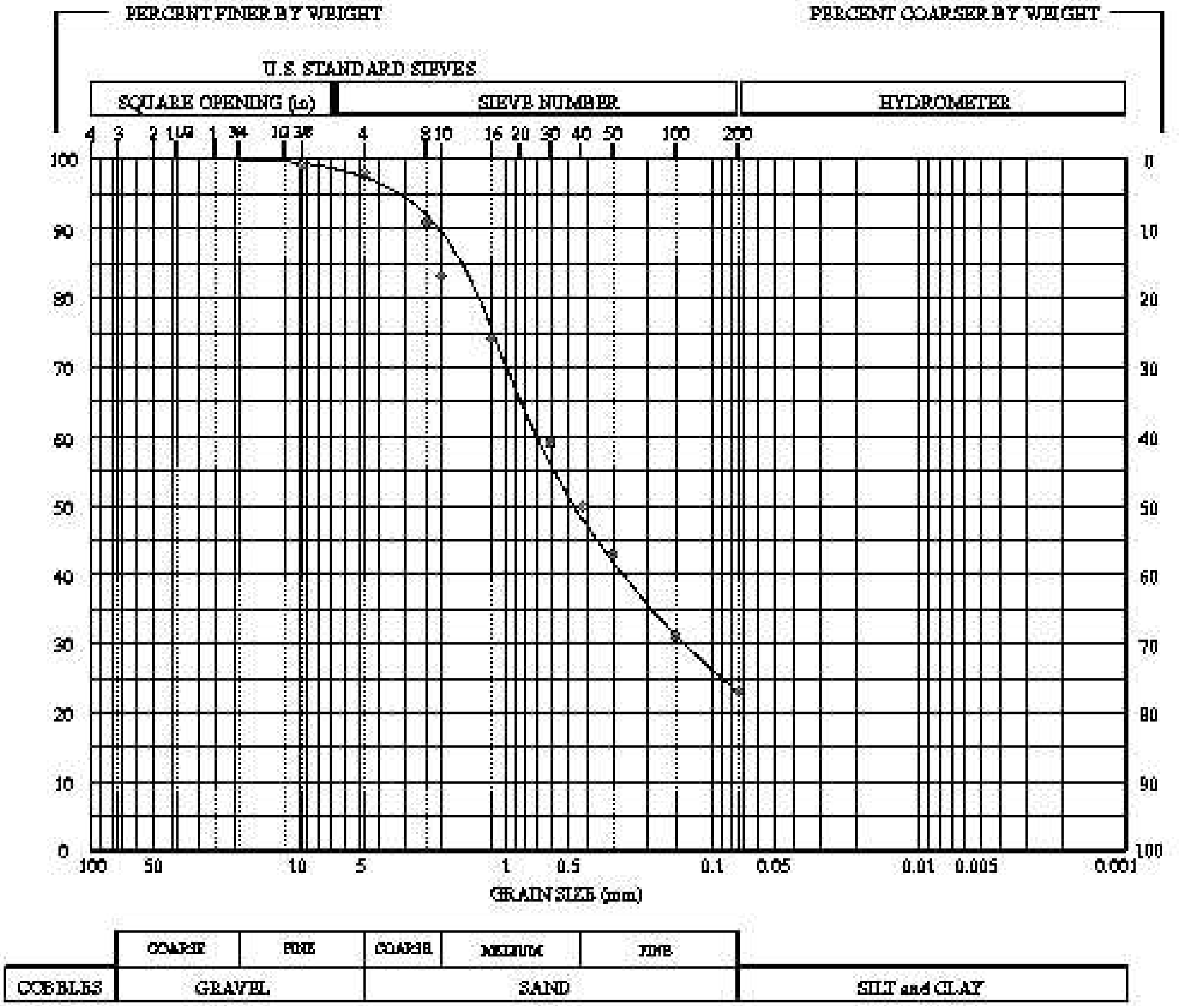
GRAIN SIZE TEST RESULTS

PROJECT NAME	Pariaso Hot Springs					**PROJECT No.**	LSW-0337-01
DRILL HOLE No.	B-3	**DEPTH (ft)**	30.0 - 31.5'	**SAMPLE**	3-5	**DATE OF TEST**	9/11/04
DESCRIPTION OF SOIL	dark grey well gradded SAND: loose, saturated, 20-30% fines, fine to coarse sand						



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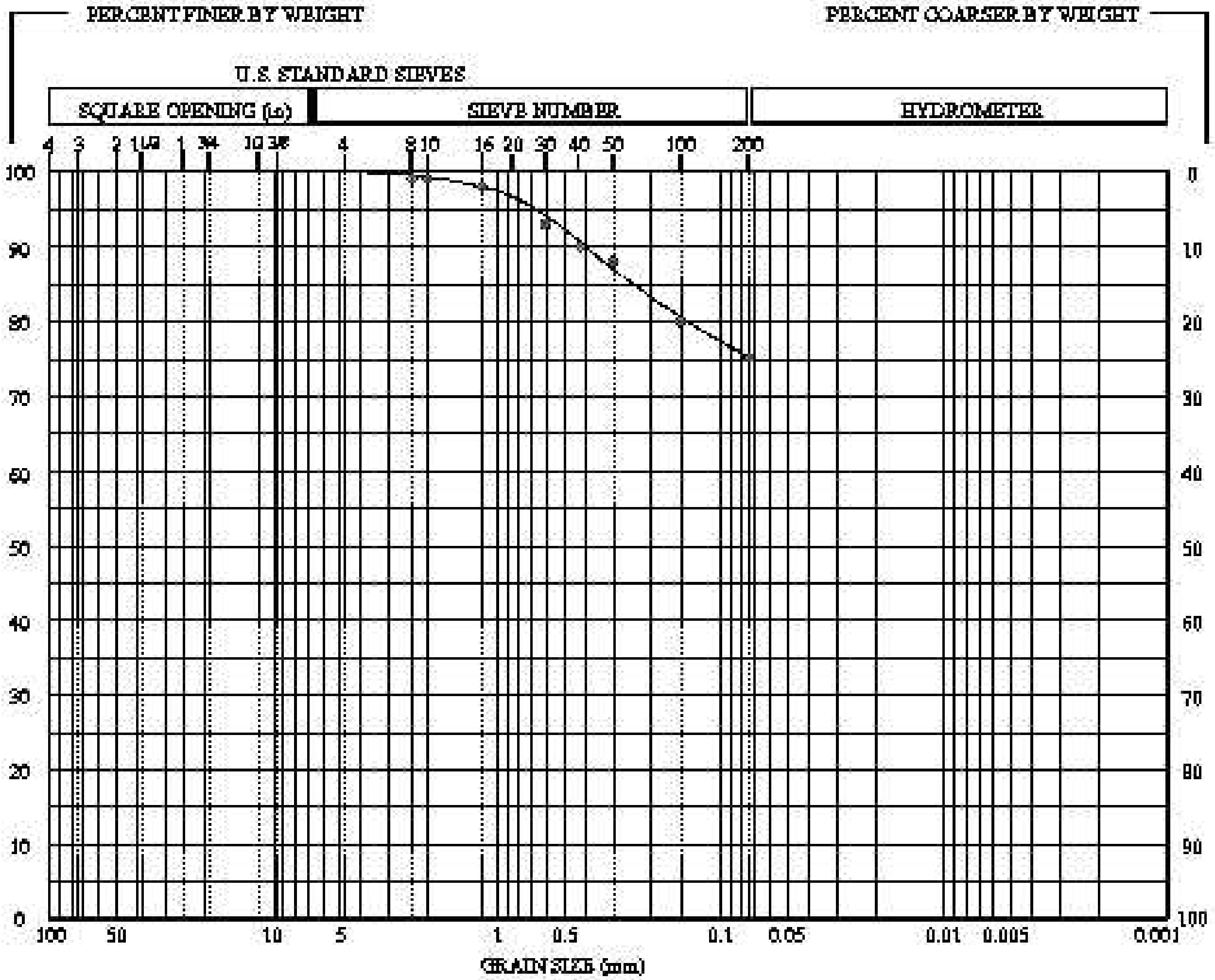
GRAIN SIZE TEST RESULTS

PROJECT NAME	Paraiso Hot Springs					**PROJECT No.**	LSW-0337-01
DRILL HOLE No.	B-5	**DEPTH (ft)**	26.0 - 26.5'	**SAMPLE**	5-6	**DATE OF TEST**	9/15/04
DESCRIPTION OF SOIL	Light Olive SANDY LEAN CLAY: stiff, very moist, 70-80% fines, fine to coarse sand						



	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	GRAVEL		SAND			SILT and CLAY

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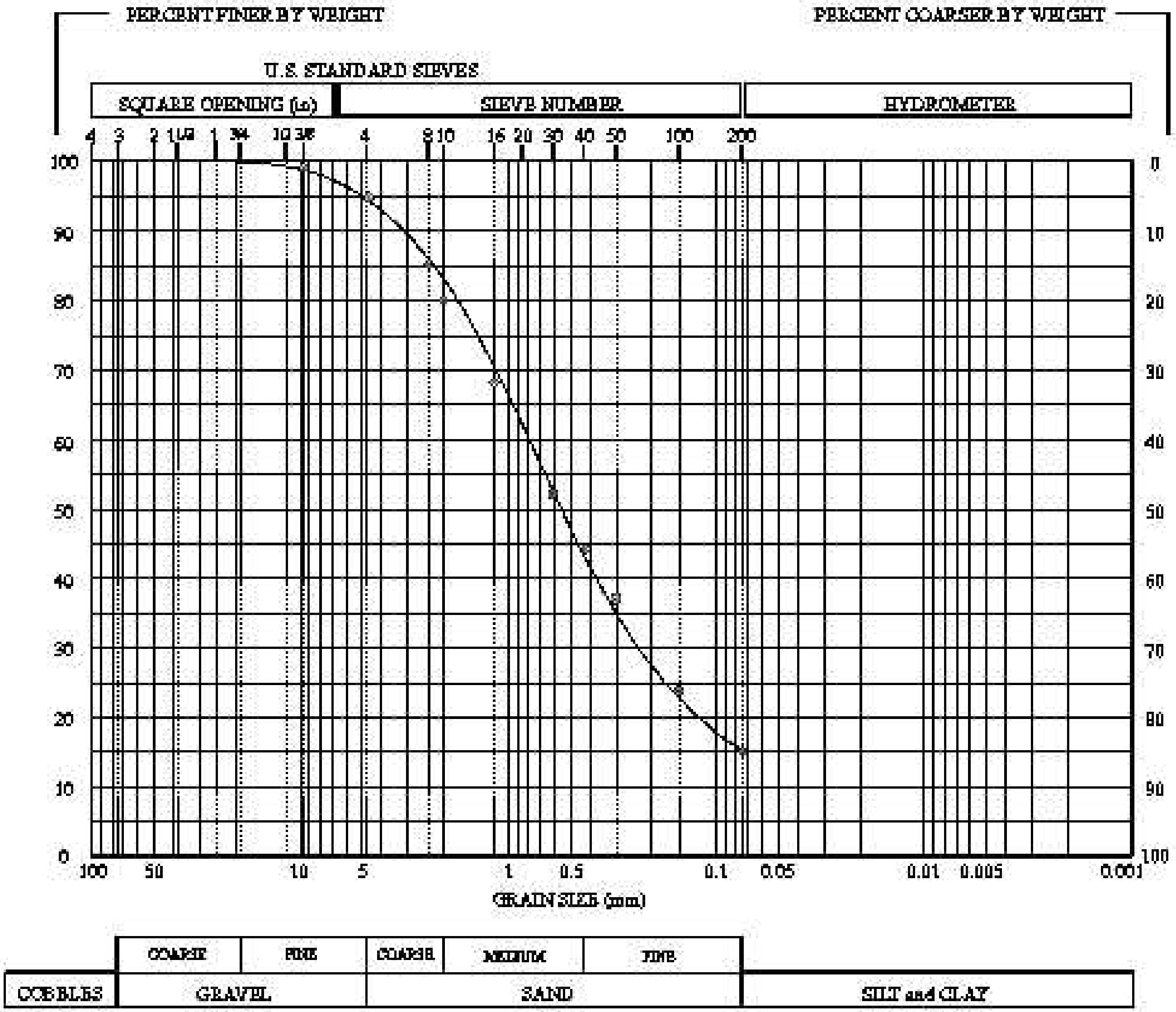
GRAIN SIZE TEST RESULTS

PROJECT NAME	Pariaso Hot Springs					**PROJECT No.**	LSW-0337-01
DRILL HOLE No.	B-7	**DEPTH (ft)**	15.0 - 16.5'	**SAMPLE**	7-4	**DATE OF TEST**	9/13/04
DESCRIPTION OF SOIL	Dark grey well gradded SAND: loose, saturated, fine to coarse sand, trace fine gravel, 15-25% fines.						



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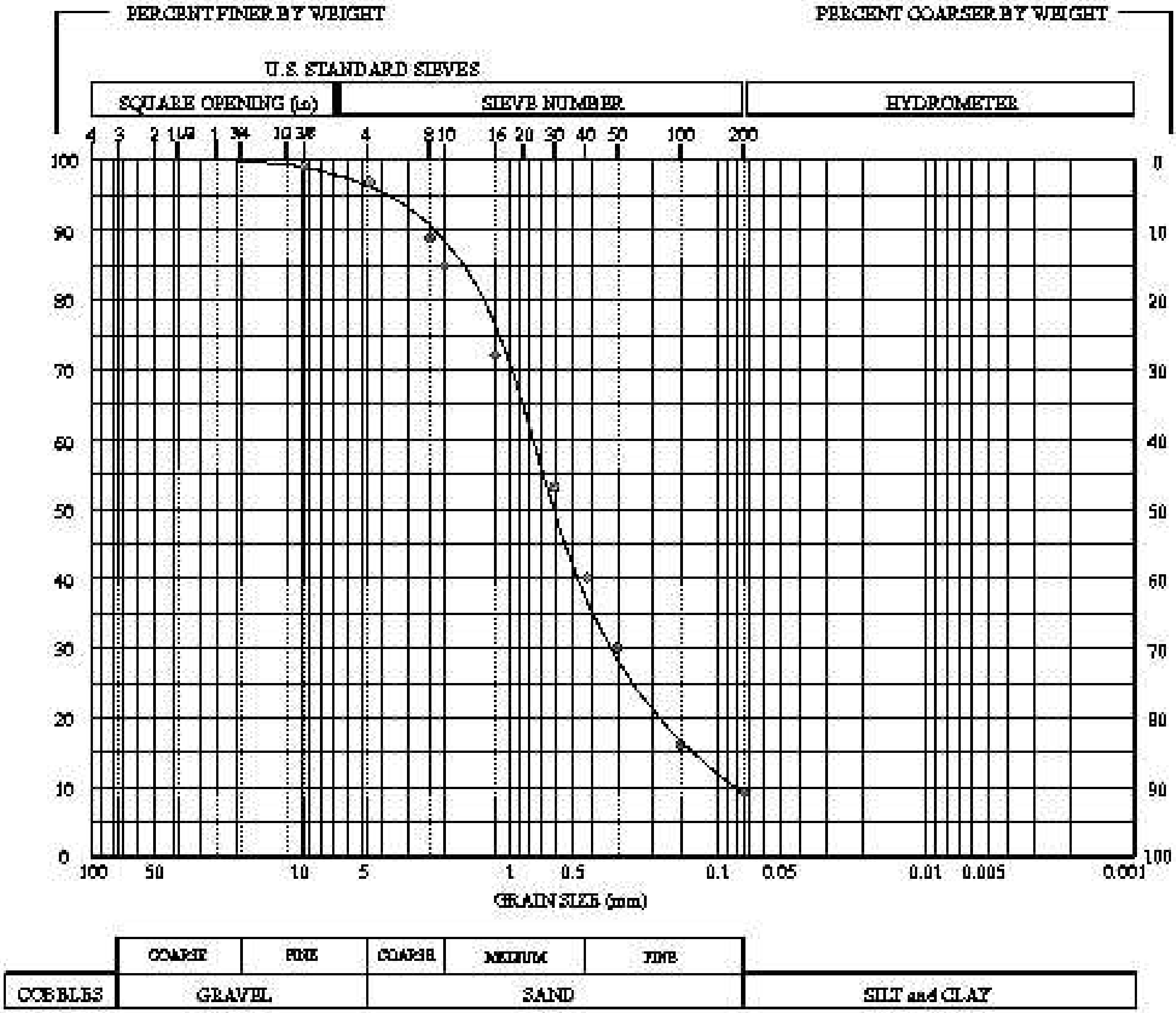
GRAIN SIZE TEST RESULTS

PROJECT NAME	Pariaso Hot Springs					PROJECT No.	LSW-0337-01
DRILL HOLE No.	B-7	DEPTH (ft)	20.0 - 21.5'	SAMPLE	7-5	DATE OF TEST	9/11/04
DESCRIPTION OF SOIL	Light grey well graded SAND: loose, saturated, fine to coarse sand, trace fine gravel, 5-15% fines.						



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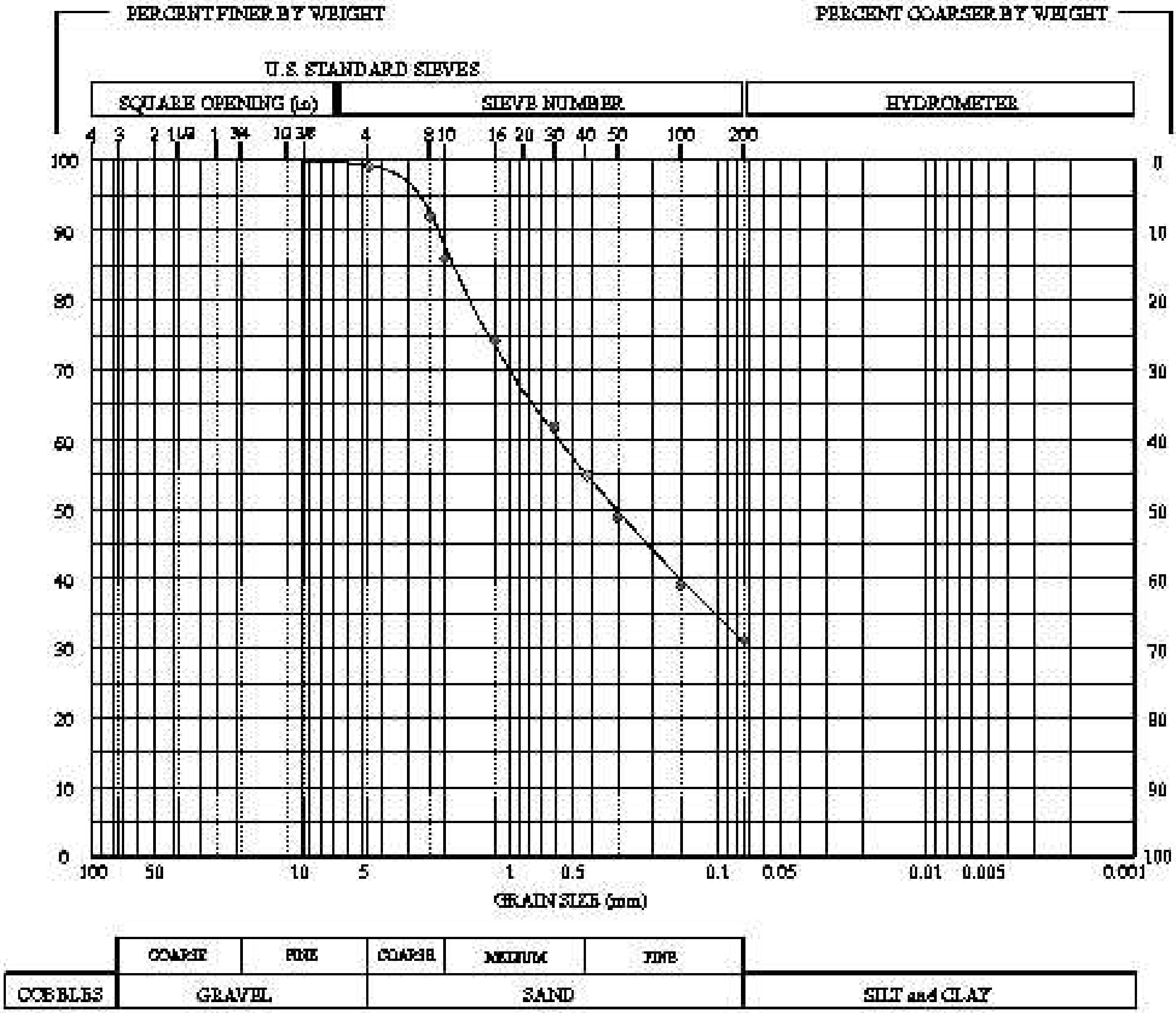
GRAIN SIZE TEST RESULTS

PROJECT NAME	Pariaso Hot Springs					**PROJECT No.**	LSW-0337-01
DRILL HOLE No.	B-9	**DEPTH (ft)**	15.0 - 16.5'	**SAMPLE**	S-4	**DATE OF TEST**	9/21/04
DESCRIPTION OF SOIL	Dark grey CLAYEY SAND: loose, saturated, 31% fines, fine to coarse sand, trace fine gravel						



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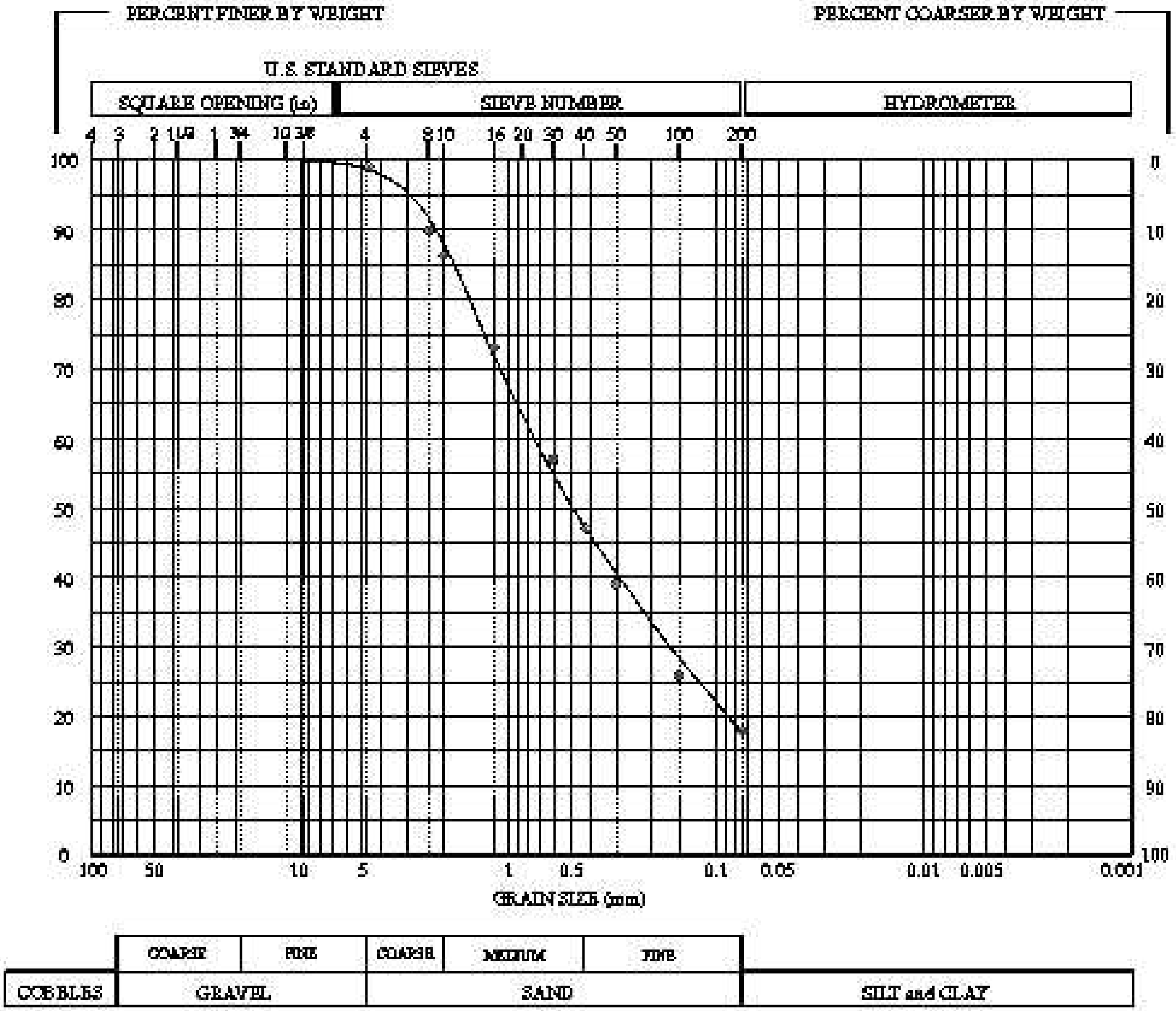
GRAIN SIZE TEST RESULTS

PROJECT NAME	Paraiso Hot Springs					**PROJECT No.**	LSW-0337-01
DRILL HOLE No.	B-9	**DEPTH (ft)**	28.5 - 30.0'	**SAMPLE**	9-7	**DATE OF TEST**	9/13/04
DESCRIPTION OF SOIL	Dark grey well gradded SAND: loose, saturated, 15-25% fines, fine to coarse sand, trace fine gravel						



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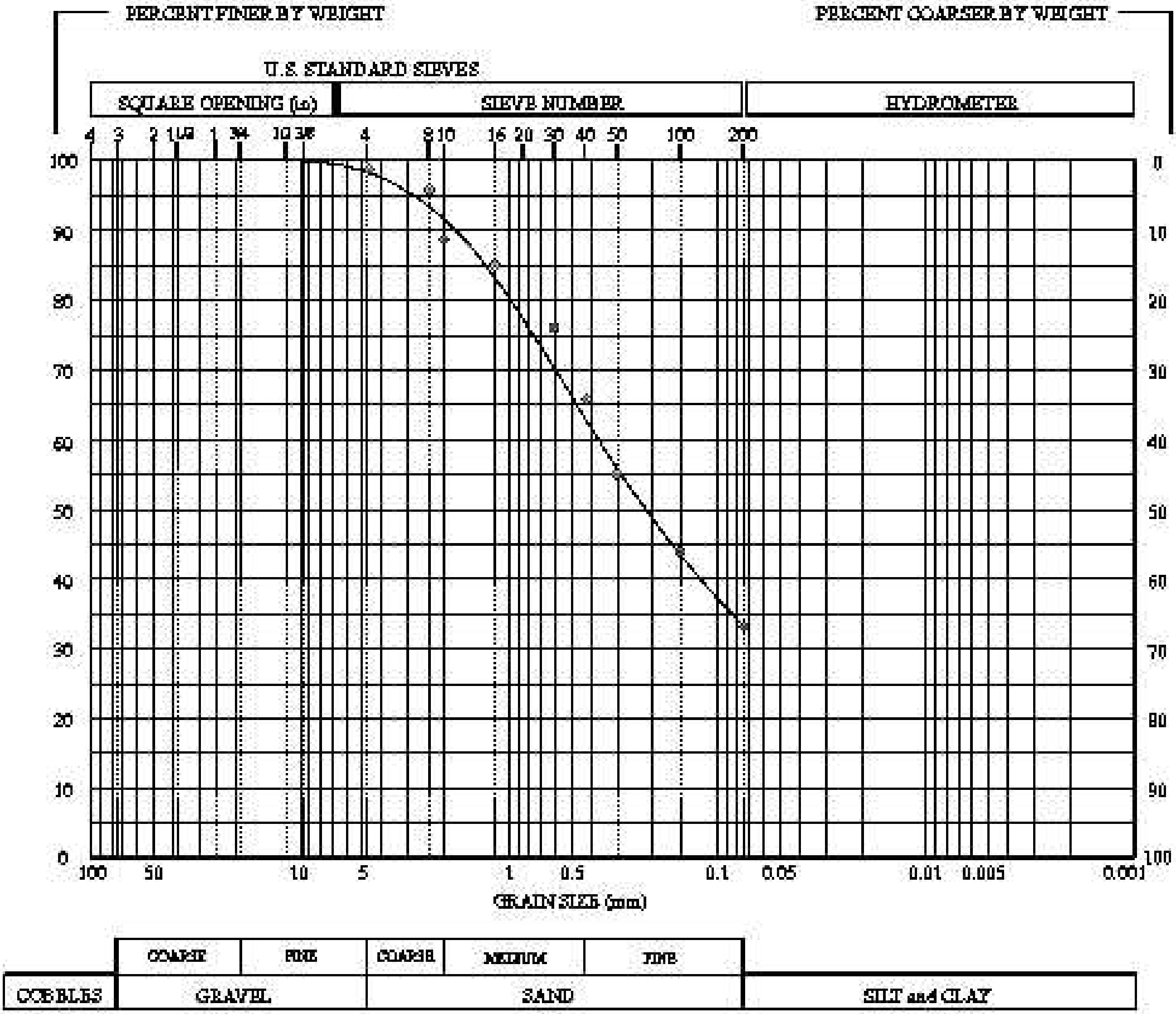
GRAIN SIZE TEST RESULTS

PROJECT NAME	Pariso Hot Springs					**PROJECT No.**	LSW-0337-01
DRILL HOLE No.	B-13	**DEPTH (ft)**	15.0-16.5'	**SAMPLE**	13-4	**DATE OF TEST**	9/13/04
DESCRIPTION OF SOIL	greyish olive SILTY SAND W/CLAY: loose, saturated, interbedded well graded sand, silt and clay, 25-35% fines, fine to coarse sand, trace fine gravel						



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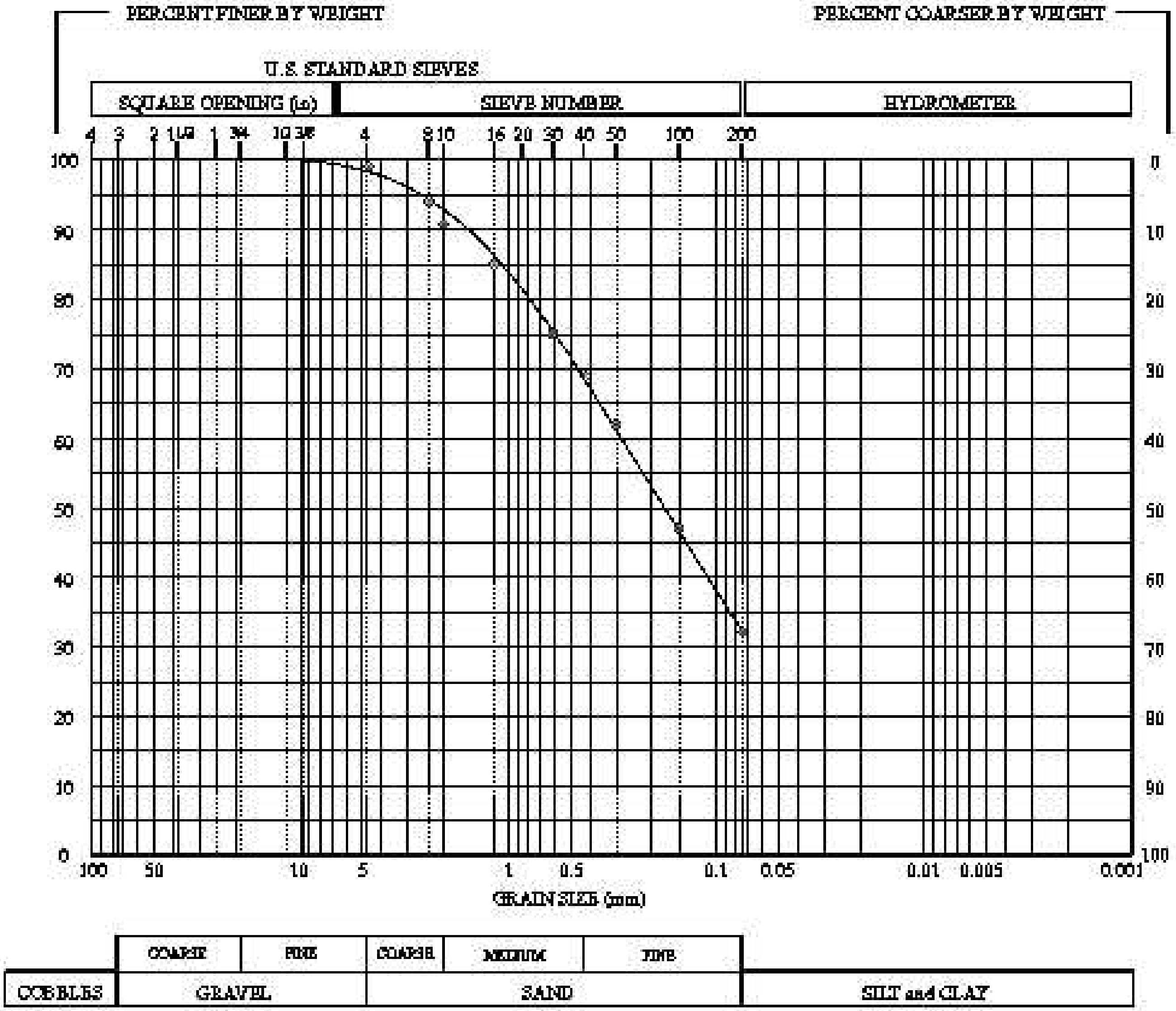
GRAIN SIZE TEST RESULTS

PROJECT NAME	Pariaso Hot Springs					**PROJECT No.**	LSW-0337-01
DRILL HOLE No.	B-13	**DEPTH (ft)**	20.0-21.5'	**SAMPLE**	13-5	**DATE OF TEST**	9/13/04
DESCRIPTION OF SOIL	greyish olive SILTY SAND W/CLAY: loose, saturated, interbedded well graded sand, silt and clay, 25-35% fines, fine to coarse sand, trace fine gravel						



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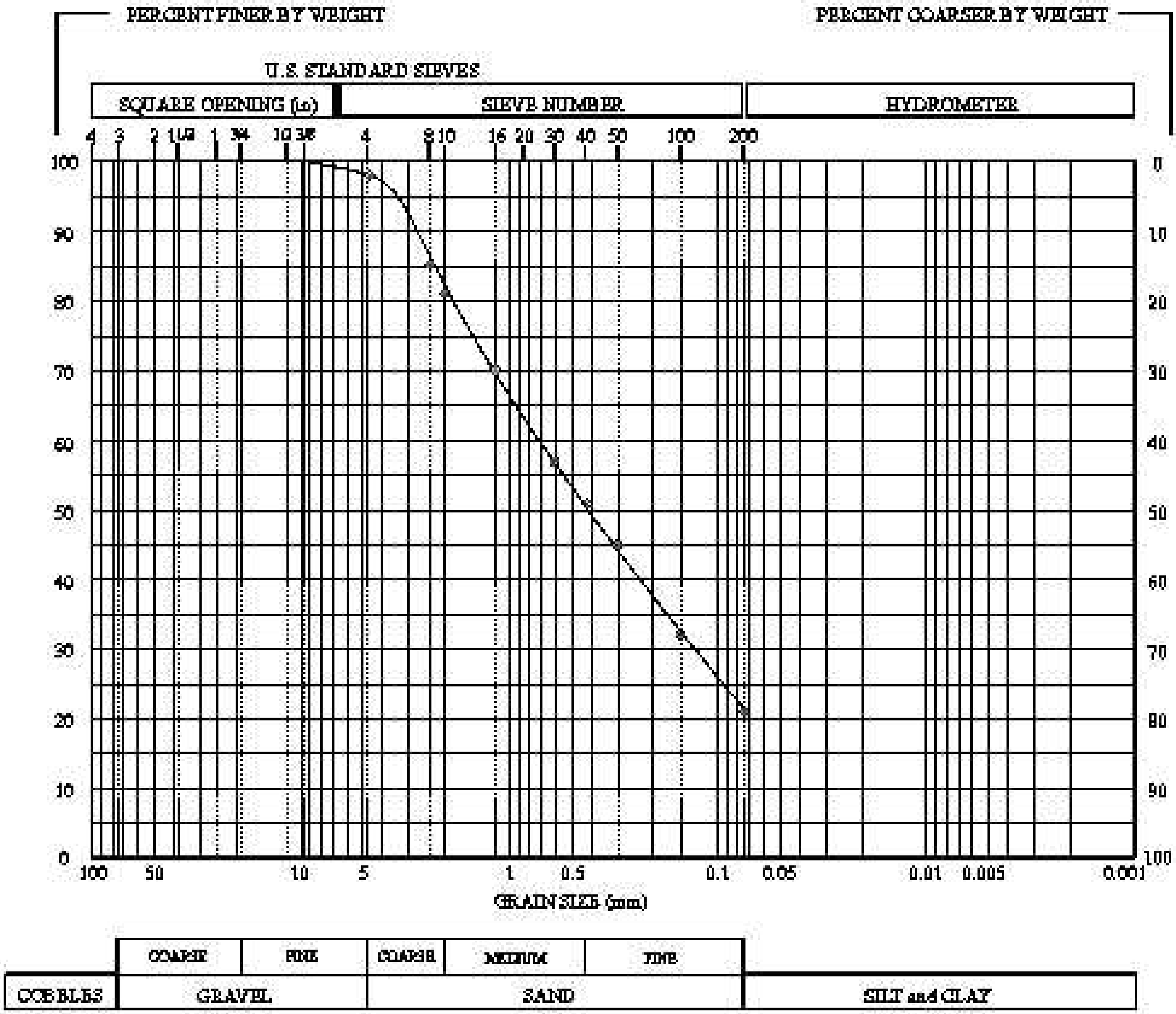
GRAIN SIZE TEST RESULTS

PROJECT NAME	Pariso Hot Springs					PROJECT No.	LSW-0337-01
DRILL HOLE No.	B-19	DEPTH (ft)	58.5-60.0'	SAMPLE	19-12	DATE OF TEST	9/11/04
DESCRIPTION OF SOIL	yellowish brown SILTY SAND: medium dense, moist, well graded, fine to coarse sand, 15-25% fines.						



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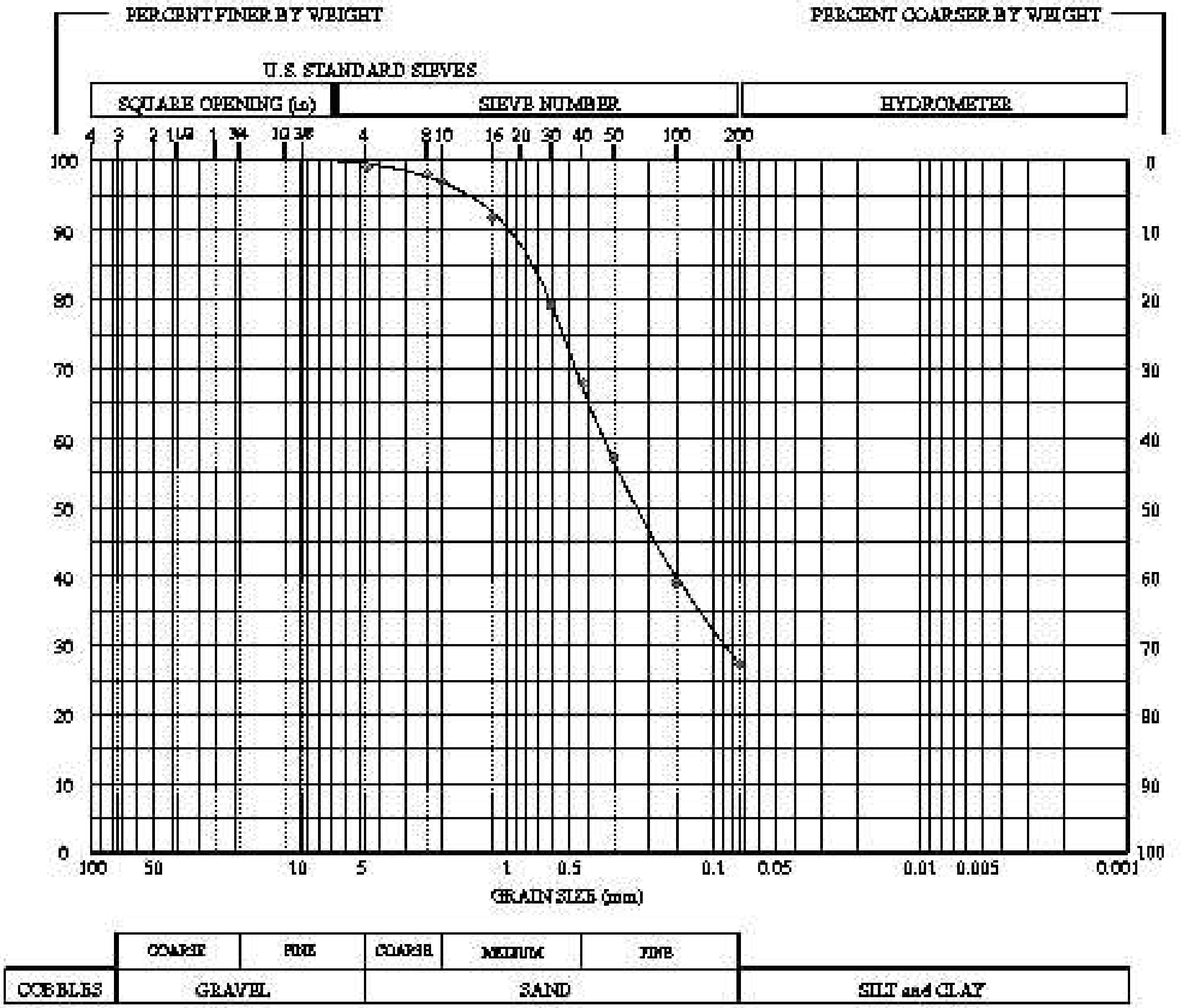
GRAIN SIZE TEST RESULTS

PROJECT NAME	Pariaso Hot Springs					PROJECT No.	LSW-0337-01
DRILL HOLE No.	B-23	DEPTH (ft)	15.0-16.5'	SAMPLE	23-4	DATE OF TEST	9/13/04
DESCRIPTION OF SOIL	reddish brown SILTY SAND: medium dense, saturated, fine to medium grained sand, 20-30% fines						



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APPENDIX D

SCALE OF ACCEPTABLE RISKS FROM GEOLOGIC HAZARDS

SCALE OF ACCEPTABLE RISKS FROM SEISMIC GEOLOGICAL HAZARDS

Level of Acceptable Risk	Kinds of Structure	Extra Project Cost Probably Required to Reduce Risk to an Acceptable Level
Extremely low [1]	Structures whose continued functioning is critical, or whose failure might be catastrophic: nuclear reactors, large dams, power intake systems, plants manufacturing or storing explosives or toxic materials.	No set percentage (whatever is required for maximum attainable safety)
Slightly higher than under extremely low level [1]	Structures whose use is critically needed after a disaster: important utility centers; hospitals; fire, police and emergency communication facilities; fire station; and critical transportation elements such as bridges and overpasses; also dams.	5 to 25 percent of project cost [2]
Lowest possible risk to occupants of the structure [3]	Structures of high occupancy, or whose use after disaster would be particularly convenient : schools, churches, theaters, large hotels, and other high rise buildings housing large numbers of people, other places normally attracting large concentrations of people, civic buildings such as fire stations, secondary utility structures, extremely large commercial enterprises, most roads, alternative or non-critical bridges and overpasses.	5 to 15 percent of project cost [4]
An ordinary level of risk to occupants of the structure [3,5]	The vast majority of structure: most commercial and industrial buildings, small hotels and apartment buildings, and single family residences.	1 to 2 percent of project cost, in most cases (2 to 10 percent of project in a minority of cases) [4]

[1] Failure of a single structure may affect substantial populations

[2] These additional percentages are based on the assumptions that the base cost is the total cost of the building or other facility when ready for occupancy. In addition, it is assumed that the structure would have been designed and built in accordance with current California practice. Moreover, the estimated additional cost presumes that structures in this acceptable risk category are to embody sufficient safety to remain functional following an earthquake.

[3] Failure of a single structure would affect primarily only the occupants.

[4] These assumptions are based on the assumption that the base cost is the total cost of the building or facility when ready for occupancy. In additions, it is assumed that the structures would have been designed and built in accordance with current California practice. Moreover the estimated additional cost presumes that structures in this acceptable-risk category are to be sufficiently safe to give reasonable assurance of preventing injury or loss of life during and following an earthquake, but otherwise not necessarily to remain functional.

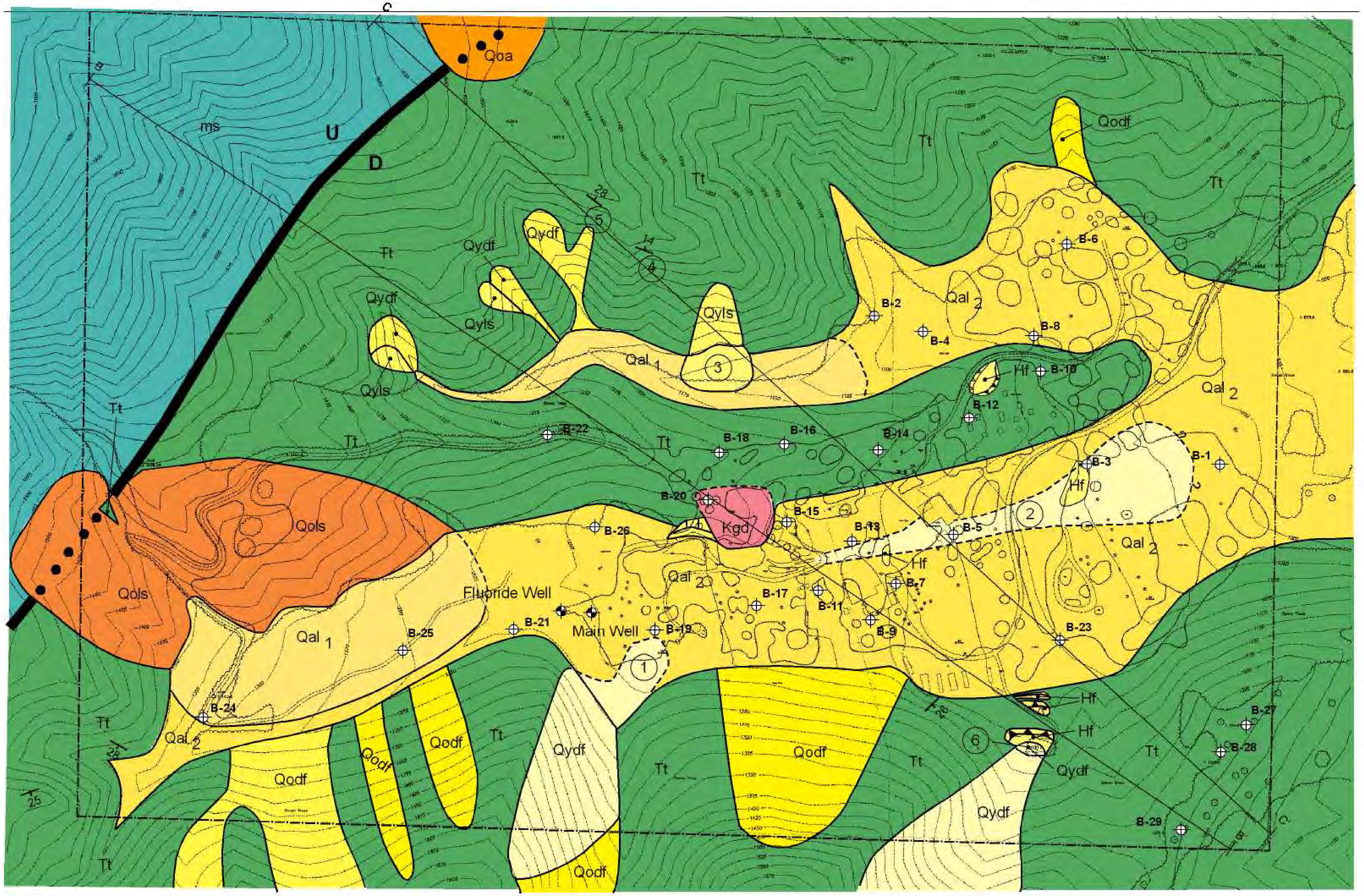
[5] “Ordinary risk”. Resist minor earthquakes without damage: resist moderate earthquakes without structural damage, but with some non-structural damage; resist major earthquakes of the intensity or severity of the strongest experienced in California, without collapse, but with some structural damage as well as non-structural damage. In most structures it is expected that structural damage, even in a major earthquake, could be limited to repairable damage. (Structural Engineers Association of California)

Source: Meeting the Earthquake, Joint Committee on Seismic Safety of the California Legislature, Jan. 1974, p.9.

SCLALE OF ACCEPTABLE RISKS FROM NON-SEISMIC GEOLOGIC HAZARD[6]

Risk Level	Structure Type	Risk Characteristics
Extremely low risks	Structures whose continued functioning is critical, or whose failure might be catastrophic: nuclear reactors, large dams, power intake systems, plants manufacturing or storing explosives or toxic materials	1. Failure affects substantial populations, risk equals nearly zero
Very low risks	Structures whose use is critically needed after a disaster: important utility centers; hospitals; fire, police and emergency communication facilities; fire station; and critical transportation elements such as bridges and overpasses; also dams.	1. Failure affects substantial populations. Risk slightly higher than 1 above.
Low risks	Structures of high occupancy, or whose use after disaster would be particularly convenient : schools, churches, theaters, large hotels, and other high rise buildings housing large numbers of people, other places normally attracting large concentrations of people, civic buildings such as fire stations, secondary utility structures, extremely large commercial enterprises, most roads, alternative or non-critical bridges and overpasses.	1. Failure of single structure would affect primarily only the occupants.
"Ordinary" risks	The vast majority of structure: most commercial and industrial buildings, small hotels and apartment buildings, and single family residences.	1. Failure only affects owners/occupants of a structure rather than a substantial population. 2. No significant potential for loss of life or serious physical injury. 3. Risk level is similar or comparable to other ordinary risks (including seismic risks) to citizens in a similar setting. 4. No collapse of structures; structural damage limited to repairable damage in most cases. This degree of damage is unlikely as a result of storms with a repeat time of 50 years or less.
Moderate risks	Fences, driveways, non-habitable structures, detached retaining walls, sanitary landfills, recreation areas and open space.	1. Structure is not occupied or occupied infrequently. 2. Low probability of physical injury. 3. Moderate probability of collapse.

[6] Non-seismic geologic hazards include flooding, landslides, erosion, wave runup and sinkhole collapse



EXPLANATION

Hf: Fill (Holocene): Fill deposits consisting of unconsolidated to semiconsolidated sand silt, clay, and trace gravel

Qyls: Landslide (Holocene): Recent landslide depostits, mostly occuring in the steeper slopes of the Tierra Redonda Formation (Tt)

Qydf: Debris flow (Holocene): Recent debris flow deposits, mostly occuring in the Tierra Redanda Formation (Tt)

Qodf: Debris flow (Holocene): Older debris flow deposits, mostly occuring in the Tierra Rodonda Formation (Tt)

Qal $_1$: Alluvium (Holocene): Unconsolidated sand, silt, gravels, and cobbles

Qal $_2$: Alluvium (Holocene): Unconsolidated sand, silt, and trace gravel

Qols: Landslide (Pleistocene): Older landslide deposits consisting of unconsolidated to semiconsolidated boulders and cobbles supported by a sand and clay matrix

Qoa: Alluvium (Pleistocene): Older alluvial deposits consisting of unconsolidated to semiconsolidated cobbles and boulders

Tt: Tierra Redonda Formation (Miocene): Marine sandstone, conglomerate, and some mudstone

Kgd: Granitic Basement Rock (Cretaceous): Hornblende granodiorite with phenocrysts of feldspar

ms: Sierra De Salinas Schist (Paleozoic ?): Biotite quartzofeldspathic schist

Contact: dashed were approximate, querried were unknown

U D **Fault:** dashed where approximate, dotted where concealed
U= upthrown side
D= downthrown side

Water well

20 **Strike and dip**

B-29 **Boring Location**

Cut ▽ and Fill ▼ Slope

Notes:
1: Debris flow deposit of March 1995 event
2: Flooding of March 1995 event
3: Recent landslide deposit
4: Location of slickenside surface trending N85W 66S, S80W 14
5: Location of slickenside surface trending N79W 40S, W10
6: Debris flow of March 1995 event

Qal 1, Qydf, Qyls, Hf, Qal 2, Qodf — Holocene
Qoa, Qols — Pleistocene
Holocene, Pleistocene — Quaternary

Tt — Miocene — Tertiary

Kgd — Cretaceous — Mesozoic

Ms — Paleozoic ?

N

0 200 ft

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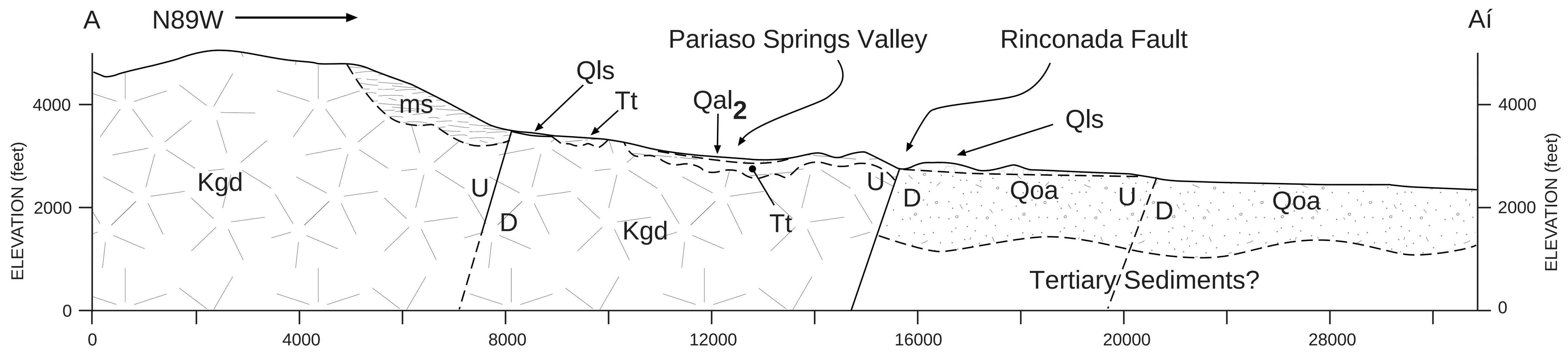
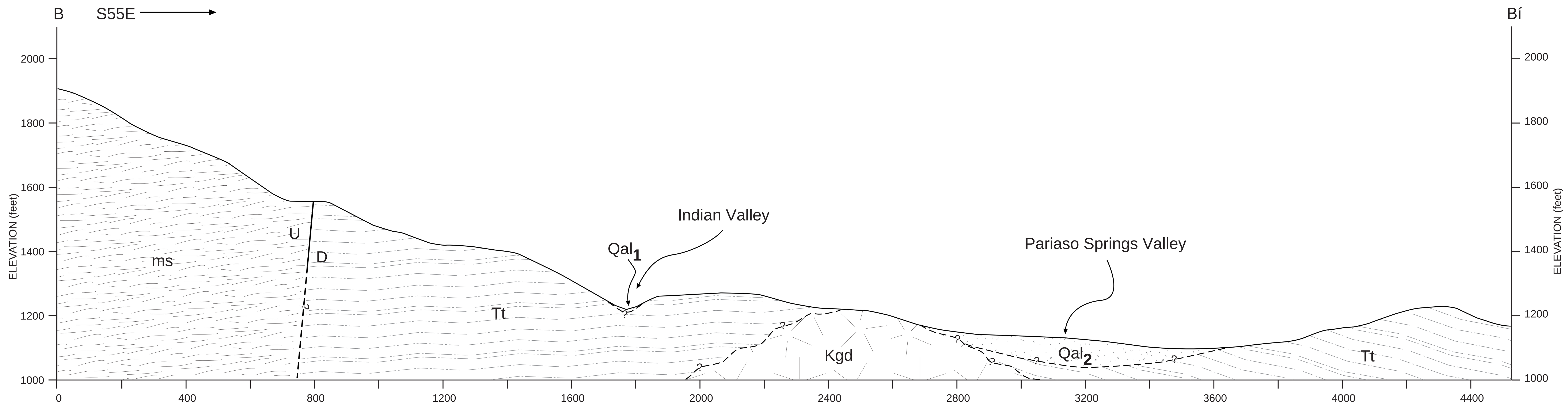
SITE GEOLOGIC MAP
Paraiso Hot Springs Resort
Paraiso Springs Road
Soledad/Greenfield Area, Monterey County, CA

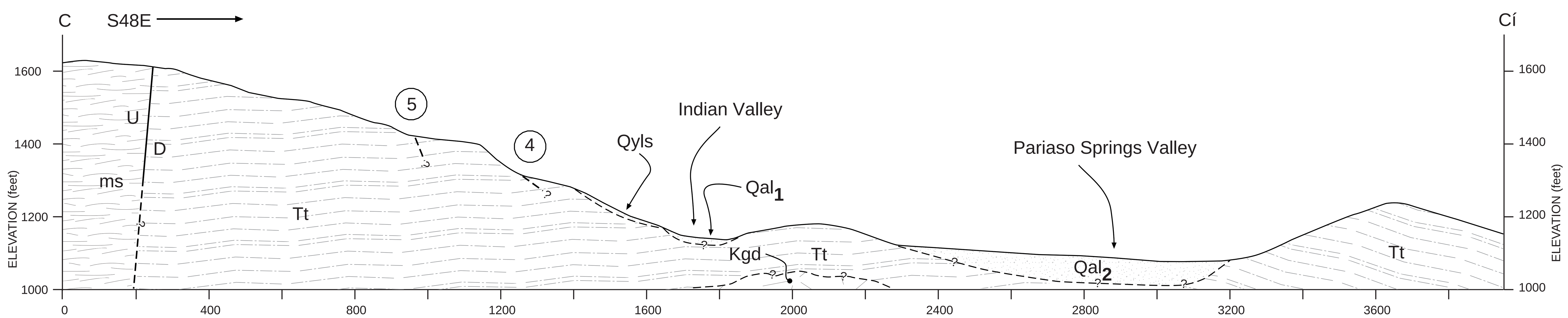
SHEET 1
PROJECT LSW-0337-01



Scale: 1:24,000 (See Figure 3)



Scale: 1:2,400 (See Sheet 1)



Scale: 1:2,400 (See Sheet 1)

EXPLANATION

Qyls: **Landslide (Holocene):** Recent landslide depostits, mostly occuring in the steeper slopes of the Tierra Redonda Formation (Tt)

Qal 1: Alluvium (Holocene): Unconsolidated sand, silt, gravels, and cobbles

Qal 2. Alluvium (Holocene): Unconsolidated sand, silt, and trace gravel

Qols: Landslide (Pleistocene): Older landslide deposits consisting of unconsolidated to semiconsolidated boulders and cobbles supported by a sand and clay matrix

Qoa: **Alluvium (Pleistocene):** Older alluvial deposits consisting of unconsolidated to semiconsolidated cobbles and boulders

Tt: **Tierra Redonda Formation (Miocene):** Marine sandstone, conglomerate, and some mudstone

Kgd: **Granitic Basement Rock (Cretaceous):** Hornblende granodiorite with phenocrysts of feldspar

ms: **Sierra De Salinas Schist (Paleozoic ?):** Biotite quartzofeldspathic schist

Geologic Contact: dashed were approximate, querried were unknown

U / D ?
Fault: dashed where approximate, dotted where concealed querried where unknown
U= upthrown side
D= downthrown side

C Cí
Geologic Cross Section

5 **Note**: Refers to location noted on Sheet 1

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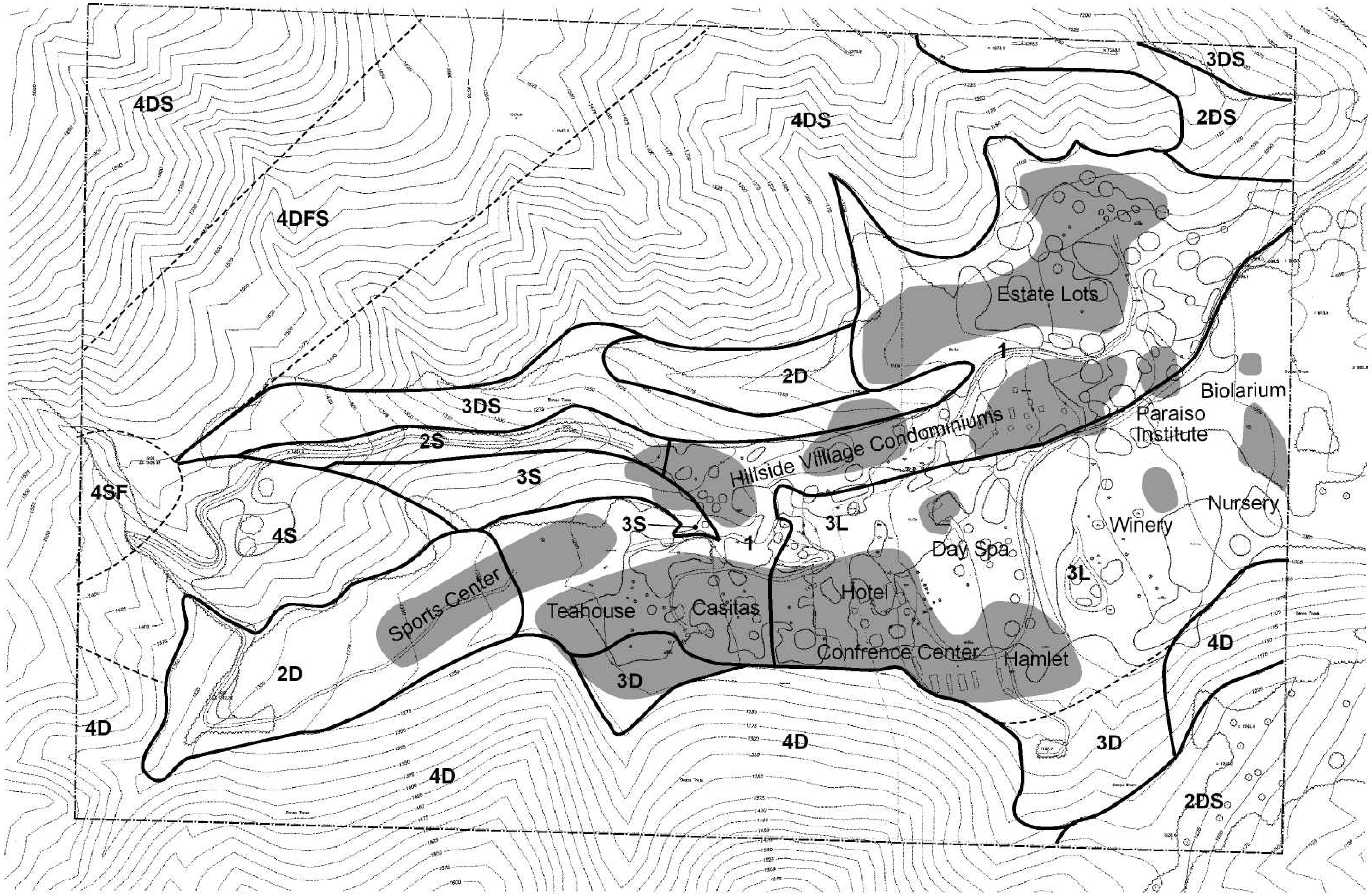
GEOLOGIC CROSS SECTIONS
Paraiso Hot Springs Resort
Paraiso Springs Road
Soledad/Greenfield Area, Monterey County, CA

SHEET
2
PROJECT
LSW-0337-01



EXPLANATION

Hazard Areas:

Area 1: Low geologic hazard potential

Area 2: Minor geologic hazard potential

Area 3: Moderate geologic hazard potential

Area 4: High geologic hazard potential

Hazard Descriptors:

F: Faulting

L: Liquefaction

D: Debris flow

S: Landslide

Proposed Developmet Areas

0 200 ft

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RELATIVE GEOLOGIC HAZARDS MAP
Paraiso Hot Springs Resort
Paraiso Springs Road
Soledad/Greenfield Area, Monterey County, CA

SHEET
3

PROJECT
LSW-0337-01

Paraiso Springs Resort: Existing Hydrologic and Hydraulic Site Conditions

PREPARED FOR: Thompson Holdings L.L.C.
(with Attachment)

PREPARED BY: David Von Rueden/CH2M HILL
Erika E. Powell/CH2M HILL
Kathy Rosinski/CH2M HILL

COPIES: Steve Ronzone/CH2M HILL
(with Attachment)

Kris Hansen/EDSA
(with Attachment)

Andrea Ramage/CH2M HILL

(with Attachment)

DATE: July 15, 2005

PROJECT NUMBER 327806.TK.03

This Memorandum provides a preliminary analysis of the current hydrologic and hydraulic conditions of the Paraiso Springs Resort (Project) Watershed and the potential for site flooding.

SUMMARY

Watershed Description

The Project is located south of Soledad and east of Greenfield, in Monterey County California. The Paraiso Springs drainage, which flows through the proposed development, begins on the eastern slopes of the Sierra de Salinas Mountains and in the westerly portion of the Arroyo Seco Watershed, travels northeasterly to the Arroyo Seco Valley floor, where flows are collected and enter the Arroyo Seco River. The Arroyo Seco River is a major tributary to the Salinas River.

The primary drainage basin, tributary to the Paraiso Springs channel, extends from the southwest, at elevation 2400 feet (NGVD), to the northeast project boundary, at elevation 1000 feet. The basin is approximately 1160 acres in size, and is surrounded by mostly undeveloped and rural agricultural land uses. The mountains and hillsides that are the primary sources of flows to the creek are covered by a mixture of native oak savannas, sycamore river valleys, grasslands, and scrub chaparral. The average slope of the hills to the southwest of the project site is 0.40 ft/ft. The average slope of the hills to the west of the project site is 0.36 ft/ft. Topographic contour patterns show that there are four points within the basin that collect and transfer flows from the higher areas of the basin to the existing stream.

Precipitation & Historical Flows

As discussed below, hydrologic data utilized in this memorandum was not compiled by the authors and could be confirmed or modified through direct measurement utilizing rainfall and stage gages present near or at the project site.

Average annual rainfall in the Project area is approximately 11-inches. Storms are few and infrequent and primarily occur in January and February. Two recent flood events occurred in January and in March of 1995, when almost 10-inches of rain fell in the watershed over five days. Using the Monterey County Rainfall Intensities Chart, the March 1995 storm was approximated to be between a 10- and 20-year event. Some damage to the pools and the road on the site was reported. This damage included a culvert whose capacity was greatly reduced by debris, brush and rocks.

Channel Characteristics

The main drainage channel through the Project site has an approximate width of 50 feet. The adjacent lands southerly of this channel are relatively flat and extend several hundred feet beyond the top of bank. The Soil Engineering Feasibility Report discusses existing soil conditions and the potential for landslides and debris production within the project area. This Report indicates that sediment and debris produced in the steeper portions of the drainage basin will migrate into the channel and will require management.

The channel slope upstream of the Project site (approximately 50 percent of its total length) is 0.25 ft/ft. The channel slope in the valley section of the channel (the length of the Project site) is approximately 0.112 ft/ft. The expected average channel velocity, within the Project site, is in the order of 27 ft/sec, at a full bank flow condition. This velocity, in combination with existing soil conditions, illustrates a potential for channel erosion during infrequent storm events.

Flood Zone

The Project site is shown on the Flood Insurance Rate Map (FIRM) for Monterey County, CA (Unincorporated Areas), Panel Number 060195 0350 D, dated January 30, 1984. This Map indicates that the Project Site is in Zone C – areas of minimal flooding. Although this indicates the Project site is not within a flood hazard area, FEMA requires all new construction to be built at the base flood elevation, which is 1-foot above the elevation of the top of bank, for undesignated flood hazard areas.

Paraiso Resort Site

The Project site, approximately 240 Acres, encompasses 21 percent of the total basin area. Only approximately 23 acres of the Project site is expected to contain impermeable surfaces. Because this is such as small percentage of the overall drainage basin at 2%, no significant increase in outflow from the basin is anticipated. However, because the project is to be built in the flatter lands that are tributary to the drainage channel, an impact to the current drainage patterns can be expected. Flows that are now delivered to the main channel via the four collection points, as discussed in Watershed Description, and overland sheet flow, will require collection and routing via culverts, piped storm drainage systems, or ditches with erosion protection. The appropriate sizing, locations and erosion protection measures for the drainage systems will be developed during subsequent Project design phases. Likewise, emergency surface drainage releases, for flow volumes beyond the design capacity of the drainage systems, will need to be provided to divert sheet flows around buildings.

The current, bankfull capacity of the primary drainage channel is approximately 4,000 cfs, excluding any existing culverts. It is estimated that approximately 400 cfs of runoff will be generated from the watershed, above the west boundary of the Project site, during a 1% (100-year) storm event. Therefore, the existing channel should have adequate capacity, with freeboard, to convey upstream flows through the site, provided that all roadway crossings of the creek provide a waterway opening that is comparable to the existing channel section. Also, erosion protection measures, such as bed stabilization, toe protection and bridge scour protection, should be implemented for the channel to preserve the channel cross section and minimize sedimentation downstream.

Conclusions

Subsequent design phases for the Project should consider the following:

- The Project is situated in an area tributary to a natural drainage channel and has the potential to impact the current site drainage patterns.
- The Project Site is not subject to flooding from a 1% (100-year) storm event, provided that the existing channel waterway cross section is maintained.
- Water surface elevations and velocities in the channel will need to be determined. Grading required for building pads and /or the foundations of all structures will be one (1) foot above the drainage channel banks. The grading or construction required for flood protection throughout the development area will be fully coordinated with the site's tree preservation requirements.
- There is a potential for significant sediment and debris production from the upper watershed. Debris basins upstream of the development should be implemented and a maintenance plan prepared.
- Efforts to control possible flooding should be considered, including:

  - diversion and/or containment of runoff above developed areas
  - measures to limit erosion of the main drainage channel
  - maintenance of the channel to prevent blockage
  - overland flow patterns should be established around proposed buildings, as part of the finish grading plan

METHODOLOGY AND ASSUMPTIONS

The preliminary hydrology data presented in this Memorandum were developed using a rough analysis of the SCS Curve Number method. Storm distributions for a duration of 24 hours were developed by SCS from U. S. National Weather Service data as typical design storms. In the SCS method, the intensity of rainfall varies considerably during the storm period. A Type 1 storm is used for areas in Central California. Runoff is affected by ground cover, soil type, and topography.

SUPPORTING DATA

Assumptions for soil type, ground cover and topography were based on cursory reviews of the Geology and Soil Engineering Feasibility Report for the Project, USGS Quadrangle maps, and field visits. A Watershed Map, based on a USGS Quadrangle Map, is attached.

Paraiso Springs Resort – Response to Hydrology and Hydraulic Analysis and Erosion Control Measures Review Comments

TO:	Thompson Holdings, LLC
COPIES:	David Von Rueden P.E./CH2M HILL
FROM:	Meabon Burns P.E./CH2M HILL Erika Powell P.E./CH2M HILL
DATE:	October 28, 2008
PROJECT NUMBER:	366335.03

REGISTERED PROFESSIONAL ENGINEER MEABON KYLE BURNS 71053 Exp 6/30/2009 CIVIL STATE OF CALIFORNIA

The purpose of this memorandum is to provide responses to review comments of Technical Memorandums titled Paraiso Springs Resort: Preliminary Hydrology and Hydraulic Analysis and Paraiso Springs Resort: Erosion Control Measures dated July 15, 2005. Comments were provided in a memorandum from Harvey Oslick, MS 1300 to Meryka Blumer, MS 1600 in a memorandum dated January 17, 2008. A copy of this memorandum is included in Attachment 1 for reference.

Additional Analysis to Support Response to Comments

An HEC-HMS (Hydrologic Engineering Center-Hydrologic Modeling System) model was developed to support the response to comments provided in this memorandum. HEC-HMS (version 3.1.0) is a hydrologic model developed and made available by the US Army Corps of Engineers. The key parameters of the analysis methodology are presented below:

- Unit system – U.S Customary
- Loss: – Soil Conservation Service (SCS) Curve Number (CN)
- Transform – Kinematic Wave
- Baseflow – None
- Routing: – Kinematic Wave
- Precipitation – SCS Storm
- Evapotranspiration – None
- Snowmelt – None

Analysis results are presented below within the response to comments.

Response to Comments

Information for Describing Existing Conditions/Setting

Hydrology and Hydraulic Analysis

1. Upon leaving the Project site, stormwater travels first through a natural ravine then through agricultural drainage ditches and culverts under road crossings (see

Attachment 2). The ditches are highly channelized, and are either located along natural drainage paths or adjacent to roads. The banks have been stabilized in some locations by the installation of sandbags. These drainage ditches are man-made, most likely by local property owners, and are characterized by steep, unvegetated side slopes. The level of maintenance for these incised channels is unknown. See Photos 1 through 3 in Attachment 3.

2. Maps showing the subbasin delineation for the Project watershed and Project site are included in Attachment 4. The Project watershed was delineated into nine subbasins. These subbasins were delineated because they either had distinct drainage characteristics or the flows collected at a location where specific project impacts could be identified, such as the potential for landslides or debris flows.

3. The HEC-HMS model facilitates a more rigorous and detailed analysis than the analysis that was conducted for the July 2005 Project evaluation and is appropriate for this application. The 10-year and 100-year event stormwater volumes for the entire watershed, using this more detailed hydrologic methodology as described above, were found to increase from 117.5 ac-ft (123.5 cfs) to 124.0 ac-ft (124.2 cfs) and 261.1 ac-ft (310.9 cfs) to 269.6 ac-ft (315.8 cfs) (see Supporting Data Tables/Figures), respectively. This increase in stormwater runoff of 6.5 ac-ft (0.7 cfs) for the 10-year storm and 8.5 ac-ft (4.9 cfs) for the 100-year storm translates to 5.5 percent and 3.3 percent, respectively, of the total runoff volume and 0.6 percent and 1.6 percent, respectively, of the peak discharge. This result is based on conservative assumptions regarding post-development conditions, such as new impervious area, overland flow roughness, and Soil Conservation Service Curve Numbers (SCS CN).

The approach to minimizing Project impacts due to stormwater runoff, as calculated above, will be to use low impact design (LID) methodologies. Specific LID techniques, often referred to as stormwater best management practices (BMPs), will be determined during the design process. For purposes of this preliminary analysis, the areas of "hardscape" shown on the Land Use Summary Table of the Project Tentative Map were evaluated for appropriate LID construction techniques. Project "hardscape" areas and related potential LID construction techniques are summarized below (CASQA, 2003). Actual BMPs and combinations of BMPs to be used will be evaluated during final design.

- Building footprints (7.22 acres)
  - Roof runoff controls
  - Site design and landscape planning
  - Alternative building materials

- Patios, Paths, and Driveways (5.99 acres)
  - Site design and landscape planning
  - Pervious paving
  - Vegetated swales
  - Vegetated buffer strip
  - Bioretention

- Parking and Roadways (9.98 acres)
  - Pervious paving
  - Vegetated swales
  - Vegetated buffer strip
  - Bioretention

As noted in the July 15, 2005 Memorandum, only 23.19 acres of the 1,160 acre Project watershed will be developed with "hardscape" features. Utilization of the LID techniques, as described above, is anticipated to limit the post-Project runoff from frequent storm events to virtually identical volumes as the pre-Project condition and to result in insignificant increases during the rare, infrequent events (i.e. 100 yr event).

It should also be noted that the Project site is underlain by predominantly sandy soils, as identified in the Project Geologic and Soil Engineering Report, prepared by Landset Engineers, Inc. and dated December 2004. This soil condition should be very compatible with the proposed LID construction techniques.

It should also be noted that the Project stormwater features will be designed to ensure that the pre-project 10-year event flow will continue to reach the drainage channel downstream of the Project site, post-development.

4. A U.S. Geological Survey (USGS) Water-Resources Investigations Report (USGS, 1994), which was used for the previous analysis and is included in Attachment 5, and private, unpublished information indicate that the mean annual precipitation (MAP) is approximately 11 inches east of the Project site. However, the elevation across the entire watershed ranges from 1,000 to approximately 2,400 feet. Therefore, it is likely that the MAP varies, potentially significantly, across the watershed with elevation. Figure 2.3 of the Monterey County Water Resources Agency's (MCWRA) *Water Resources Data Report: Water Year 1994-1995* presents MAP for Monterey County (MCWRA, 2007). The project site location was approximated on this map to find the MAP, see Attachment 6. MAP for the Project site was found to be 23 inches. This MAP was verified by data collected by a rain gage from 1950 to 1982. The Paloma Station is located approximately 9 miles southeast of the Project site (Longitude 121.500 W, Latitude 36.350 N) at an elevation 1,835 feet. The data collected at this station indicates that the MAP is 23.25 inches for the period of record (DWR, 1983).

   Based on available data, the MAP could range from 11 to 23 inches across the entire watershed. To be conservative, a MAP of 23 inches was used for the purpose of this analysis.

   The MAP, 23 inches, was used to calculate precipitation depth for the 10-year and 100-year storms for a duration of 24 hours. Precipitation depth was calculated using the Santa Clara County's Return Period-Duration-Specific (TDS) Regional Equation, which establishes a relationship between precipitation depth and MAP for various storm return periods. This equation was developed based on the three-day December 1995 rainfall event that is still considered to be the storm of record for Northern California. (Santa Clara County, 2007)

5. Detention ponds are not included in the Project, because the LID stormwater mitigation methodologies described above will be implemented. Debris basins, as recommended in

the previously referenced Geologic and Soil Engineering Report, would be implemented and located at the point of concentration for Subbasins N-1 (see Photo 4 in Attachment 3) and N-2, located in Indian Valley along the Northern edge of the Project site, and Subbasins S-1, S-2, and S-3, located along the southern edge of the Project site (see Attachment 4). These debris basins would intercept debris flows/slides from the identified Subbasins, above the developed areas of the Project. They will be located immediately adjacent to Project features and incorporated into the site grading footprint for the overall Project. The debris basins are expected to include a series of two-to-four small soil and rock checkdams, approximately three-feet tall, constructed at the low flow line of the natural drainage feature. Minimal excavation behind the checkdams is planned and no additional trees would be removed for construction. The debris basins would be constructed adjacent to Project roadways, parking lots or maintenance paths to facilitate inspection and maintenance.

Although Subbasin V-1 was identified as a potential site for debris flows, it is not anticipated that a debris basin will be needed at the point of concentration for this basin. The drainage channel was found to be well defined and relatively clear of debris at this location. Rocks that were present were in general no greater than approximately 24 inches in diameter. Debris flowing through the main drainage channel did cause flooding on site during a storm in 1995. However, this was due to the debris blocking flow through an existing culvert located upstream of the hot springs pools (see Photo 8 in Appendix 3). The culverts at this location and the culverts located just upstream of the property line (see Photo 1 in Appendix 3) will be removed as part of the Project to restore the drainage channel capacity. Bridges will be installed to allow vehicular and pedestrian access across the drainage channel. The bridges are expected to be single-span structures, with abutments on each bank of the stream. Stream banks would be reconstructed as part of the bridge construction and lined with rock riprap for scour protection immediately adjacent to the abutments. Small storm drain outfalls would be located within the bridge and rock riprap footprints.

Erosion Control Measures

Because the intention is to implement stormwater BMPs to ensure that post-development stormwater flows in excess of pre-development conditions for a 100-year storm event do not leave the Project site, aggradation of the channel downstream of the project site, is not expected. Based on field observations, most of the sediment that travels from the steeper areas of the watershed to the valley of the watershed during annual rainfall events, is naturally deposited on the flatter areas of the watershed (i.e., within the Project site). Sediment that currently feeds the channel downstream of the Paraiso Springs Resort Project site, during more frequent or annual rainfall events, is contributed by the adjacent floodplain below the Project site through sheetflow. Onsite debris basins will be designed to retain large-particle sediment and other debris, but not suspended sediment. Passage of suspended sediment will also be aided by the removal of existing culverts and the restoration of natural drainage channel conditions as part of the Project. Therefore, it is expected that nutrients that are necessary for the health of the channel, downstream of the project site, will continue to be replenished.

Any points where stormwater flows collect and it is necessary to discharge into the channel will be designed with appropriate and primarily natural erosion protection measures, such as rock slope protection and vegetation.

Regulatory Background Information

Comment noted regarding compliance with the requirements of Monterey County Ordinance Chapter 16.2 Erosion Control and Ordinance Chapter 19.10 Design and Standard Improvements, paragraph 19.10.050, Drainage. Analysis and design efforts for the Project will comply with County policies in place when construction documents are developed. Mitigation measures, such as permeable pavements and vegetated drainage swales, and stormwater collection systems will be designed to ensure that stormwater drainage volume and peak flows do not increase from existing conditions, as a result of the Project.

Comment noted regarding the anticipated new statewide NPDES Construction General Permit. Project construction documents will comply with the most current General Permit.

Analytical Methodology and Significance Thresholds

The proposed project will not alter the course of flow through the main drainage channel, will not significantly alter existing drainage patterns, and will not significantly increase the rate of runoff. Minimal impacts to peak flow discharge and flow volume will be mitigated onsite to ensure that no downstream impacts will result directly from the Project. Downstream capacity will not be exceeded due to the Project, flow in excess of current flows will be allowed to infiltrate on site.

Pre- and post-Project stormwater drainage volumes for 10-year and 100-year storm events are summarized under Supporting Data Tables/Figures below. Stormwater runoff in excess of existing conditions will be allowed to infiltrate on site. Design options that include roof drain catchments, permeable surfaces for roads and pedestrian paths, permeable drainage swales, and other alternatives to typical storm drain facilities will be applied (see Attachment 7). Mitigation and LID improvements are not expected to create any additional environmental impacts and are planned to be located in already disturbed areas as indicated more specifically above.

Project Characteristics and Design Features Description Pertinent to Resource Category

Comment noted, the previous responses provide general information on the proposed design of stormwater features, based upon the LID methodology, and also for the proposed debris basins. Additional information needed for analysis and final design of Project features, such as debris basins and channel stabilization measures would be collected and utilized during the design phase. Resources would include documents such as the *California Stormwater Best Management Practice Handbook: New Development and Redevelopment* developed by the California Stormwater Quality Association (CASQA). These resources would reflect industry accepted, proven BMPs for stormwater management. Additional information and examples from the California Stormwater BMP Handbook is provided in Attachment 7.

Impact Analysis Information

Potential impacts associated with the Project, relative to site drainage and runoff are expected to be mitigated by the proposed LID techniques that would include, but not be limited to, the following design elements (CASQA, 2003), and are highlighted in the responses above.

- Site design and landscape planning
- Roof runoff controls
- Alternative building materials
- Pervious paving
- Vegetated swales
- Vegetated buffer strips
- Bioretention

The existing stream that runs through the Project site will not be modified, except for the removal of existing culverts and bridge construction mentioned previously.

Supporting Data Tables/Figures

Site and watershed photos are presented in Attachment 3.

SCS CN were developed for the HEC-HMS model. The hydrologic soil group (A through D) was identified utilizing an online soils database and mapping system provided by the Natural Resource Conservation Service (NRCS) called Web Soil Survey 2.0. Attachment 8 includes a map of the Project watershed developed using Web Soil Survey 2.0 showing soil type and identifying the hydrologic soil groups appropriate for developing the SCS CN. The basis for SCS CN development is summarized in Table 1; SCS CN used in the HEC-HMS model are summarized in Table 2 by Subbasin.

TABLE 1
Basis for development of Subbasin Soil Conservation Service Curve Numbers
Paraiso Springs Resort – Response to Hydrology and Hydraulic Analysis and Erosion Control Measures Review Comments

Cover/Land Use [1]	Hydrologic Condition	Curve Numbers for Hydrologic Soil Group			
		A	B	C	D
Forestland – grass or orchards, evergreen or deciduous	Good	32	58	72	79
Residential – average lot size 1/3 acre (average 30% imperious, includes paved streets)	N/A	57	72	81	86

Notes:
1. Taken from Table 8.7.3 (Mays, 2001)

TABLE 2
HEC-HMS Subbasin Soil Conservation Service Curve Numbers
Paraiso Springs Resort – Response to Hydrology and Hydraulic Analysis and Erosion Control Measures Review Comments

Subbasin	Hydrologic Soil Group [1]	CN: Existing Conditions [2]	CN: Proposed Conditions [3]
V-1	B, C, and D	72	72
V-2			
Plane 1	A, C, and D	72	72
Plane 2	A, B, and D	72	81
N-1	C	72	72
N-2	C	72	72
N-3			
Plane 1	C and D	79	79
Plane 2	B and D	79	86
S-1	B	58	58
S-2	B	58	58
S-3	C	72	72
S-4			
Plane 1	B and C	72	72
Plane 2	B and D	72	81

Notes:

1. When more than one Hydrologic Soil Group was found to be present in a given Subbasin, soil group was determine
2. Assumes cover is Forestland for all Subbasins
3. Assumes cover changes from Forestland to Residential – average lot size 1/3 acre in Subbasins where development is proposed

Based on the current tentative map for the Project, approximately 24 acres of the proposed development could be impervious surfaces post construction if traditional design methods were utilized. However, the goal of the Project is to use LID to minimize the effect of the development to stormwater drainage patterns, to the extent feasible, with the ultimate goal of no net impact. Therefore, the percentage of impervious surface included in the model for post-Project conditions was assumed to be approximately 26 percent of the potential impervious surface area.

Table 3 presents the overall results for the Project watershed, volume and peak discharge, obtained from the HEC-HMS model for pre- and post-project conditions for 10-year and 100-year storm events.

TABLE 3
HEC-HMS Results, Pre- and Post-Project for 10-year and 100-year Storm Events
Paraiso Springs Resort – Response to Hydrology and Hydraulic Analysis and Erosion Control Measures Review Comments

Parameter	10-year Storm Event		100-year Storm Event	
	Pre-Project	Post Project	Pre-Project	Post Project
Volume (ac-ft)	117.5	124.0	261.1	269.6
Peak Discharge (cfs)	123.5	124.2	310.9	315.8

References

California Department of Water Resources (DWR). 1983. Paloma Gage Station, Station Number D20 6650 00.

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U.S. Geological Survey (USGS). 1994. *Nationwide Summary of U.S. Geological Survey Regional Regression Equations for Estimating Magnitude and Frequency of Floods for Ungaged Sites, 1993*. Water Resources Investigations Report 94-4002. Reston, Virginia.

MEMORANDUM **CH2MHILL**

Paraiso Springs Resort – Response to Hydrology and Hydraulic Analysis and Erosion Control Measures Review Comments

TO:	Thompson Holdings, LLC
COPIES:	David Von Rueden P.E./CH2M HILL
FROM:	Meabon Burns P.E./CH2M HILL Erika Powell P.E./CH2M HILL
DATE:	October 28, 2008
PROJECT NUMBER:	366335.03

REGISTERED PROFESSIONAL ENGINEER
MEABON KYLE BURNS
71053
EXP 6/30/2009
CIVIL
STATE OF CALIFORNIA

The purpose of this memorandum is to provide responses to review comments of Technical Memorandums titled Paraiso Springs Resort: Preliminary Hydrology and Hydraulic Analysis and Paraiso Springs Resort: Erosion Control Measures dated July 15, 2005. Comments were provided in a memorandum from Harvey Oslick, MS 1300 to Meryka Blumer, MS 1600 in a memorandum dated January 17, 2008. A copy of this memorandum is included in Attachment 1 for reference.

Additional Analysis to Support Response to Comments

An HEC-HMS (Hydrologic Engineering Center-Hydrologic Modeling System) model was developed to support the response to comments provided in this memorandum. HEC-HMS (version 3.1.0) is a hydrologic model developed and made available by the US Army Corps of Engineers. The key parameters of the analysis methodology are presented below:

- Unit system – U.S Customary
- Loss: – Soil Conservation Service (SCS) Curve Number (CN)
- Transform – Kinematic Wave
- Baseflow – None
- Routing: – Kinematic Wave
- Precipitation – SCS Storm
- Evapotranspiration – None
- Snowmelt – None

Analysis results are presented below within the response to comments.

Response to Comments

Information for Describing Existing Conditions/Setting

Hydrology and Hydraulic Analysis

1. Upon leaving the Project site, stormwater travels first through a natural ravine then through agricultural drainage ditches and culverts under road crossings (see

Attachment 2). The ditches are highly channelized, and are either located along natural drainage paths or adjacent to roads. The banks have been stabilized in some locations by the installation of sandbags. These drainage ditches are man-made, most likely by local property owners, and are characterized by steep, unvegetated side slopes. The level of maintenance for these incised channels is unknown. See Photos 1 through 3 in Attachment 3.

2. Maps showing the subbasin delineation for the Project watershed and Project site are included in Attachment 4. The Project watershed was delineated into nine subbasins. These subbasins were delineated because they either had distinct drainage characteristics or the flows collected at a location where specific project impacts could be identified, such as the potential for landslides or debris flows.

3. The HEC-HMS model facilitates a more rigorous and detailed analysis than the analysis that was conducted for the July 2005 Project evaluation and is appropriate for this application. The 10-year and 100-year event stormwater volumes for the entire watershed, using this more detailed hydrologic methodology as described above, were found to increase from 117.5 ac-ft (123.5 cfs) to 124.0 ac-ft (124.2 cfs) and 261.1 ac-ft (310.9 cfs) to 269.6 ac-ft (315.8 cfs) (see Supporting Data Tables/Figures), respectively. This increase in stormwater runoff of 6.5 ac-ft (0.7 cfs) for the 10-year storm and 8.5 ac-ft (4.9 cfs) for the 100-year storm translates to 5.5 percent and 3.3 percent, respectively, of the total runoff volume and 0.6 percent and 1.6 percent, respectively, of the peak discharge. This result is based on conservative assumptions regarding post-development conditions, such as new impervious area, overland flow roughness, and Soil Conservation Service Curve Numbers (SCS CN).

The approach to minimizing Project impacts due to stormwater runoff, as calculated above, will be to use low impact design (LID) methodologies. Specific LID techniques, often referred to as stormwater best management practices (BMPs), will be determined during the design process. For purposes of this preliminary analysis, the areas of "hardscape" shown on the Land Use Summary Table of the Project Tentative Map were evaluated for appropriate LID construction techniques. Project "hardscape" areas and related potential LID construction techniques are summarized below (CASQA, 2003). Actual BMPs and combinations of BMPs to be used will be evaluated during final design.

- Building footprints (7.22 acres)
  - Roof runoff controls
  - Site design and landscape planning
  - Alternative building materials

- Patios, Paths, and Driveways (5.99 acres)
  - Site design and landscape planning
  - Pervious paving
  - Vegetated swales
  - Vegetated buffer strip
  - Bioretention

- Parking and Roadways (9.98 acres)
  - Pervious paving
  - Vegetated swales
  - Vegetated buffer strip
  - Bioretention

As noted in the July 15, 2005 Memorandum, only 23.19 acres of the 1,160 acre Project watershed will be developed with "hardscape" features. Utilization of the LID techniques, as described above, is anticipated to limit the post-Project runoff from frequent storm events to virtually identical volumes as the pre-Project condition and to result in insignificant increases during the rare, infrequent events (i.e. 100 yr event).

It should also be noted that the Project site is underlain by predominantly sandy soils, as identified in the Project Geologic and Soil Engineering Report, prepared by Landset Engineers, Inc. and dated December 2004. This soil condition should be very compatible with the proposed LID construction techniques.

It should also be noted that the Project stormwater features will be designed to ensure that the pre-project 10-year event flow will continue to reach the drainage channel downstream of the Project site, post-development.

4. A U.S. Geological Survey (USGS) Water-Resources Investigations Report (USGS, 1994), which was used for the previous analysis and is included in Attachment 5, and private, unpublished information indicate that the mean annual precipitation (MAP) is approximately 11 inches east of the Project site. However, the elevation across the entire watershed ranges from 1,000 to approximately 2,400 feet. Therefore, it is likely that the MAP varies, potentially significantly, across the watershed with elevation. Figure 2.3 of the Monterey County Water Resources Agency's (MCWRA) *Water Resources Data Report: Water Year 1994-1995* presents MAP for Monterey County (MCWRA, 2007). The project site location was approximated on this map to find the MAP, see Attachment 6. MAP for the Project site was found to be 23 inches. This MAP was verified by data collected by a rain gage from 1950 to 1982. The Paloma Station is located approximately 9 miles southeast of the Project site (Longitude 121.500 W, Latitude 36.350 N) at an elevation 1,835 feet. The data collected at this station indicates that the MAP is 23.25 inches for the period of record (DWR, 1983).

   Based on available data, the MAP could range from 11 to 23 inches across the entire watershed. To be conservative, a MAP of 23 inches was used for the purpose of this analysis.

   The MAP, 23 inches, was used to calculate precipitation depth for the 10-year and 100-year storms for a duration of 24 hours. Precipitation depth was calculated using the Santa Clara County's Return Period-Duration-Specific (TDS) Regional Equation, which establishes a relationship between precipitation depth and MAP for various storm return periods. This equation was developed based on the three-day December 1995 rainfall event that is still considered to be the storm of record for Northern California. (Santa Clara County, 2007)

5. Detention ponds are not included in the Project, because the LID stormwater mitigation methodologies described above will be implemented. Debris basins, as recommended in

the previously referenced Geologic and Soil Engineering Report, would be implemented and located at the point of concentration for Subbasins N-1 (see Photo 4 in Attachment 3) and N-2, located in Indian Valley along the Northern edge of the Project site, and Subbasins S-1, S-2, and S-3, located along the southern edge of the Project site (see Attachment 4). These debris basins would intercept debris flows/slides from the identified Subbasins, above the developed areas of the Project. They will be located immediately adjacent to Project features and incorporated into the site grading footprint for the overall Project. The debris basins are expected to include a series of two-to-four small soil and rock checkdams, approximately three-feet tall, constructed at the low flow line of the natural drainage feature. Minimal excavation behind the checkdams is planned and no additional trees would be removed for construction. The debris basins would be constructed adjacent to Project roadways, parking lots or maintenance paths to facilitate inspection and maintenance.

Although Subbasin V-1 was identified as a potential site for debris flows, it is not anticipated that a debris basin will be needed at the point of concentration for this basin. The drainage channel was found to be well defined and relatively clear of debris at this location. Rocks that were present were in general no greater than approximately 24 inches in diameter. Debris flowing through the main drainage channel did cause flooding on site during a storm in 1995. However, this was due to the debris blocking flow through an existing culvert located upstream of the hot springs pools (see Photo 8 in Appendix 3). The culverts at this location and the culverts located just upstream of the property line (see Photo 1 in Appendix 3) will be removed as part of the Project to restore the drainage channel capacity. Bridges will be installed to allow vehicular and pedestrian access across the drainage channel. The bridges are expected to be single-span structures, with abutments on each bank of the stream. Stream banks would be reconstructed as part of the bridge construction and lined with rock riprap for scour protection immediately adjacent to the abutments. Small storm drain outfalls would be located within the bridge and rock riprap footprints.

Erosion Control Measures

Because the intention is to implement stormwater BMPs to ensure that post-development stormwater flows in excess of pre-development conditions for a 100-year storm event do not leave the Project site, aggradation of the channel downstream of the project site, is not expected. Based on field observations, most of the sediment that travels from the steeper areas of the watershed to the valley of the watershed during annual rainfall events, is naturally deposited on the flatter areas of the watershed (i.e., within the Project site). Sediment that currently feeds the channel downstream of the Paraiso Springs Resort Project site, during more frequent or annual rainfall events, is contributed by the adjacent floodplain below the Project site through sheetflow. Onsite debris basins will be designed to retain large-particle sediment and other debris, but not suspended sediment. Passage of suspended sediment will also be aided by the removal of existing culverts and the restoration of natural drainage channel conditions as part of the Project. Therefore, it is expected that nutrients that are necessary for the health of the channel, downstream of the project site, will continue to be replenished.

Any points where stormwater flows collect and it is necessary to discharge into the channel will be designed with appropriate and primarily natural erosion protection measures, such as rock slope protection and vegetation.

Regulatory Background Information

Comment noted regarding compliance with the requirements of Monterey County Ordinance Chapter 16.2 Erosion Control and Ordinance Chapter 19.10 Design and Standard Improvements, paragraph 19.10.050, Drainage. Analysis and design efforts for the Project will comply with County policies in place when construction documents are developed. Mitigation measures, such as permeable pavements and vegetated drainage swales, and stormwater collection systems will be designed to ensure that stormwater drainage volume and peak flows do not increase from existing conditions, as a result of the Project.

Comment noted regarding the anticipated new statewide NPDES Construction General Permit. Project construction documents will comply with the most current General Permit.

Analytical Methodology and Significance Thresholds

The proposed project will not alter the course of flow through the main drainage channel, will not significantly alter existing drainage patterns, and will not significantly increase the rate of runoff. Minimal impacts to peak flow discharge and flow volume will be mitigated onsite to ensure that no downstream impacts will result directly from the Project. Downstream capacity will not be exceeded due to the Project, flow in excess of current flows will be allowed to infiltrate on site.

Pre- and post-Project stormwater drainage volumes for 10-year and 100-year storm events are summarized under Supporting Data Tables/Figures below. Stormwater runoff in excess of existing conditions will be allowed to infiltrate on site. Design options that include roof drain catchments, permeable surfaces for roads and pedestrian paths, permeable drainage swales, and other alternatives to typical storm drain facilities will be applied (see Attachment 7). Mitigation and LID improvements are not expected to create any additional environmental impacts and are planned to be located in already disturbed areas as indicated more specifically above.

Project Characteristics and Design Features Description Pertinent to Resource Category

Comment noted, the previous responses provide general information on the proposed design of stormwater features, based upon the LID methodology, and also for the proposed debris basins. Additional information needed for analysis and final design of Project features, such as debris basins and channel stabilization measures would be collected and utilized during the design phase. Resources would include documents such as the *California Stormwater Best Management Practice Handbook: New Development and Redevelopment* developed by the California Stormwater Quality Association (CASQA). These resources would reflect industry accepted, proven BMPs for stormwater management. Additional information and examples from the California Stormwater BMP Handbook is provided in Attachment 7.

Impact Analysis Information

Potential impacts associated with the Project, relative to site drainage and runoff are expected to be mitigated by the proposed LID techniques that would include, but not be limited to, the following design elements (CASQA, 2003), and are highlighted in the responses above.

- Site design and landscape planning
- Roof runoff controls
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- Pervious paving
- Vegetated swales
- Vegetated buffer strips
- Bioretention

The existing stream that runs through the Project site will not be modified, except for the removal of existing culverts and bridge construction mentioned previously.

Supporting Data Tables/Figures

Site and watershed photos are presented in Attachment 3.

SCS CN were developed for the HEC-HMS model. The hydrologic soil group (A through D) was identified utilizing an online soils database and mapping system provided by the Natural Resource Conservation Service (NRCS) called Web Soil Survey 2.0. Attachment 8 includes a map of the Project watershed developed using Web Soil Survey 2.0 showing soil type and identifying the hydrologic soil groups appropriate for developing the SCS CN. The basis for SCS CN development is summarized in Table 1; SCS CN used in the HEC-HMS model are summarized in Table 2 by Subbasin.

TABLE 1
Basis for development of Subbasin Soil Conservation Service Curve Numbers
Paraiso Springs Resort – Response to Hydrology and Hydraulic Analysis and Erosion Control Measures Review Comments

Cover/Land Use [1]	Hydrologic Condition	Curve Numbers for Hydrologic Soil Group			
		A	B	C	D
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Residential – average lot size 1/3 acre (average 30% imperious, includes paved streets)	N/A	57	72	81	86

Notes:
1. Taken from Table 8.7.3 (Mays, 2001)

TABLE 2
HEC-HMS Subbasin Soil Conservation Service Curve Numbers
Paraiso Springs Resort – Response to Hydrology and Hydraulic Analysis and Erosion Control Measures Review Comments

Subbasin	Hydrologic Soil Group [1]	CN: Existing Conditions [2]	CN: Proposed Conditions [3]
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V-2			
Plane 1	A, C, and D	72	72
Plane 2	A, B, and D	72	81
N-1	C	72	72
N-2	C	72	72
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Plane 1	C and D	79	79
Plane 2	B and D	79	86
S-1	B	58	58
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Plane 1	B and C	72	72
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Notes:

1. When more than one Hydrologic Soil Group was found to be present in a given Subbasin, soil group was determine
2. Assumes cover is Forestland for all Subbasins
3. Assumes cover changes from Forestland to Residential – average lot size 1/3 acre in Subbasins where development is proposed

Based on the current tentative map for the Project, approximately 24 acres of the proposed development could be impervious surfaces post construction if traditional design methods were utilized. However, the goal of the Project is to use LID to minimize the effect of the development to stormwater drainage patterns, to the extent feasible, with the ultimate goal of no net impact. Therefore, the percentage of impervious surface included in the model for post-Project conditions was assumed to be approximately 26 percent of the potential impervious surface area.

Table 3 presents the overall results for the Project watershed, volume and peak discharge, obtained from the HEC-HMS model for pre- and post-project conditions for 10-year and 100-year storm events.

TABLE 3
HEC-HMS Results, Pre- and Post-Project for 10-year and 100-year Storm Events
Paraiso Springs Resort – Response to Hydrology and Hydraulic Analysis and Erosion Control Measures Review Comments

Parameter	10-year Storm Event		100-year Storm Event	
	Pre-Project	Post Project	Pre-Project	Post Project
Volume (ac-ft)	117.5	124.0	261.1	269.6
Peak Discharge (cfs)	123.5	124.2	310.9	315.8

References

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Natural Resource Conservation Service (NRCS). 20 June 2007. Web Soil Survey 2.0. <http://websoilsurvey.nrcs.usda.gov/app/>. 5 May 2008.

Santa Clara County. 14 August 2007. *Drainage Manual*.

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Attachment 1
Review of CH2M HILL Technical Memoranda (Hydrology and Hydraulic Analysis and Erosion Control Measures)

MEMORANDUM

To:	Meryka Blumer, MS 1600	JN 70-100140
From:	Harvey Oslick, MS 1300	
Cc:	Elizabeth Caraker, MS 1600	
Date:	January 17, 2008	
Subject:	Review of CH2MHill Technical Memoranda Paraiso Springs Resort: Preliminary Hydrology and Hydraulic Analysis and Paraiso Springs Resort: Erosion Control Measures (both dated July 15, 2005)	

The purpose of this memorandum is to provide a peer review of the technical memoranda, "Paraiso Springs Resort: Preliminary Hydrology and Hydraulic Analysis" and "Paraiso Springs Resort: Erosion Control Measures," both dated July 15, 2005. The focus of the review was to identify additional information necessary for RBF to complete a CEQA review of the project related to drainage and erosion impacts.

Information for Describing Existing Condition/Setting

The information in the technical memorandum on "Existing Hydrologic and Hydraulic Site Conditions" generally adequately describes existing conditions and setting of the site from a hydrologic standpoint, except as noted:

1. The Watershed Description, "travels northeasterly to the Arroyo Seco Valley floor, where flows are collected and enter the Arroyo Seco River," does not completely describe the condition of the receiving waterways that have the greatest potential to be impacted by the proposed development. The channels downstream from the project are not addressed on the Channel Characteristics section, either. Additional description of the receiving channels that cross the agricultural fields, Arroyo Seco Road and Los Coches Road should be provided.
2. The statement, "Topographic contour patterns show that there are four points within the basin that collect and transfer flows from the higher areas of the basin to the existing stream," is not supported by the exhibit. Additional clarification should be provided.
3. Calculations should be provided to support the statement, "Because this is such a(s) small percentage of the overall drainage basin at 2%, no significant increase on outflow from the basin is anticipated." The impacts of interceptor drainage ditches on hillside, increased impervious area, and channelization on both frequent storm flows should be addressed. Though the proposed project may not significantly alter the 100-year runoff from the site, increased discharges during frequent storm events may significantly impact flows and sediment transport to the agricultural fields downstream from the project. The impacts of the site on flows and sediment transport need to be addressed.
4. No reference is identified as the source of the listed average annual rainfall in the Project area of 11-inches. A value of 11-inches per year may be appropriate for the floor of the Salinas River Valley in the vicinity of the Project, however, it appears low for the

watershed tributary to the project with a centroid elevation of about 1800 feet. A suggested reference is the "Mean Annual Precipitation Map San Francisco & Monterey Bay Region, 1988" prepared by Santa Clara Valley Water District.

5. The report needs to include information regarding location and function of detention pond(s). It is suggested that the developer the Monterey County Water Resources Agency (MCWRA) to identify specific requirements. Typically, MCWRA has required that a subdivision include detention pond(s) with adequate volume to store the difference between the 100-year post-development runoff rate and the 10-year pre-development runoff rate, while limiting the discharge to the 10-year pre-development rate."

The information in the technical memorandum on "Preliminary Erosion Protection Measures" states that the site surface soils are erodible and that the hillside areas are susceptible to landslides and debris flow. The information in the technical memorandum on "Existing Hydrologic and Hydraulic Site Conditions" states that the bankfull capacity of the primary drainage channel far exceeds the 100-year storm flow. The documents should provide a detailed assessment of potential aggradation and degradation of the channels through and downstream from the project.

Regulatory Background Information

The project will need to satisfy the requirements of County Ordinance Chapter 16.12, Erosion Control. Preliminary erosion protection measures should be described in the context of meeting the specific provisions of this ordinance.

The project will need to satisfy the requirements of County Ordinance Chapter 19.10 Design and Standard Improvements, Paragraph 19.10.050, Drainage. Measures to mitigate for impacts to off site properties should be described in the context of meeting the specific provisions of this ordinance.

Note that it is anticipated that a new Statewide NPDES Construction General Permit (http://www.swrcb.ca.gov/stormwtr/constpermits.html) will be in place that will require additional measures to be addressed.

Analytical Methodology and Significance Thresholds

As required under CEQA, the project documents should present analysis related to the potential for the project to:

- Substantially alter the existing drainage pattern of the site or area, including through the alteration of the course of a stream or river, or substantially increase the rate or amount of surface runoff in a manner, which would result in flooding on- or off-site.
- Create or contribute runoff water, which would exceed the capacity of existing or planned stormwater drainage systems or provide substantial additional sources of polluted runoff.

Hydrology, hydraulic, and sediment transport analysis should be included to quantify the potential of the impacts of the project on drainage patterns, off-site flooding and water quality. The conclusions in the technical memorandum on "Existing Hydrologic and Hydraulic Site Conditions" state that debris basins upstream from the development should be implemented.

The impact that debris basins may have on the degradation of downstream channels as a result of a reduction in bed load should be addressed. Analysis should be provided to: identify locations of limited capacity downstream from the project; estimate the frequency that the capacity of the downstream system would be exceeded based on existing conditions; and estimate the impact the proposed project would have on this frequency.

Project Characteristics and Design Features Description Pertinent to Resource Category

The preliminary documents only discuss features in general terms. Additional information would be required to perform analysis on detention basins, permanent sediment traps, channel stabilization measures, and other design features that may be incorporated to mitigate for project impacts.

Impact Analysis Information

Potential impacts associated with the project as defined by the thresholds above should be clearly identified. Modifications to the project and or proposed mitigation measures should be included.

Supporting Data Tables/Figures

Data tables and figures should include soils maps and SCS curve numbers. Site and watershed photographs should be included to document existing conditions. Pre- and post-project times of concentration and flow rates for a wide range of storm events should be tabulated. Sediment and debris quantities should be addressed to identify preliminary debris basin sizes.

H:\PDATA\70100140\Admin\correspndnc\140HY-MEM01 Drainage Peer Review.doc

Attachment 2
Localized Stormwater Drainage Patterns

Aerial photo source: © Google, additional content added by CH2M HILL

Stormwater Drainage Route

Approximate Property Line

ATTACHMENT 2A
Stormwater Drainage Route Downstream of the Project Site
Localized Stormwater Drainage Patterns
Paraiso Springs Resort Subbasin Delineation

CH2MHILL

See Attachment 2A

Aerial photo source: © Google, additional content added by CH2M HILL

Stormwater Drainage Route

ATTACHMENT 2B
Stormwater Drainage Route Downstream of the Project Site
Localized Stormwater Drainage Patterns
Paraiso Springs Resort Subbasin Delineation

CH2MHILL

Attachment 3
Project Site Photos

Photo 1: Existing culverts on the Project Site above the eastern property line

Photo 2: Drainage channel passing through a vineyard downstream of the Project site

Photo 3: Roadside drainage ditch downstream of the Project site

Photo 4: Approximate point of concentration for Subbasin N-1

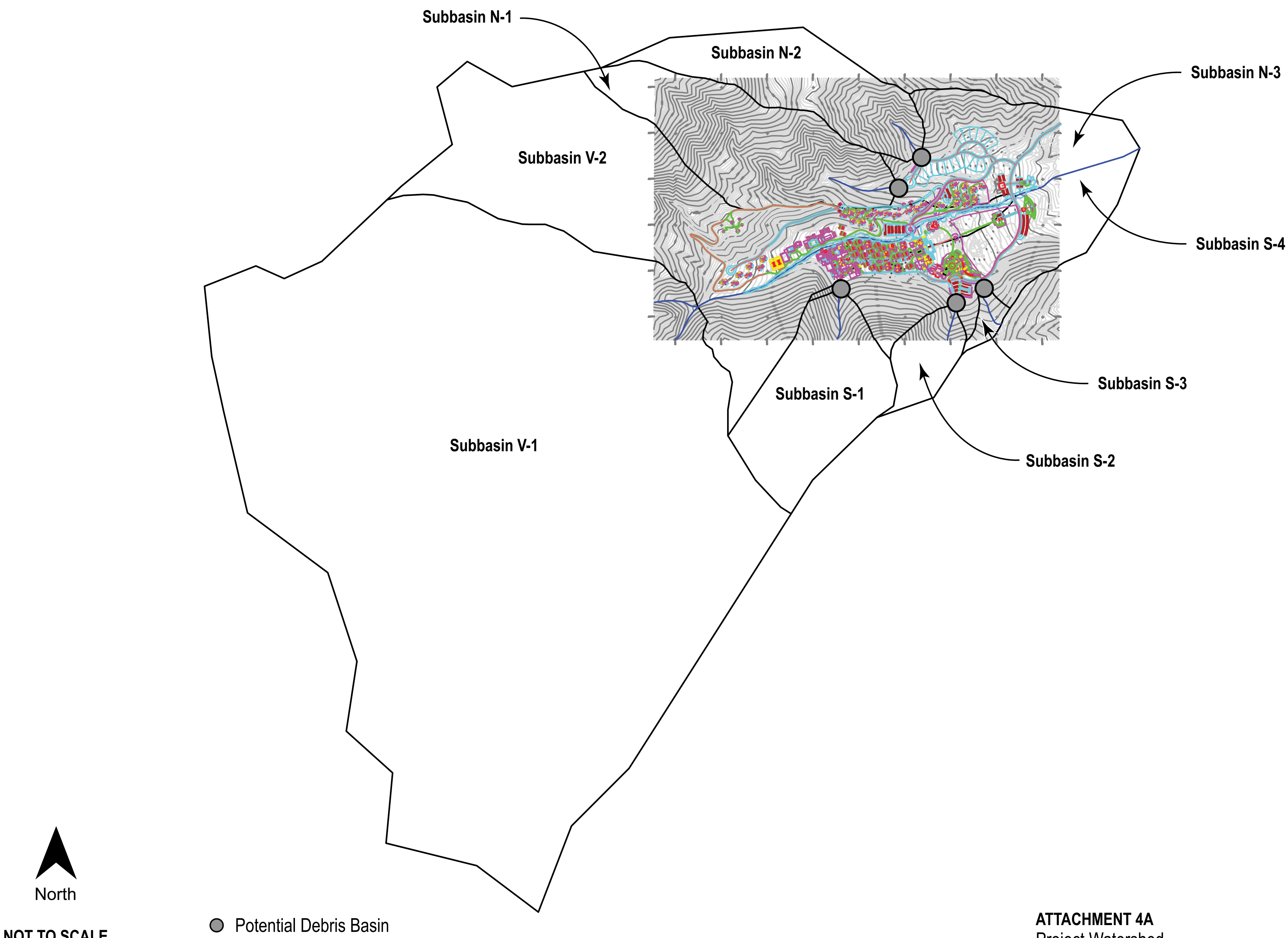
Photo 5: Approximate point of concentration for Subbasin V-1

Photo 6: Main drainage channel looking upstream, downstream of Photo 5 and downstream of Photo 7

Photo 7: Main drainage channel looking upstream, just upstream from Photo 8

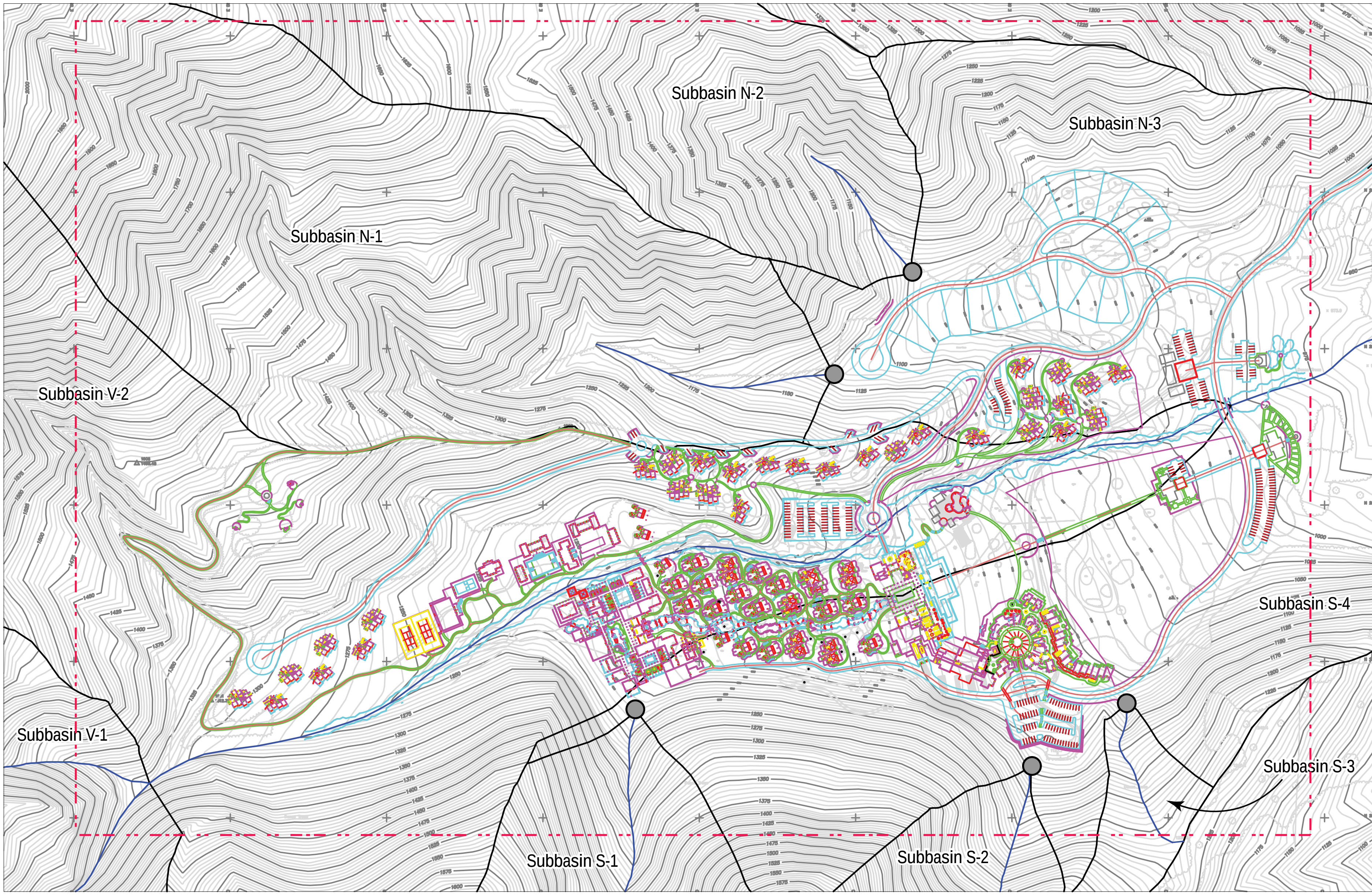
Photo 8: Main drainage channel looking downstream; culverts located upstream of the existing hot springs pools

Attachment 4
Subbasin Delineation



ATTACHMENT 4A
Project Watershed
Paraiso Springs Resort Subbasin Delineation

CH2MHILL



North
NOT TO SCALE

Potential Debris Basin

Approximate Property Line

ATTACHMENT 4B
Project Site
Paraiso Springs Resort Subbasin Delineation

CH2MHILL

Attachment 5
U.S. Geological Survey Water Resources Investigations Report 94-4002 Mean Annual Precipitation Analysis

Nationwide Summary of U.S. Geological Survey Regional Regression Equations for Estimating Magnitude and Frequency of Floods for Ungaged Sites, 1993

Compiled By M.E. Jennings, W.O. Thomas, Jr., and H.C. Riggs

U.S. GEOLOGICAL SURVEY
Water-Resources Investigations Report 94-4002

Prepared in cooperation with the
FEDERAL HIGHWAY ADMINISTRATION
and the
FEDERAL EMERGENCY MANAGEMENT AGENCY

Reston, Virginia
1994

U.S. DEPARTMENT OF THE INTERIOR

BRUCE BABBITT, Secretary

U.S. GEOLOGICAL SURVEY

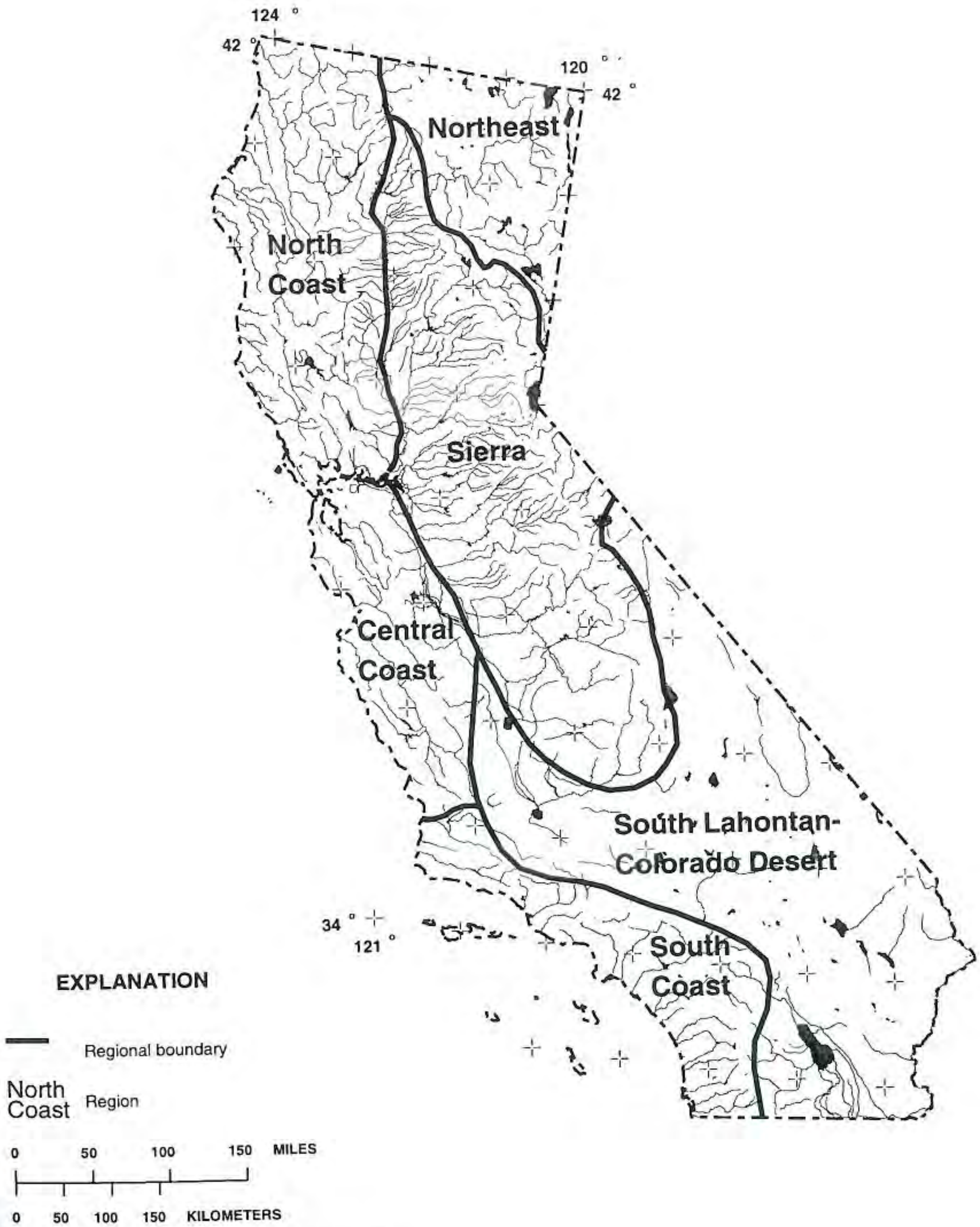
Gordon P. Eaton, Director

For additional information write to:

District Chief
U.S. Geological Survey
8011 Cameron Rd.
Austin, TX 78754-3898

Copies of this report can be purchased from:

U.S. Geological Survey
Earth Science Information Center
Open-File Reports Section
Box 25286, Mail Stop 517
Denver Federal Center
Denver, CO 80225-0046



Digital base from U.S. Geological Survey 1:2,000,000 , 1970
Albers equal-area projection based on standard parallels 29.5 and 45.5 degrees

Figure 1. Flood-frequency region map for California.

South Lahontan-Colorado Desert Region

$$Q2 = 7.3A^{0.30}$$
$$Q5 = 53A^{0.44}$$
$$Q10 = 150A^{0.53}$$
$$Q25 = 410A^{0.63}$$
$$Q50 = 700A^{0.68}$$
$$Q100 = 1{,}080A^{0.71}$$

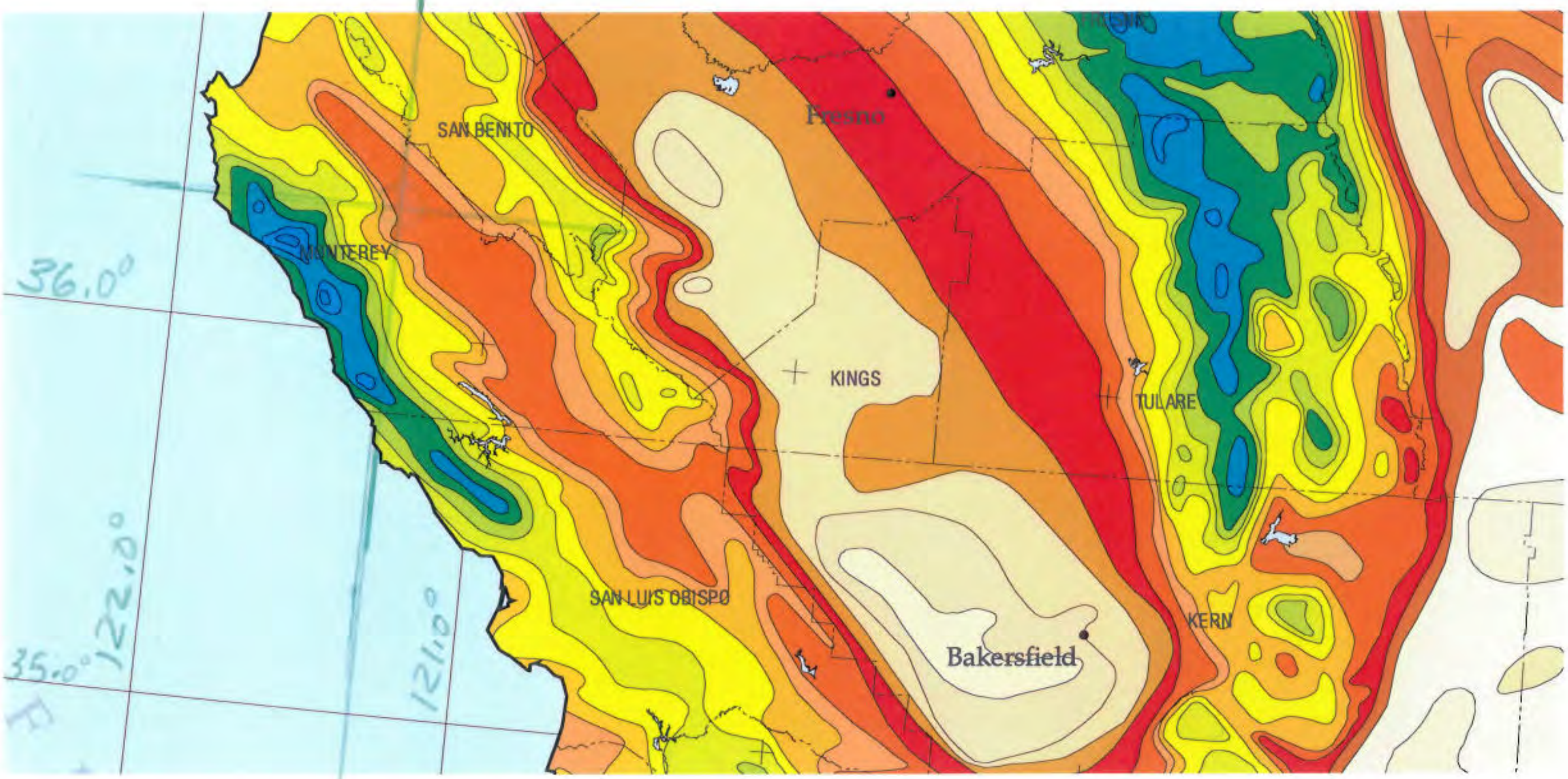
In the North Coast region, use a minimum value of 1.0 for the altitude index (H). Equations are defined only for basins of 25 mi^2 or less in the Northeast and South Lahontan-Colorado Desert regions.

Reference

Waananen, A.O. and Crippen, J.R., 1977, Magnitude and frequency of floods in California: U.S. Geological Survey Water-Resources Investigations Report 77-21, 96 p.

Additional Reference

Rantz, S.E., 1969, Mean annual precipitation in the California region: U.S. Geological Survey Open-File Map (Reprinted 1972, 1975).



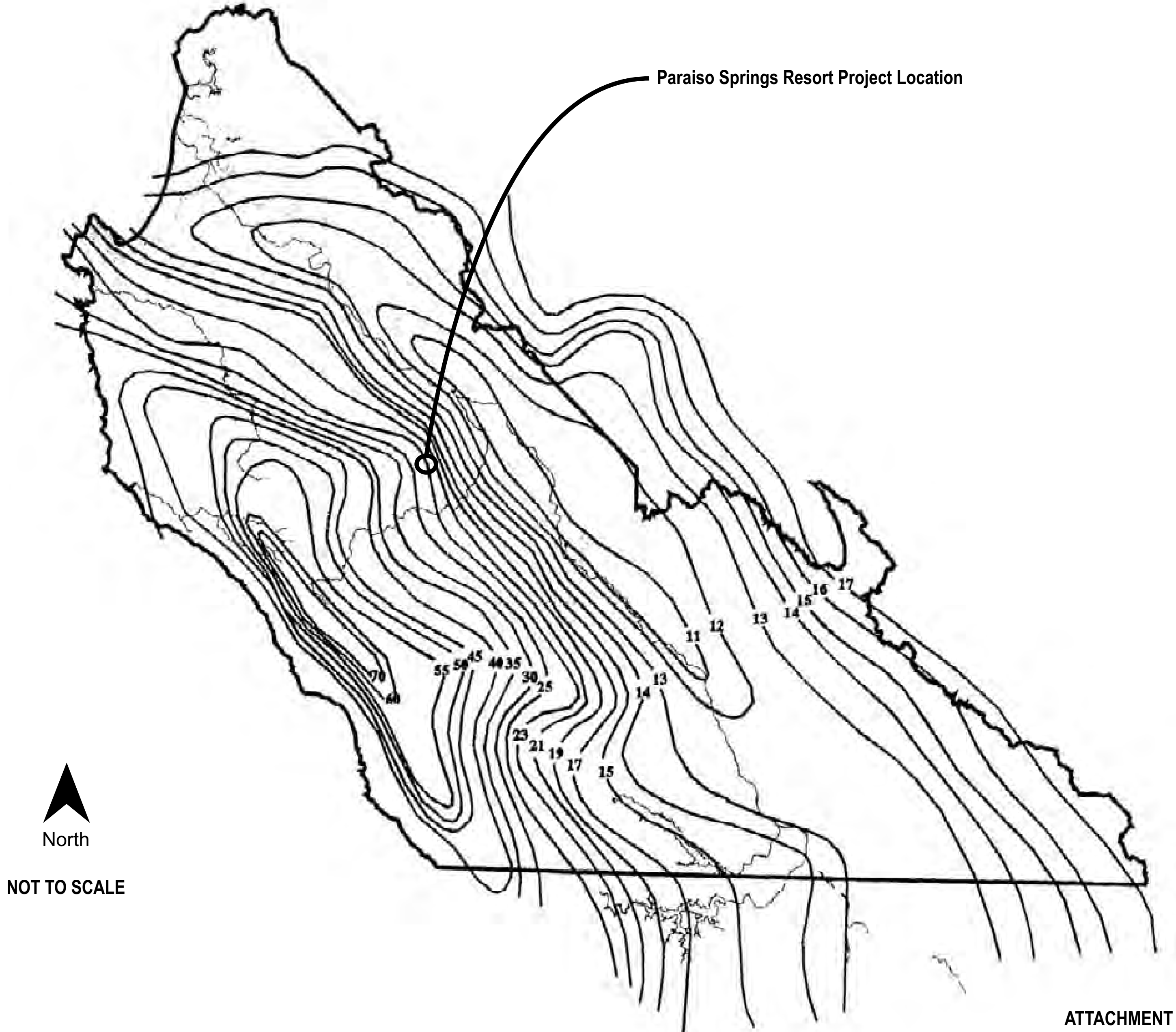
SAN BENITO
Fresno
MONTEREY
36.0°
KINGS
TULARE
122.0°
SAN LUIS OBISPO
121.0°
KERN
35.0°
Bakersfield

PRECIPITATION

2.5	7.5	15.0	30.0	65.0
3.5	8.0	17.0	32.5	75.0
4.5	9.0	17.5	35.0	85.0
5.0	11.0	18.0	37.5	95.0
5.5	12.5	22.5	40.0	105.0
6.5	13.0	25.0	45.0	115.0
7.0	14.0	27.5	55.0	125.0

Annual average precipitation polygons represent areas described by isohyetal lines of precipitation, measured in inches, averaged over the period 1900-1960.

Attachment 6
Monterey County Mean Annual Precipitation Map Used for HEC-HMS Analysis



ATTACHMENT 5
Monterey County Mean Annual Precipitation
Paraiso Springs Resort Subbasin Delineation

SOURCE: MCWRA Annual Report 1995 Water Year (MCWRA, 1997)

CH2MHILL

Attachment 7
Potential Mitigation Measures

Design Objectives

- ☑ Maximize Infiltration
- ☑ Provide Retention
- ☑ Slow Runoff
- ☑ Minimize Impervious Land Coverage
- Prohibit Dumping of Improper Materials
- Contain Pollutants
- Collect and Convey

Description

Each project site possesses unique topographic, hydrologic, and vegetative features, some of which are more suitable for development than others. Integrating and incorporating appropriate landscape planning methodologies into the project design is the most effective action that can be done to minimize surface and groundwater contamination from stormwater.

Approach

Landscape planning should couple consideration of land suitability for urban uses with consideration of community goals and projected growth. Project plan designs should conserve natural areas to the extent possible, maximize natural water storage and infiltration opportunities, and protect slopes and channels.

Suitable Applications

Appropriate applications include residential, commercial and industrial areas planned for development or redevelopment.

Design Considerations

Design requirements for site design and landscapes planning should conform to applicable standards and specifications of agencies with jurisdiction and be consistent with applicable General Plan and Local Area Plan policies.

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Designing New Installations

Begin the development of a plan for the landscape unit with attention to the following general principles:

- Formulate the plan on the basis of clearly articulated community goals. Carefully identify conflicts and choices between retaining and protecting desired resources and community growth.
- Map and assess land suitability for urban uses. Include the following landscape features in the assessment: wooded land, open unwooded land, steep slopes, erosion-prone soils, foundation suitability, soil suitability for waste disposal, aquifers, aquifer recharge areas, wetlands, floodplains, surface waters, agricultural lands, and various categories of urban land use. When appropriate, the assessment can highlight outstanding local or regional resources that the community determines should be protected (e.g., a scenic area, recreational area, threatened species habitat, farmland, fish run). Mapping and assessment should recognize not only these resources but also additional areas needed for their sustenance.

Project plan designs should conserve natural areas to the extent possible, maximize natural water storage and infiltration opportunities, and protect slopes and channels.

Conserve Natural Areas during Landscape Planning

If applicable, the following items are required and must be implemented in the site layout during the subdivision design and approval process, consistent with applicable General Plan and Local Area Plan policies:

- Cluster development on least-sensitive portions of a site while leaving the remaining land in a natural undisturbed condition.
- Limit clearing and grading of native vegetation at a site to the minimum amount needed to build lots, allow access, and provide fire protection.
- Maximize trees and other vegetation at each site by planting additional vegetation, clustering tree areas, and promoting the use of native and/or drought tolerant plants.
- Promote natural vegetation by using parking lot islands and other landscaped areas.
- Preserve riparian areas and wetlands.

Maximize Natural Water Storage and Infiltration Opportunities Within the Landscape Unit

- Promote the conservation of forest cover. Building on land that is already deforested affects basin hydrology to a lesser extent than converting forested land. Loss of forest cover reduces interception storage, detention in the organic forest floor layer, and water losses by evapotranspiration, resulting in large peak runoff increases and either their negative effects or the expense of countering them with structural solutions.
- Maintain natural storage reservoirs and drainage corridors, including depressions, areas of permeable soils, swales, and intermittent streams. Develop and implement policies and

regulations to discourage the clearing, filling, and channelization of these features. Utilize them in drainage networks in preference to pipes, culverts, and engineered ditches.

- Evaluating infiltration opportunities by referring to the stormwater management manual for the jurisdiction and pay particular attention to the selection criteria for avoiding groundwater contamination, poor soils, and hydrogeological conditions that cause these facilities to fail. If necessary, locate developments with large amounts of impervious surfaces or a potential to produce relatively contaminated runoff away from groundwater recharge areas.

Protection of Slopes and Channels during Landscape Design

- Convey runoff safely from the tops of slopes.
- Avoid disturbing steep or unstable slopes.
- Avoid disturbing natural channels.
- Stabilize disturbed slopes as quickly as possible.
- Vegetate slopes with native or drought tolerant vegetation.
- Control and treat flows in landscaping and/or other controls prior to reaching existing natural drainage systems.
- Stabilize temporary and permanent channel crossings as quickly as possible, and ensure that increases in run-off velocity and frequency caused by the project do not erode the channel.
- Install energy dissipaters, such as riprap, at the outlets of new storm drains, culverts, conduits, or channels that enter unlined channels in accordance with applicable specifications to minimize erosion. Energy dissipaters shall be installed in such a way as to minimize impacts to receiving waters.
- Line on-site conveyance channels where appropriate, to reduce erosion caused by increased flow velocity due to increases in tributary impervious area. The first choice for linings should be grass or some other vegetative surface, since these materials not only reduce runoff velocities, but also provide water quality benefits from filtration and infiltration. If velocities in the channel are high enough to erode grass or other vegetative linings, riprap, concrete, soil cement, or geo-grid stabilization are other alternatives.
- Consider other design principles that are comparable and equally effective.

Redeveloping Existing Installations

Various jurisdictional stormwater management and mitigation plans (SUSMP, WQMP, etc.) define "redevelopment" in terms of amounts of additional impervious area, increases in gross floor area and/or exterior construction, and land disturbing activities with structural or impervious surfaces. The definition of " redevelopment" must be consulted to determine whether or not the requirements for new development apply to areas intended for redevelopment. If the definition applies, the steps outlined under "designing new installations" above should be followed.

Redevelopment may present significant opportunity to add features which had not previously been implemented. Examples include incorporation of depressions, areas of permeable soils, and swales in newly redeveloped areas. While some site constraints may exist due to the status of already existing infrastructure, opportunities should not be missed to maximize infiltration, slow runoff, reduce impervious areas, disconnect directly connected impervious areas.

Other Resources

A Manual for the Standard Urban Stormwater Mitigation Plan (SUSMP), Los Angeles County Department of Public Works, May 2002.

Stormwater Management Manual for Western Washington, Washington State Department of Ecology, August 2001.

Model Standard Urban Storm Water Mitigation Plan (SUSMP) for San Diego County, Port of San Diego, and Cities in San Diego County, February 14, 2002.

Model Water Quality Management Plan (WQMP) for County of Orange, Orange County Flood Control District, and the Incorporated Cities of Orange County, Draft February 2003.

Ventura Countywide Technical Guidance Manual for Stormwater Quality Control Measures, July 2002.

Rain Garden

Design Objectives

- ☑ Maximize Infiltration
- ☑ Provide Retention
- ☑ Slow Runoff
- Minimize Impervious Land Coverage
- Prohibit Dumping of Improper Materials
- ☑ Contain Pollutants
- Collect and Convey

Description

Various roof runoff controls are available to address stormwater that drains off rooftops. The objective is to reduce the total volume and rate of runoff from individual lots, and retain the pollutants on site that may be picked up from roofing materials and atmospheric deposition. Roof runoff controls consist of directing the roof runoff away from paved areas and mitigating flow to the storm drain system through one of several general approaches: cisterns or rain barrels; dry wells or infiltration trenches; pop-up emitters, and foundation planting. The first three approaches require the roof runoff to be contained in a gutter and downspout system. Foundation planting provides a vegetated strip under the drip line of the roof.

Approach

Design of individual lots for single-family homes as well as lots for higher density residential and commercial structures should consider site design provisions for containing and infiltrating roof runoff or directing roof runoff to vegetative swales or buffer areas. Retained water can be reused for watering gardens, lawns, and trees. Benefits to the environment include reduced demand for potable water used for irrigation, improved stormwater quality, increased groundwater recharge, decreased runoff volume and peak flows, and decreased flooding potential.

Suitable Applications

Appropriate applications include residential, commercial and industrial areas planned for development or redevelopment.

Design Considerations

Designing New Installations

Cisterns or Rain Barrels

One method of addressing roof runoff is to direct roof downspouts to cisterns or rain barrels. A cistern is an above ground storage vessel with either a manually operated valve or a permanently open outlet. Roof runoff is temporarily stored and then released for irrigation or infiltration between storms. The number of rain

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barrels needed is a function of the rooftop area. Some low impact developers recommend that every house have at least 2 rain barrels, with a minimum storage capacity of 1000 liters. Roof barrels serve several purposes including mitigating the first flush from the roof which has a high volume, amount of contaminants, and thermal load. Several types of rain barrels are commercially available. Consideration must be given to selecting rain barrels that are vector proof and childproof. In addition, some barrels are designed with a bypass valve that filters out grit and other contaminants and routes overflow to a soak-away pit or rain garden.

If the cistern has an operable valve, the valve can be closed to store stormwater for irrigation or infiltration between storms. This system requires continual monitoring by the resident or grounds crews, but provides greater flexibility in water storage and metering. If a cistern is provided with an operable valve and water is stored inside for long periods, the cistern must be covered to prevent mosquitoes from breeding.

A cistern system with a permanently open outlet can also provide for metering stormwater runoff. If the cistern outlet is significantly smaller than the size of the downspout inlet (say ¼ to ½ inch diameter), runoff will build up inside the cistern during storms, and will empty out slowly after peak intensities subside. This is a feasible way to mitigate the peak flow increases caused by rooftop impervious land coverage, especially for the frequent, small storms.

Dry wells and Infiltration Trenches

Roof downspouts can be directed to dry wells or infiltration trenches. A dry well is constructed by excavating a hole in the ground and filling it with an open graded aggregate, and allowing the water to fill the dry well and infiltrate after the storm event. An underground connection from the downspout conveys water into the dry well, allowing it to be stored in the voids. To minimize sedimentation from lateral soil movement, the sides and top of the stone storage matrix can be wrapped in a permeable filter fabric, though the bottom may remain open. A perforated observation pipe can be inserted vertically into the dry well to allow for inspection and maintenance.

In practice, dry wells receiving runoff from single roof downspouts have been successful over long periods because they contain very little sediment. They must be sized according to the amount of rooftop runoff received, but are typically 4 to 5 feet square, and 2 to 3 feet deep, with a minimum of 1-foot soil cover over the top (maximum depth of 10 feet).

To protect the foundation, dry wells must be set away from the building at least 10 feet. They must be installed in solids that accommodate infiltration. In poorly drained soils, dry wells have very limited feasibility.

Infiltration trenches function in a similar manner and would be particularly effective for larger roof areas. An infiltration trench is a long, narrow, rock-filled trench with no outlet that receives stormwater runoff. These are described under Treatment Controls.

Pop-up Drainage Emitter

Roof downspouts can be directed to an underground pipe that daylights some distance from the building foundation, releasing the roof runoff through a pop-up emitter. Similar to a pop-up irrigation head, the emitter only opens when there is flow from the roof. The emitter remains flush to the ground during dry periods, for ease of lawn or landscape maintenance.

Foundation Planting

Landscape planting can be provided around the base to allow increased opportunities for stormwater infiltration and protect the soil from erosion caused by concentrated sheet flow coming off the roof. Foundation plantings can reduce the physical impact of water on the soil and provide a subsurface matrix of roots that encourage infiltration. These plantings must be sturdy enough to tolerate the heavy runoff sheet flows, and periodic soil saturation.

Redeveloping Existing Installations

Various jurisdictional stormwater management and mitigation plans (SUSMP, WQMP, etc.) define "redevelopment" in terms of amounts of additional impervious area, increases in gross floor area and/or exterior construction, and land disturbing activities with structural or impervious surfaces. The definition of " redevelopment" must be consulted to determine whether or not the requirements for new development apply to areas intended for redevelopment. If the definition applies, the steps outlined under "designing new installations" above should be followed.

Supplemental Information

Examples

- City of Ottawa's Water Links Surface –Water Quality Protection Program
- City of Toronto Downspout Disconnection Program
- City of Boston, MA, Rain Barrel Demonstration Program

Other Resources

Hager, Marty Catherine, Stormwater, "Low-Impact Development", January/February 2003. www.stormh2o.com

Low Impact Urban Design Tools, Low Impact Development Design Center, Beltsville, MD. www.lid-stormwater.net

Start at the Source, Bay Area Stormwater Management Agencies Association, 1999 Edition

Design Objectives

- ☑ Maximize Infiltration
- ☑ Provide Retention
- ☑ Slow Runoff
- ☑ Minimize Impervious Land Coverage
- Prohibit Dumping of Improper Materials
- Contain Pollutants
- Collect and Convey

Description

Pervious paving is used for light vehicle loading in parking areas. The term describes a system comprising a load-bearing, durable surface together with an underlying layered structure that temporarily stores water prior to infiltration or drainage to a controlled outlet. The surface can itself be porous such that water infiltrates across the entire surface of the material (e.g., grass and gravel surfaces, porous concrete and porous asphalt), or can be built up of impermeable blocks separated by spaces and joints, through which the water can drain. This latter system is termed 'permeable' paving. Advantages of pervious pavements is that they reduce runoff volume while providing treatment, and are unobtrusive resulting in a high level of acceptability.

Approach

Attenuation of flow is provided by the storage within the underlying structure or sub base, together with appropriate flow controls. An underlying geotextile may permit groundwater recharge, thus contributing to the restoration of the natural water cycle. Alternatively, where infiltration is inappropriate (e.g., if the groundwater vulnerability is high, or the soil type is unsuitable), the surface can be constructed above an impermeable membrane. The system offers a valuable solution for drainage of spatially constrained urban areas.

Significant attenuation and improvement in water quality can be achieved by permeable pavements, whichever method is used. The surface and subsurface infrastructure can remove both the soluble and fine particulate pollutants that occur within urban runoff. Roof water can be piped into the storage area directly, adding areas from which the flow can be attenuated. Also, within lined systems, there is the opportunity for stored runoff to be piped out for reuse.

Suitable Applications

Residential, commercial and industrial applications are possible. The use of permeable pavement may be restricted in cold regions, arid regions or regions with high wind erosion. There are some specific disadvantages associated with permeable pavement, which are as follows:

- Permeable pavement can become clogged if improperly installed or maintained. However, this is countered by the ease with which small areas of paving can be cleaned or replaced when blocked or damaged.

- Their application should be limited to highways with low traffic volumes, axle loads and speeds (less than 30 mph limit), car parking areas and other lightly trafficked or non-trafficked areas. Permeable surfaces are currently not considered suitable for adoptable roads due to the risks associated with failure on high speed roads, the safety implications of ponding, and disruption arising from reconstruction.

- When using un-lined, infiltration systems, there is some risk of contaminating groundwater, depending on soil conditions and aquifer susceptibility. However, this risk is likely to be small because the areas drained tend to have inherently low pollutant loadings.

- The use of permeable pavement is restricted to gentle slopes.

- Porous block paving has a higher risk of abrasion and damage than solid blocks.

Design Considerations

Designing New Installations

If the grades, subsoils, drainage characteristics, and groundwater conditions are suitable, permeable paving may be substituted for conventional pavement on parking areas, cul de sacs and other areas with light traffic. Slopes should be flat or very gentle. Scottish experience has shown that permeable paving systems can be installed in a wide range of ground conditions, and the flow attenuation performance is excellent even when the systems are lined.

The suitability of a pervious system at a particular pavement site will, however, depend on the loading criteria required of the pavement.

Where the system is to be used for infiltrating drainage waters into the ground, the vulnerability of local groundwater sources to pollution from the site should be low, and the seasonal high water table should be at least 4 feet below the surface.

Ideally, the pervious surface should be horizontal in order to intercept local rainfall at source. On sloping sites, pervious surfaces may be terraced to accommodate differences in levels.

Design Guidelines

The design of each layer of the pavement must be determined by the likely traffic loadings and their required operational life. To provide satisfactory performance, the following criteria should be considered:

- The subgrade should be able to sustain traffic loading without excessive deformation.

- The granular capping and sub-base layers should give sufficient load-bearing to provide an adequate construction platform and base for the overlying pavement layers.

- The pavement materials should not crack of suffer excessive rutting under the influence of traffic. This is controlled by the horizontal tensile stress at the base of these layers.

There is no current structural design method specifically for pervious pavements. Allowances should be considered the following factors in the design and specification of materials:

- Pervious pavements use materials with high permeability and void space. All the current UK pavement design methods are based on the use of conventional materials that are dense and relatively impermeable. The stiffness of the materials must therefore be assessed.

- Water is present within the construction and can soften and weaken materials, and this must be allowed for.

- Existing design methods assume full friction between layers. Any geotextiles or geomembranes must be carefully specified to minimize loss of friction between layers.

- Porous asphalt loses adhesion and becomes brittle as air passes through the voids. Its durability is therefore lower than conventional materials.

The single sized grading of materials used means that care should be taken to ensure that loss of finer particles between unbound layers does not occur.

Positioning a geotextile near the surface of the pervious construction should enable pollutants to be trapped and retained close to the surface of the construction. This has both advantages and disadvantages. The main disadvantage is that the filtering of sediments and their associated pollutants at this level may hamper percolation of waters and can eventually lead to surface ponding. One advantage is that even if eventual maintenance is required to reinstate infiltration, only a limited amount of the construction needs to be disturbed, since the sub-base below the geotextile is protected. In addition, the pollutant concentration at a high level in the structure allows for its release over time. It is slowly transported in the stormwater to lower levels where chemical and biological processes may be operating to retain or degrade pollutants.

The design should ensure that sufficient void space exists for the storage of sediments to limit the period between remedial works.

- Pervious pavements require a single size grading to give open voids. The choice of materials is therefore a compromise between stiffness, permeability and storage capacity.

- Because the sub-base and capping will be in contact with water for a large part of the time, the strength and durability of the aggregate particles when saturated and subjected to wetting and drying should be assessed.

- A uniformly graded single size material cannot be compacted and is liable to move when construction traffic passes over it. This effect can be reduced by the use of angular crushed rock material with a high surface friction.

In pollution control terms, these layers represent the site of long term chemical and biological pollutant retention and degradation processes. The construction materials should be selected, in addition to their structural strength properties, for their ability to sustain such processes. In general, this means that materials should create neutral or slightly alkaline conditions and they should provide favorable sites for colonization by microbial populations.

Construction/Inspection Considerations

- Permeable surfaces can be laid without cross-falls or longitudinal gradients.
- The blocks should be lain level
- They should not be used for storage of site materials, unless the surface is well protected from deposition of silt and other spillages.
- The pavement should be constructed in a single operation, as one of the last items to be built, on a development site. Landscape development should be completed before pavement construction to avoid contamination by silt or soil from this source.
- Surfaces draining to the pavement should be stabilized before construction of the pavement.
- Inappropriate construction equipment should be kept away from the pavement to prevent damage to the surface, sub-base or sub-grade.

Maintenance Requirements

The maintenance requirements of a pervious surface should be reviewed at the time of design and should be clearly specified. Maintenance is required to prevent clogging of the pervious surface. The factors to be considered when defining maintenance requirements must include:

- Type of use
- Ownership
- Level of trafficking
- The local environment and any contributing catchments

Studies in the UK have shown satisfactory operation of porous pavement systems without maintenance for over 10 years and recent work by Imbe et al. at 9th ICUD, Portland, 2002 describes systems operating for over 20 years without maintenance. However, performance under such regimes could not be guaranteed, Table 1 shows typical recommended maintenance regimes:

Table 1 Typical Recommended Maintenance Regimes

Activity	Schedule
■ Minimize use of salt or grit for de-icing ■ Keep landscaped areas well maintained ■ Prevent soil being washed onto pavement	Ongoing
■ Vacuum clean surface using commercially available sweeping machines at the following times: - End of winter (April) - Mid-summer (July / August) - After Autumn leaf-fall (November)	2/3 x per year
■ Inspect outlets	Annual
■ If routine cleaning does not restore infiltration rates, then reconstruction of part of the whole of a pervious surface may be required. ■ The surface area affected by hydraulic failure should be lifted for inspection of the internal materials to identify the location and extent of the blockage. ■ Surface materials should be lifted and replaced after brush cleaning. Geotextiles may need complete replacement. ■ Sub-surface layers may need cleaning and replacing. ■ Removed silts may need to be disposed of as controlled waste.	As needed (infrequent) Maximum 15-20 years

Permeable pavements are up to 25 % cheaper (or at least no more expensive than the traditional forms of pavement construction), when all construction and drainage costs are taken into account. (Accepting that the porous asphalt itself is a more expensive surfacing, the extra cost of which is offset by the savings in underground pipework etc.) (Niemczynowicz, et al., 1987)

Table 1 gives US cost estimates for capital and maintenance costs of porous pavements (Landphair et al., 2000)

Redeveloping Existing Installations

Various jurisdictional stormwater management and mitigation plans (SUSMP, WQMP, etc.) define "redevelopment" in terms of amounts of additional impervious area, increases in gross floor area and/or exterior construction, and land disturbing activities with structural or impervious surfaces. The definition of " redevelopment" must be consulted to determine whether or not the requirements for new development apply to areas intended for redevelopment. If the definition applies, the steps outlined under "designing new installations" above should be followed.

Additional Information

Cost Considerations

Permeable pavements are up to 25 % cheaper (or at least no more expensive than the traditional forms of pavement construction), when all construction and drainage costs are taken into account. (Accepting that the porous asphalt itself is a more expensive surfacing, the extra cost of which is offset by the savings in underground pipework etc.) (Niemczynowicz, et al., 1987)

Table 2 gives US cost estimates for capital and maintenance costs of porous pavements (Landphair et al., 2000)

Table 2 Engineer's Estimate for Porous Pavement

Porous Pavement													
Item	Units	Price	Cycles/ Year	Quant. 1 Acre WS	Total	Quant. 2 Acre WS	Total	Quant. 3 Acre WS	Total	Quant. 4 Acre WS	Total	Quant. 5 Acre WS	Total
Grading	SY	$2.00		604	$1,208	1209	$2,418	1812	$3,624	2419	$4,838	3020	$6,040
Paving	SY	$19.00		212	$4,028	424	$8,056	636	$12,084	848	$16,112	1060	$20,140
Excavation	CY	$3.60		201	$724	403	$1,451	604	$2,174	806	$2,902	1008	$3,629
Filter Fabric	SY	$1.15		700	$805	1400	$1,610	2000	$2,300	2800	$3,220	3600	$4,140
Stone Fill	CY	$16.00		201	$3,216	403	$6,448	604	$9,664	806	$12,896	1008	$16,128
Sand	CY	$7.00		100	$700	200	$1,400	300	$2,100	400	$2,800	500	$3,500
Sight Well	EA	$300.00		2	$600	3	$900	4	$1,200	7	$2,100	7	$2,100
Seeding	LF	$0.05		644	$32	1288	$64	1932	$97	2576	$129	3220	$161
Check Dam	CY	$35.00		0	$0	0	$0	0	$0	0	$0	0	$0
Total Construction Costs					**$10,105**		**$19,929**		**$29,619**		**$40,158**		**$49,798**
Construction Costs Amortized for 20 Years					**$505**		**$996**		**$1,481**		**$2,008**		**$2,490**
Annual Maintenance Expense													
Item	Units	Price	Cycles/ Year	Quant. 1 Acre WS	Total	Quant. 2 Acre WS	Total	Quant. 3 Acre WS	Total	Quant. 4 Acre WS	Total	Quant. 5 Acre WS	Total
Sweeping	AC	$250.00	6	1	$1,500	2	$3,000	3	$4,500	4	$6,000	5	$7,500
Washing	AC	$250.00	6	1	$1,500	2	$3,000	3	$4,500	4	$6,000	5	$7,500
Inspection	MH	$20.00	5	5	$100	5	$100	5	$100	5	$100	5	$100
Deep Clean	AC	$450.00	0.5	1	$225	2	$450	3	$675	3.9	$878	5	$1,125
Total Annual Maintenance Expense					**$3,960**		**$7,792**		**$11,651**		**$15,483**		**$19,370**

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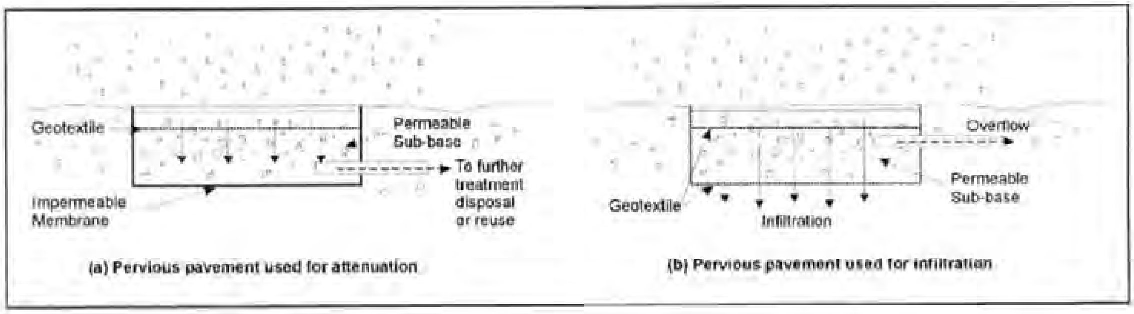

Schematics of a Pervious Pavement System

Alternative Building Materials SD-21

Design Objectives

☑ Maximize Infiltration

☑ Provide Retention

☑ Source Control

Minimize Impervious Land Coverage

Prohibit Dumping of Improper Materials

Contain Pollutant

Collect and Convey

Description

Alternative building materials are selected instead of conventional materials for new construction and renovation. These materials reduce potential sources of pollutants in stormwater runoff by eliminating compounds that can leach into runoff, reducing the need for pesticide application, reducing the need for painting and other maintenance, or by reducing the volume of runoff.

Approach

Alternative building materials are available for use as lumber for decking, roofing materials, home siding, and paving for driveways, decks, and sidewalks.

Suitable Applications

Appropriate applications include residential, commercial and industrial areas planned for development or redevelopment.

Design Considerations

Designing New Installations

Decking

One of the most common materials for construction of decks and other outdoor construction has traditionally been pressure treated wood, which is now being phased out. The standard treatment is called CCA, for chromated copper arsenate. The key ingredients are arsenic (which kills termites, carpenter ants and other insects), copper (which kills the fungi that cause wood to rot) and chromium (which reacts with the other ingredients to bind them to the wood). The amount of arsenic is far from trivial. A deck just 8 feet x 10 feet contains more than 1 1/3 pounds of this highly potent poison. Replacement materials include a new type of pressure treated wood, plastic and composite lumber.

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There are currently over 20 products in the market consisting of plastic or plastic-wood composites. Plastic lumber is made from 100% recycled plastic, # 2 HDPE and polyethylene plastic milk jugs and soap bottles. Plastic-wood composites are a combination of plastic and wood fibers or sawdust. These materials are a long lasting exterior weather, insect, and chemical resistant wood lumber replacement for non structural applications. Use it for decks, docks, raised garden beds and planter boxes, pallets, hand railings, outdoor furniture, animal pens, boat decks, etc.

New pressure treated wood uses a much safer recipe, ACQ, which stands for ammoniacal copper quartenary. It contains no arsenic and no chromium. Yet the American Wood Preservers Association has found it to be just as effective as the standard formula. ACQ is common in Japan and Europe.

Roofing

Several studies have indicated that metal used as roofing material, flashing, or gutters can leach metals into the environment. The leaching occurs because rainfall is slightly acidic and slowly dissolved the exposed metals. Common traditional applications include copper sheathing and galvanized (zinc) gutters.

Coated metal products are available for both roofing and gutter applications. These products eliminate contact of bare metal with rainfall, eliminating one source of metals in runoff. There are also roofing materials made of recycled rubber and plastic that resemble traditional materials.

A less traditional approach is the use of green roofs. These roofs are not just green, they're alive. Planted with grasses and succulents, low- profile green roofs reduce the urban heat island effect, stormwater runoff, and cooling costs, while providing wildlife habitat and a connection to nature for building occupants. These roofs are widely used on industrial facilities in Europe and have been established as experimental installations in several locations in the US, including Portland, Oregon. Their feasibility is questionable in areas of California with prolonged, dry, hot weather.

Paved Areas

Traditionally, concrete is used for construction of patios, sidewalks, and driveways. Although it is non-toxic, these paved areas reduce stormwater infiltration and increase the volume and rate of runoff. This increase in the amount of runoff is the leading cause of stream channel degradation in urban areas.

There are a number of alternative materials that can be used in these applications, including porous concrete and asphalt, modular blocks, and crushed granite. These materials, especially modular paving blocks, are widely available and a well established method to reduce stormwater runoff.

Building Siding

Wood siding is commonly used on the exterior of residential construction. This material weathers fairly rapidly and requires repeated painting to prevent rotting. Alternative "new" products for this application include cement-fiber and vinyl. Cement-fiber siding is a masonry product made from Portland cement, sand, and cellulose and will not burn, cup, swell, or shrink.

Pesticide Reduction
A common use of powerful pesticides is for the control of termites. Chlordane was used for many years for this purpose and is now found in urban streams and lakes nationwide. There are a number of physical barriers that can be installed during construction to help reduce the use of pesticides.

Sand barriers for subterranean termites are a physical deterrent because the termites cannot tunnel through it. Sand barriers can be applied in crawl spaces under pier and beam foundations, under slab foundations, and between the foundation and concrete porches, terraces, patios and steps. Other possible locations include under fence posts, underground electrical cables, water and gas lines, telephone and electrical poles, inside hollow tile cells and against retaining walls.

Metal termite shields are physical barriers to termites which prevent them from building invisible tunnels. In reality, metal shields function as a helpful termite detection device, forcing them to build tunnels on the outside of the shields which are easily seen. Metal termite shields also help prevent dampness from wicking to adjoining wood members which can result in rot, thus making the material more attractive to termites and other pests. Metal flashing and metal plates can also be used as a barrier between piers and beams of structures such as decks, which are particularly vulnerable to termite attack.

Redeveloping Existing Installations

Various jurisdictional stormwater management and mitigation plans (SUSMP, WQMP, etc.) define "redevelopment" in terms of amounts of additional impervious area, increases in gross floor area and/or exterior construction, and land disturbing activities with structural or impervious surfaces. The definition of "redevelopment" must be consulted to determine whether or not the requirements for new development apply to areas intended for redevelopment. If the definition applies, the steps outlined under "designing new installations" above should be followed.

Other Resources

There are no good, independent, comprehensive sources of information on alternative building materials for use in minimizing the impacts of stormwater runoff. Most websites or other references to "green" or "alternative" building materials focus on indoor applications, such as formaldehyde free plywood and low VOC paints, carpets, and pads. Some supplemental information on alternative materials is available from the manufacturers.

Fires are a source of concern in many areas of California. Information on the flammability of alternative decking materials is available from the University of California Forest Product Laboratory (UCFPL) website at: http://www.ucfpl.ucop.edu/WDDeckIntro.htm

Design Considerations

- Tributary Area
- Area Required
- Slope
- Water Availability

Targeted Constituents

☑	Sediment	▲
☑	Nutrients	●
☑	Trash	●
☑	Metals	▲
☑	Bacteria	●
☑	Oil and Grease	▲
☑	Organics	▲

Legend ***(Removal Effectiveness)***

● Low ■ High

▲ Medium

Description

Vegetated swales are open, shallow channels with vegetation covering the side slopes and bottom that collect and slowly convey runoff flow to downstream discharge points. They are designed to treat runoff through filtering by the vegetation in the channel, filtering through a subsoil matrix, and/or infiltration into the underlying soils. Swales can be natural or manmade. They trap particulate pollutants (suspended solids and trace metals), promote infiltration, and reduce the flow velocity of stormwater runoff. Vegetated swales can serve as part of a stormwater drainage system and can replace curbs, gutters and storm sewer systems.

California Experience

Caltrans constructed and monitored six vegetated swales in southern California. These swales were generally effective in reducing the volume and mass of pollutants in runoff. Even in the areas where the annual rainfall was only about 10 inches/yr, the vegetation did not require additional irrigation. One factor that strongly affected performance was the presence of large numbers of gophers at most of the sites. The gophers created earthen mounds, destroyed vegetation, and generally reduced the effectiveness of the controls for TSS reduction.

Advantages

- If properly designed, vegetated, and operated, swales can serve as an aesthetic, potentially inexpensive urban development or roadway drainage conveyance measure with significant collateral water quality benefits.

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- Roadside ditches should be regarded as significant potential swale/buffer strip sites and should be utilized for this purpose whenever possible.

Limitations

- Can be difficult to avoid channelization.
- May not be appropriate for industrial sites or locations where spills may occur
- Grassed swales cannot treat a very large drainage area. Large areas may be divided and treated using multiple swales.
- A thick vegetative cover is needed for these practices to function properly.
- They are impractical in areas with steep topography.
- They are not effective and may even erode when flow velocities are high, if the grass cover is not properly maintained.
- In some places, their use is restricted by law: many local municipalities require curb and gutter systems in residential areas.
- Swales are mores susceptible to failure if not properly maintained than other treatment BMPs.

Design and Sizing Guidelines

- Flow rate based design determined by local requirements or sized so that 85% of the annual runoff volume is discharged at less than the design rainfall intensity.
- Swale should be designed so that the water level does not exceed 2/3rds the height of the grass or 4 inches, which ever is less, at the design treatment rate.
- Longitudinal slopes should not exceed 2.5%
- Trapezoidal channels are normally recommended but other configurations, such as parabolic, can also provide substantial water quality improvement and may be easier to mow than designs with sharp breaks in slope.
- Swales constructed in cut are preferred, or in fill areas that are far enough from an adjacent slope to minimize the potential for gopher damage. Do not use side slopes constructed of fill, which are prone to structural damage by gophers and other burrowing animals.
- A diverse selection of low growing, plants that thrive under the specific site, climatic, and watering conditions should be specified. Vegetation whose growing season corresponds to the wet season are preferred. Drought tolerant vegetation should be considered especially for swales that are not part of a regularly irrigated landscaped area.
- The width of the swale should be determined using Manning's Equation using a value of 0.25 for Manning's n.

Construction/Inspection Considerations

- Include directions in the specifications for use of appropriate fertilizer and soil amendments based on soil properties determined through testing and compared to the needs of the vegetation requirements.
- Install swales at the time of the year when there is a reasonable chance of successful establishment without irrigation; however, it is recognized that rainfall in a given year may not be sufficient and temporary irrigation may be used.
- If sod tiles must be used, they should be placed so that there are no gaps between the tiles; stagger the ends of the tiles to prevent the formation of channels along the swale or strip.
- Use a roller on the sod to ensure that no air pockets form between the sod and the soil.
- Where seeds are used, erosion controls will be necessary to protect seeds for at least 75 days after the first rainfall of the season.

Performance

The literature suggests that vegetated swales represent a practical and potentially effective technique for controlling urban runoff quality. While limited quantitative performance data exists for vegetated swales, it is known that check dams, slight slopes, permeable soils, dense grass cover, increased contact time, and small storm events all contribute to successful pollutant removal by the swale system. Factors decreasing the effectiveness of swales include compacted soils, short runoff contact time, large storm events, frozen ground, short grass heights, steep slopes, and high runoff velocities and discharge rates.

Conventional vegetated swale designs have achieved mixed results in removing particulate pollutants. A study performed by the Nationwide Urban Runoff Program (NURP) monitored three grass swales in the Washington, D.C., area and found no significant improvement in urban runoff quality for the pollutants analyzed. However, the weak performance of these swales was attributed to the high flow velocities in the swales, soil compaction, steep slopes, and short grass height.

Another project in Durham, NC, monitored the performance of a carefully designed artificial swale that received runoff from a commercial parking lot. The project tracked 11 storms and concluded that particulate concentrations of heavy metals (Cu, Pb, Zn, and Cd) were reduced by approximately 50 percent. However, the swale proved largely ineffective for removing soluble nutrients.

The effectiveness of vegetated swales can be enhanced by adding check dams at approximately 17 meter (50 foot) increments along their length (See Figure 1). These dams maximize the retention time within the swale, decrease flow velocities, and promote particulate settling. Finally, the incorporation of vegetated filter strips parallel to the top of the channel banks can help to treat sheet flows entering the swale.

Only 9 studies have been conducted on all grassed channels designed for water quality (Table 1). The data suggest relatively high removal rates for some pollutants, but negative removals for some bacteria, and fair performance for phosphorus.

Table 1 Grassed swale pollutant removal efficiency data							
Removal Efficiencies (% Removal)							
Study	TSS	TP	TN	NO_3	Metals	Bacteria	Type
Caltrans 2002	77	8	67	66	83-90	-33	dry swales
Goldberg 1993	67.8	4.5	-	31.4	42–62	-100	grassed channel
Seattle Metro and Washington Department of Ecology 1992	60	45	-	-25	2–16	-25	grassed channel
Seattle Metro and Washington Department of Ecology, 1992	83	29	-	-25	46–73	-25	grassed channel
Wang et al., 1981	80	-	-	-	70–80	-	dry swale
Dorman et al., 1989	98	18	-	45	37–81	-	dry swale
Harper, 1988	87	83	84	80	88–90	-	dry swale
Kercher et al., 1983	99	99	99	99	99	-	dry swale
Harper, 1988.	81	17	40	52	37–69	-	wet swale
Koon, 1995	67	39	-	9	-35 to 6	-	wet swale

While it is difficult to distinguish between different designs based on the small amount of available data, grassed channels generally have poorer removal rates than wet and dry swales, although some swales appear to export soluble phosphorus (Harper, 1988; Koon, 1995). It is not clear why swales export bacteria. One explanation is that bacteria thrive in the warm swale soils.

Siting Criteria

The suitability of a swale at a site will depend on land use, size of the area serviced, soil type, slope, imperviousness of the contributing watershed, and dimensions and slope of the swale system (Schueler et al., 1992). In general, swales can be used to serve areas of less than 10 acres, with slopes no greater than 5 %. Use of natural topographic lows is encouraged and natural drainage courses should be regarded as significant local resources to be kept in use (Young et al., 1996).

Selection Criteria (NCTCOG, 1993)

- Comparable performance to wet basins
- Limited to treating a few acres
- Availability of water during dry periods to maintain vegetation
- Sufficient available land area

Research in the Austin area indicates that vegetated controls are effective at removing pollutants even when dormant. Therefore, irrigation is not required to maintain growth during dry periods, but may be necessary only to prevent the vegetation from dying.

The topography of the site should permit the design of a channel with appropriate slope and cross-sectional area. Site topography may also dictate a need for additional structural controls. Recommendations for longitudinal slopes range between 2 and 6 percent. Flatter slopes can be used, if sufficient to provide adequate conveyance. Steep slopes increase flow velocity, decrease detention time, and may require energy dissipating and grade check. Steep slopes also can be managed using a series of check dams to terrace the swale and reduce the slope to within acceptable limits. The use of check dams with swales also promotes infiltration.

Additional Design Guidelines

Most of the design guidelines adopted for swale design specify a minimum hydraulic residence time of 9 minutes. This criterion is based on the results of a single study conducted in Seattle, Washington (Seattle Metro and Washington Department of Ecology, 1992), and is not well supported. Analysis of the data collected in that study indicates that pollutant removal at a residence time of 5 minutes was not significantly different, although there is more variability in that data. Therefore, additional research in the design criteria for swales is needed. Substantial pollutant removal has also been observed for vegetated controls designed solely for conveyance (Barrett et al, 1998); consequently, some flexibility in the design is warranted.

Many design guidelines recommend that grass be frequently mowed to maintain dense coverage near the ground surface. Recent research (Colwell et al., 2000) has shown mowing frequency or grass height has little or no effect on pollutant removal.

Summary of Design Recommendations

1) The swale should have a length that provides a minimum hydraulic residence time of at least 10 minutes. The maximum bottom width should not exceed 10 feet unless a dividing berm is provided. The depth of flow should not exceed 2/3rds the height of the grass at the peak of the water quality design storm intensity. The channel slope should not exceed 2.5%.

2) A design grass height of 6 inches is recommended.

3) Regardless of the recommended detention time, the swale should be not less than 100 feet in length.

4) The width of the swale should be determined using Manning's Equation, at the peak of the design storm, using a Manning's n of 0.25.

5) The swale can be sized as both a treatment facility for the design storm and as a conveyance system to pass the peak hydraulic flows of the 100-year storm if it is located "on-line." The side slopes should be no steeper than 3:1 (H:V).

6) Roadside ditches should be regarded as significant potential swale/buffer strip sites and should be utilized for this purpose whenever possible. If flow is to be introduced through curb cuts, place pavement slightly above the elevation of the vegetated areas. Curb cuts should be at least 12 inches wide to prevent clogging.

7) Swales must be vegetated in order to provide adequate treatment of runoff. It is important to maximize water contact with vegetation and the soil surface. For general purposes, select fine, close-growing, water-resistant grasses. If possible, divert runoff (other than necessary irrigation) during the period of vegetation

establishment. Where runoff diversion is not possible, cover graded and seeded areas with suitable erosion control materials.

Maintenance

The useful life of a vegetated swale system is directly proportional to its maintenance frequency. If properly designed and regularly maintained, vegetated swales can last indefinitely. The maintenance objectives for vegetated swale systems include keeping up the hydraulic and removal efficiency of the channel and maintaining a dense, healthy grass cover.

Maintenance activities should include periodic mowing (with grass never cut shorter than the design flow depth), weed control, watering during drought conditions, reseeding of bare areas, and clearing of debris and blockages. Cuttings should be removed from the channel and disposed in a local composting facility. Accumulated sediment should also be removed manually to avoid concentrated flows in the swale. The application of fertilizers and pesticides should be minimal.

Another aspect of a good maintenance plan is repairing damaged areas within a channel. For example, if the channel develops ruts or holes, it should be repaired utilizing a suitable soil that is properly tamped and seeded. The grass cover should be thick; if it is not, reseed as necessary. Any standing water removed during the maintenance operation must be disposed to a sanitary sewer at an approved discharge location. Residuals (e.g., silt, grass cuttings) must be disposed in accordance with local or State requirements. Maintenance of grassed swales mostly involves maintenance of the grass or wetland plant cover. Typical maintenance activities are summarized below:

- Inspect swales at least twice annually for erosion, damage to vegetation, and sediment and debris accumulation preferably at the end of the wet season to schedule summer maintenance and before major fall runoff to be sure the swale is ready for winter. However, additional inspection after periods of heavy runoff is desirable. The swale should be checked for debris and litter, and areas of sediment accumulation.

- Grass height and mowing frequency may not have a large impact on pollutant removal. Consequently, mowing may only be necessary once or twice a year for safety or aesthetics or to suppress weeds and woody vegetation.

- Trash tends to accumulate in swale areas, particularly along highways. The need for litter removal is determined through periodic inspection, but litter should always be removed prior to mowing.

- Sediment accumulating near culverts and in channels should be removed when it builds up to 75 mm (3 in.) at any spot, or covers vegetation.

- Regularly inspect swales for pools of standing water. Swales can become a nuisance due to mosquito breeding in standing water if obstructions develop (e.g. debris accumulation, invasive vegetation) and/or if proper drainage slopes are not implemented and maintained.

Cost

Construction Cost

Little data is available to estimate the difference in cost between various swale designs. One study (SWRPC, 1991) estimated the construction cost of grassed channels at approximately \$0.25 per ft^2. This price does not include design costs or contingencies. Brown and Schueler (1997) estimate these costs at approximately 32 percent of construction costs for most stormwater management practices. For swales, however, these costs would probably be significantly higher since the construction costs are so low compared with other practices. A more realistic estimate would be a total cost of approximately \$0.50 per ft^2, which compares favorably with other stormwater management practices.

Table 2 Swale Cost Estimate (SEWRPC, 1991)

Component	Unit	Extent	Unit Cost			Total Cost		
			Low	Moderate	High	Low	Moderate	High
Mobilization / Demobilization-Light	Swale	1	$107	$274	$441	$107	$274	$441
Site Preparation								
Clearing[b]...............	Acre	0.5	$2,200	$3,800	$5,400	$1,100	$1,900	$2,700
Grubbing[c].............	Acre	0.25	$3,800	$5,200	$6,600	$950	$1,300	$1,650
General Excavation[d]...........	Yd^3	372	$2.10	$3.70	$5.30	$781	$1,376	$1,972
Level and Till[e]........	Yd^2	1,210	$0.20	$0.35	$0.50	$242	$424	$605
Sites Development								
Salvaged Topsoil Seed, and Mulch[f]..	Yd^2	1,210	$0.40	$1.00	$1.60	$484	$1,210	$1,936
Sod[g]......................	Yd^2	1,210	$1.20	$2.40	$3.60	$1,452	$2,904	$4,356
Subtotal	--	--	--	--	--	$5,116	$9,388	$13,660
Contingencies	Swale	1	25%	25%	25%	$1,279	$2,347	$3,415
Total	--	--	--	--	--	$6,395	$11,735	$17,075

Source: (SEWRPC, 1991)

Note: Mobilization/demobilization refers to the organization and planning involved in establishing a vegetative swale.

[a] Swale has a bottom width of 1.0 foot, a top width of 10 feet with 1:3 side slopes, and a 1,000-foot length.

[b] Area cleared = (top width + 10 feet) x swale length.

[c] Area grubbed = (top width x swale length).

[d] Volume excavated = (0.67 x top width x swale depth) x swale length (parabolic cross-section).

[e] Area tilled = $\left(\text{top width} + \frac{8(\text{swale depth}^2)}{3(\text{top width})}\right)$ x swale length (parabolic cross-section).

[f] Area seeded = area cleared x 0.5.

[g] Area sodded = area cleared x 0.5.

Table 3 Estimated Maintenance Costs (SEWRPC, 1991)

Component	Unit Cost	Swale Size (Depth and Top Width)		Comment
		1.5 Foot Depth, One-Foot Bottom Width, 10-Foot Top Width	3-Foot Depth, 3-Foot Bottom Width, 21-Foot Top Width	
Lawn Mowing	$0.85 / 1,000 ft^2/ mowing	$0.14 / linear foot	$0.21 / linear foot	Lawn maintenance area=(top width + 10 feet) x length. Mow eight times per year
General Lawn Care	$9.00 / 1,000 ft^2/ year	$0.18 / linear foot	$0.28 / linear foot	Lawn maintenance area = (top width + 10 feet) x length
Swale Debris and Litter Removal	$0.10 / linear foot / year	$0.10 / linear foot	$0.10 / linear foot	–
Grass Reseeding with Mulch and Fertilizer	$0.30 / yd^2	$0.01 / linear foot	$0.01 / linear foot	Area revegetated equals 1% of lawn maintenance area per year
Program Administration and Swale Inspection	$0.15 / linear foot / year, plus $25 / inspection	$0.15 / linear foot	$0.15 / linear foot	Inspect four times per year
Total	--	**$0.58 / linear foot**	**$ 0.75 / linear foot**	--

Maintenance Cost

Caltrans (2002) estimated the expected annual maintenance cost for a swale with a tributary area of approximately 2 ha at approximately $2,700. Since almost all maintenance consists of mowing, the cost is fundamentally a function of the mowing frequency. Unit costs developed by SEWRPC are shown in Table 3. In many cases vegetated channels would be used to convey runoff and would require periodic mowing as well, so there may be little additional cost for the water quality component. Since essentially all the activities are related to vegetation management, no special training is required for maintenance personnel.

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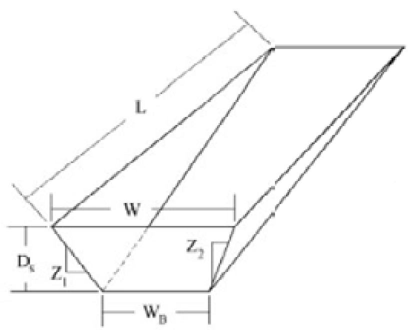
(a) Cross section of swale with check dam.



Notation:

L = Length of swale impoundment area per check dam (ft)
D_S = Depth of check dam (ft)
S_S = Bottom slpe of swale (ft/ft)
W = Top width of check dam (ft)
W_B = Bottom width of check dam (ft)
$Z_{1\&2}$ = Ratio of horizontal to vertical change in swale side slope (ft/ft)

(b) Dimensional view of swale impoundment area.

Design Considerations

- Tributary Area
- Slope
- Water Availability
- Aesthetics

Description

Grassed buffer strips (vegetated filter strips, filter strips, and grassed filters) are vegetated surfaces that are designed to treat sheet flow from adjacent surfaces. Filter strips function by slowing runoff velocities and allowing sediment and other pollutants to settle and by providing some infiltration into underlying soils. Filter strips were originally used as an agricultural treatment practice and have more recently evolved into an urban practice. With proper design and maintenance, filter strips can provide relatively high pollutant removal. In addition, the public views them as landscaped amenities and not as stormwater infrastructure. Consequently, there is little resistance to their use.

California Experience

Caltrans constructed and monitored three vegetated buffer strips in southern California and is currently evaluating their performance at eight additional sites statewide. These strips were generally effective in reducing the volume and mass of pollutants in runoff. Even in the areas where the annual rainfall was only about 10 inches/yr, the vegetation did not require additional irrigation. One factor that strongly affected performance was the presence of large numbers of gophers at most of the southern California sites. The gophers created earthen mounds, destroyed vegetation, and generally reduced the effectiveness of the controls for TSS reduction.

Advantages

- Buffers require minimal maintenance activity (generally just erosion prevention and mowing).
- If properly designed, vegetated, and operated, buffer strips can provide reliable water quality benefits in conjunction with high aesthetic appeal.

Targeted Constituents

☑	Sediment	■
☑	Nutrients	●
☑	Trash	▲
☑	Metals	■
☑	Bacteria	●
☑	Oil and Grease	■
☑	Organics	▲

Legend ***(Removal Effectiveness)***

● Low ■ High

▲ Medium

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- Flow characteristics and vegetation type and density can be closely controlled to maximize BMP effectiveness.
- Roadside shoulders act as effective buffer strips when slope and length meet criteria described below.

Limitations

- May not be appropriate for industrial sites or locations where spills may occur.
- Buffer strips cannot treat a very large drainage area.
- A thick vegetative cover is needed for these practices to function properly.
- Buffer or vegetative filter length must be adequate and flow characteristics acceptable or water quality performance can be severely limited.
- Vegetative buffers may not provide treatment for dissolved constituents except to the extent that flows across the vegetated surface are infiltrated into the soil profile.
- This technology does not provide significant attenuation of the increased volume and flow rate of runoff during intense rain events.

Design and Sizing Guidelines

- Maximum length (in the direction of flow towards the buffer) of the tributary area should be 60 feet.
- Slopes should not exceed 15%.
- Minimum length (in direction of flow) is 15 feet.
- Width should be the same as the tributary area.
- Either grass or a diverse selection of other low growing, drought tolerant, native vegetation should be specified. Vegetation whose growing season corresponds to the wet season is preferred.

Construction/Inspection Considerations

- Include directions in the specifications for use of appropriate fertilizer and soil amendments based on soil properties determined through testing and compared to the needs of the vegetation requirements.
- Install strips at the time of the year when there is a reasonable chance of successful establishment without irrigation; however, it is recognized that rainfall in a given year may not be sufficient and temporary irrigation may be required.
- If sod tiles must be used, they should be placed so that there are no gaps between the tiles; stagger the ends of the tiles to prevent the formation of channels along the strip.
- Use a roller on the sod to ensure that no air pockets form between the sod and the soil.

- Where seeds are used, erosion controls will be necessary to protect seeds for at least 75 days after the first rainfall of the season.

Performance

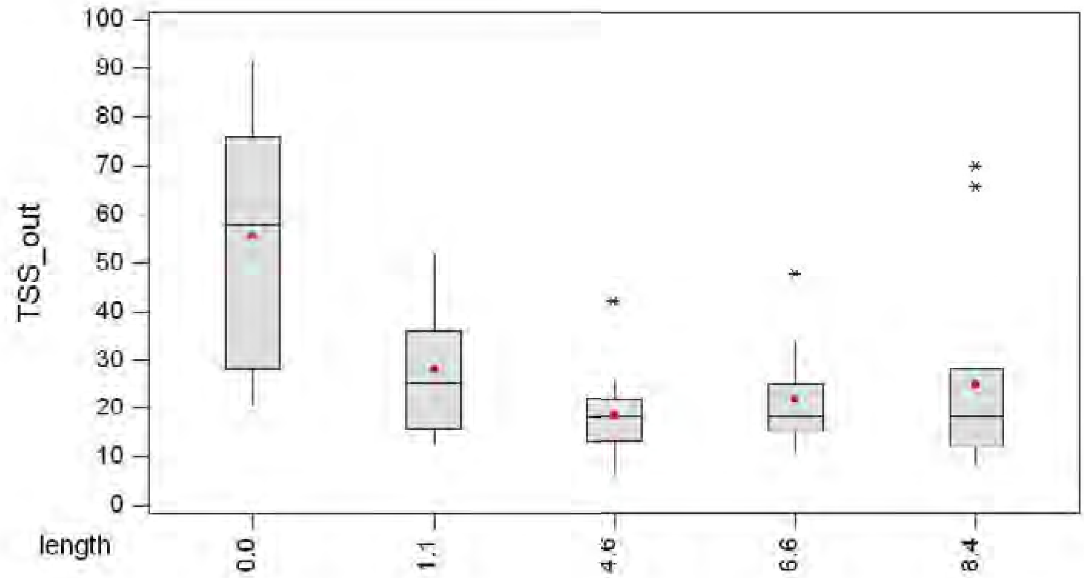
Vegetated buffer strips tend to provide somewhat better treatment of stormwater runoff than swales and have fewer tendencies for channelization or erosion. Table 1 documents the pollutant removal observed in a recent study by Caltrans (2002) based on three sites in southern California. The column labeled "Significance" is the probability that the mean influent and effluent EMCs are not significantly different based on an analysis of variance.

The removal of sediment and dissolved metals was comparable to that observed in much more complex controls. Reduction in nitrogen was not significant and all of the sites exported phosphorus for the entire study period. This may have been the result of using salt grass, a warm weather species that is dormant during the wet season, and which leaches phosphorus when dormant.

Another Caltrans study (unpublished) of vegetated highway shoulders as buffer strips also found substantial reductions often within a very short distance of the edge of pavement. Figure 1 presents a box and whisker plot of the concentrations of TSS in highway runoff after traveling various distances (shown in meters) through a vegetated filter strip with a slope of about 10%. One can see that the TSS median concentration reaches an irreducible minimum concentration of about 20 mg/L within 5 meters of the pavement edge.

Table 1 Pollutant Reduction in a Vegetated Buffer Strip

Constituent	Mean EMC		Removal %	Significance
	Influent (mg/L)	Effluent (mg/L)		P
TSS	119	31	74	<0.000
NO_3-N	0.67	0.58	13	0.367
TKN-N	2.50	2.10	16	0.542
Total N[a]	3.17	2.68	15	-
Dissolved P	0.15	0.46	-206	0.047
Total P	0.42	0.52	-52	0.035
Total Cu	0.058	0.009	84	<0.000
Total Pb	0.046	0.006	88	<0.000
Total Zn	0.245	0.055	78	<0.000
Dissolved Cu	0.029	0.007	77	0.004
Dissolved Pb	0.004	0.002	66	0.006
Dissolved Zn	0.099	0.035	65	<0.000



Filter strips also exhibit good removal of litter and other floatables because the water depth in these systems is well below the vegetation height and consequently these materials are not easily transported through them. Unfortunately little attenuation of peak runoff rates and volumes (particularly for larger events) is normally observed, depending on the soil properties. Therefore it may be prudent to follow the strips with another practice than can reduce flooding and channel erosion downstream.

Siting Criteria

The use of buffer strips is limited to gently sloping areas where the vegetative cover is robust and diffuse, and where shallow flow characteristics are possible. The practical water quality benefits can be effectively eliminated with the occurrence of significant erosion or when flow concentration occurs across the vegetated surface. Slopes should not exceed 15 percent or be less than 1 percent. The vegetative surface should extend across the full width of the area being drained. The upstream boundary of the filter should be located contiguous to the developed area. Use of a level spreading device (vegetated berm, sawtooth concrete border, rock trench, etc) to facilitate overland sheet flow is not normally recommended because of maintenance considerations and the potential for standing water.

Filter strips are applicable in most regions, but are restricted in some situations because they consume a large amount of space relative to other practices. Filter strips are best suited to treating runoff from roads and highways, roof downspouts, small parking lots, and pervious surfaces. They are also ideal components of the "outer zone" of a stream buffer or as pretreatment to a structural practice. In arid areas, however, the cost of irrigating the grass on the practice will most likely outweigh its water quality benefits, although aesthetic considerations may be sufficient to overcome this constraint. Filter strips are generally impractical in ultra-urban areas where little pervious surface exists.

Some cold water species, such as trout, are sensitive to changes in temperature. While some treatment practices, such as wet ponds, can warm stormwater substantially, filter strips do not are not expected to increase stormwater temperatures. Thus, these practices are good for protection of cold-water streams.

Filter strips should be separated from the ground water by between 2 and 4 ft to prevent contamination and to ensure that the filter strip does not remain wet between storms.

Additional Design Guidelines

Filter strips appear to be a minimal design practice because they are basically no more than a grassed slope. In general the slope of the strip should not exceed 15fc% and the strip should be at least 15 feet long to provide water quality treatment. Both the top and toe of the slope should be as flat as possible to encourage sheet flow and prevent erosion. The top of the strip should be installed 2-5 inches below the adjacent pavement, so that vegetation and sediment accumulation at the edge of the strip does not prevent runoff from entering.

A major question that remains unresolved is how large the drainage area to a strip can be. Research has conclusively demonstrated that these are effective on roadside shoulders, where the contributing area is about twice the buffer area. They have also been installed on the perimeter of large parking lots where they performed fairly effectively; however much lower slopes may be needed to provide adequate water quality treatment.

The filter area should be densely vegetated with a mix of erosion-resistant plant species that effectively bind the soil. Native or adapted grasses, shrubs, and trees are preferred because they generally require less fertilizer and are more drought resistant than exotic plants. Runoff flow velocities should not exceed about 1 fps across the vegetated surface.

For engineered vegetative strips, the facility surface should be graded flat prior to placement of vegetation. Initial establishment of vegetation requires attentive care including appropriate watering, fertilization, and prevention of excessive flow across the facility until vegetation completely covers the area and is well established. Use of a permanent irrigation system may help provide maximal water quality performance.

In cold climates, filter strips provide a convenient area for snow storage and treatment. If used for this purpose, vegetation in the filter strip should be salt-tolerant (e.g., creeping bentgrass), and a maintenance schedule should include the removal of sand built up at the bottom of the slope. In arid or semi-arid climates, designers should specify drought-tolerant grasses to minimize irrigation requirements.

Maintenance

Filter strips require mainly vegetation management; therefore little special training is needed for maintenance crews. Typical maintenance activities and frequencies include:

- Inspect strips at least twice annually for erosion or damage to vegetation, preferably at the end of the wet season to schedule summer maintenance and before major fall run-off to be sure the strip is ready for winter. However, additional inspection after periods of heavy run-off is most desirable. The strip should be checked for debris and litter and areas of sediment accumulation.

- Recent research on biofiltration swales, but likely applicable to strips (Colwell et al., 2000), indicates that grass height and mowing frequency have little impact on pollutant removal;

consequently, mowing may only be necessary once or twice a year for safety and aesthetics or to suppress weeds and woody vegetation.

- Trash tends to accumulate in strip areas, particularly along highways. The need for litter removal should be determined through periodic inspection but litter should always be removed prior to mowing.

- Regularly inspect vegetated buffer strips for pools of standing water. Vegetated buffer strips can become a nuisance due to mosquito breeding in level spreaders (unless designed to dewater completely in 48-72 hours), in pools of standing water if obstructions develop (e.g. debris accumulation, invasive vegetation), and/or if proper drainage slopes are not implemented and maintained.

Cost

Construction Cost

Little data is available on the actual construction costs of filter strips. One rough estimate can be the cost of seed or sod, which is approximately 30¢ per ft^2 for seed or 70¢ per ft^2 for sod. This amounts to between $13,000 and $30,000 per acre of filter strip. This cost is relatively high compared with other treatment practices. However, the grassed area used as a filter strip may have been seeded or sodded even if it were not used for treatment. In these cases, the only additional cost is the design. Typical maintenance costs are about $350/acre/year (adapted from SWRPC, 1991). This cost is relatively inexpensive and, again, might overlap with regular landscape maintenance costs.

The true cost of filter strips is the land they consume. In some situations this land is available as wasted space beyond back yards or adjacent to roadsides, but this practice is cost-prohibitive when land prices are high and land could be used for other purposes.

Maintenance Cost

Maintenance of vegetated buffer strips consists mainly of vegetation management (mowing, irrigation if needed, weeding) and litter removal. Consequently the costs are quite variable depending on the frequency of these activities and the local labor rate.

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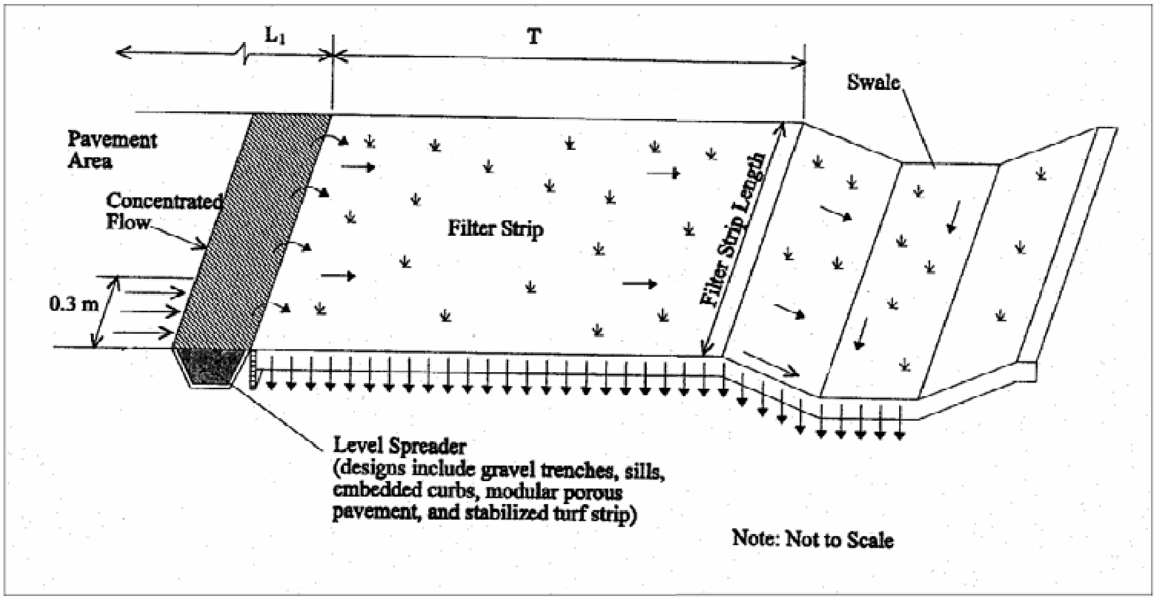
L1
T
Swale
Pavement Area
Concentrated Flow
Filter Strip
Filter Strip Length
0.3 m
Level Spreader
(designs include gravel trenches, sills, embedded curbs, modular porous pavement, and stabilized turf strip)
Note: Not to Scale

Design Considerations

- Soil for Infiltration
- Tributary Area
- Slope
- Aesthetics
- Environmental Side-effects

Description

The bioretention best management practice (BMP) functions as a soil and plant-based filtration device that removes pollutants through a variety of physical, biological, and chemical treatment processes. These facilities normally consist of a grass buffer strip, sand bed, ponding area, organic layer or mulch layer, planting soil, and plants. The runoff's velocity is reduced by passing over or through buffer strip and subsequently distributed evenly along a ponding area. Exfiltration of the stored water in the bioretention area planting soil into the underlying soils occurs over a period of days.

Targeted Constituents

☑	Sediment	■
☑	Nutrients	▲
☑	Trash	■
☑	Metals	■
☑	Bacteria	■
☑	Oil and Grease	■
☑	Organics	■

Legend ***(Removal Effectiveness)***

● Low ■ High

▲ Medium

California Experience

None documented. Bioretention has been used as a stormwater BMP since 1992. In addition to Prince George's County, MD and Alexandria, VA, bioretention has been used successfully at urban and suburban areas in Montgomery County, MD; Baltimore County, MD; Chesterfield County, VA; Prince William County, VA; Smith Mountain Lake State Park, VA; and Cary, NC.

Advantages

- Bioretention provides stormwater treatment that enhances the quality of downstream water bodies by temporarily storing runoff in the BMP and releasing it over a period of four days to the receiving water (EPA, 1999).

- The vegetation provides shade and wind breaks, absorbs noise, and improves an area's landscape.

Limitations

- The bioretention BMP is not recommended for areas with slopes greater than 20% or where mature tree removal would be required since clogging may result, particularly if the BMP receives runoff with high sediment loads (EPA, 1999).

- Bioretention is not a suitable BMP at locations where the water table is within 6 feet of the ground surface and where the surrounding soil stratum is unstable.

- By design, bioretention BMPs have the potential to create very attractive habitats for mosquitoes and other vectors because of highly organic, often heavily vegetated areas mixed with shallow water.

- In cold climates the soil may freeze, preventing runoff from infiltrating into the planting soil.

Design and Sizing Guidelines

- The bioretention area should be sized to capture the design storm runoff.

- In areas where the native soil permeability is less than 0.5 in/hr an underdrain should be provided.

- Recommended minimum dimensions are 15 feet by 40 feet, although the preferred width is 25 feet. Excavated depth should be 4 feet.

- Area should drain completely within 72 hours.

- Approximately 1 tree or shrub per 50 ft^2 of bioretention area should be included.

- Cover area with about 3 inches of mulch.

Construction/Inspection Considerations

Bioretention area should not be established until contributing watershed is stabilized.

Performance

Bioretention removes stormwater pollutants through physical and biological processes, including adsorption, filtration, plant uptake, microbial activity, decomposition, sedimentation and volatilization (EPA, 1999). Adsorption is the process whereby particulate pollutants attach to soil (e.g., clay) or vegetation surfaces. Adequate contact time between the surface and pollutant must be provided for in the design of the system for this removal process to occur. Thus, the infiltration rate of the soils must not exceed those specified in the design criteria or pollutant removal may decrease. Pollutants removed by adsorption include metals, phosphorus, and hydrocarbons. Filtration occurs as runoff passes through the bioretention area media, such as the sand bed, ground cover, and planting soil.

Common particulates removed from stormwater include particulate organic matter, phosphorus, and suspended solids. Biological processes that occur in wetlands result in pollutant uptake by plants and microorganisms in the soil. Plant growth is sustained by the uptake of nutrients from the soils, with woody plants locking up these nutrients through the seasons. Microbial activity within the soil also contributes to the removal of nitrogen and organic matter. Nitrogen is removed by nitrifying and denitrifying bacteria, while aerobic bacteria are responsible for the decomposition of the organic matter. Microbial processes require oxygen and can result in depleted oxygen levels if the bioretention area is not adequately aerated. Sedimentation occurs in the swale or ponding area as the velocity slows and solids fall out of suspension.

The removal effectiveness of bioretention has been studied during field and laboratory studies conducted by the University of Maryland (Davis et al, 1998). During these experiments, synthetic stormwater runoff was pumped through several laboratory and field bioretention areas to simulate typical storm events in Prince George's County, MD. Removal rates for heavy metals and nutrients are shown in Table 1.

Table 1 Laboratory and Estimated Bioretention Davis et al. (1998); PGDER (1993)

Pollutant	Removal Rate
Total Phosphorus	70-83%
Metals (Cu, Zn, Pb)	93-98%
TKN	68-80%
Total Suspended Solids	90%
Organics	90%
Bacteria	90%

Results for both the laboratory and field experiments were similar for each of the pollutants analyzed. Doubling or halving the influent pollutant levels had little effect on the effluent pollutants concentrations (Davis et al, 1998).

The microbial activity and plant uptake occurring in the bioretention area will likely result in higher removal rates than those determined for infiltration BMPs.

Siting Criteria

Bioretention BMPs are generally used to treat stormwater from impervious surfaces at commercial, residential, and industrial areas (EPA, 1999). Implementation of bioretention for stormwater management is ideal for median strips, parking lot islands, and swales. Moreover, the runoff in these areas can be designed to either divert directly into the bioretention area or convey into the bioretention area by a curb and gutter collection system.

The best location for bioretention areas is upland from inlets that receive sheet flow from graded areas and at areas that will be excavated (EPA, 1999). In order to maximize treatment effectiveness, the site must be graded in such a way that minimizes erosive conditions as sheet flow is conveyed to the treatment area. Locations where a bioretention area can be readily incorporated into the site plan without further environmental damage are preferred. Furthermore, to effectively minimize sediment loading in the treatment area, bioretention only should be used in stabilized drainage areas.

Additional Design Guidelines

The layout of the bioretention area is determined after site constraints such as location of utilities, underlying soils, existing vegetation, and drainage are considered (EPA, 1999). Sites with loamy sand soils are especially appropriate for bioretention because the excavated soil can be backfilled and used as the planting soil, thus eliminating the cost of importing planting soil.

The use of bioretention may not be feasible given an unstable surrounding soil stratum, soils with clay content greater than 25 percent, a site with slopes greater than 20 percent, and/or a site with mature trees that would be removed during construction of the BMP.

Bioretention can be designed to be off-line or on-line of the existing drainage system (EPA, 1999). The drainage area for a bioretention area should be between 0.1 and 0.4 hectares (0.25 and 1.0 acres). Larger drainage areas may require multiple bioretention areas. Furthermore, the maximum drainage area for a bioretention area is determined by the expected rainfall intensity and runoff rate. Stabilized areas may erode when velocities are greater than 5 feet per second (1.5 meter per second). The designer should determine the potential for erosive conditions at the site.

The size of the bioretention area, which is a function of the drainage area and the runoff generated from the area is sized to capture the water quality volume.

The recommended minimum dimensions of the bioretention area are 15 feet (4.6 meters) wide by 40 feet (12.2 meters) long, where the minimum width allows enough space for a dense, randomly-distributed area of trees and shrubs to become established. Thus replicating a natural forest and creating a microclimate, thereby enabling the bioretention area to tolerate the effects of heat stress, acid rain, runoff pollutants, and insect and disease infestations which landscaped areas in urban settings typically are unable to tolerate. The preferred width is 25 feet (7.6 meters), with a length of twice the width. Essentially, any facilities wider than 20 feet (6.1 meters) should be twice as long as they are wide, which promotes the distribution of flow and decreases the chances of concentrated flow.

In order to provide adequate storage and prevent water from standing for excessive periods of time the ponding depth of the bioretention area should not exceed 6 inches (15 centimeters). Water should not be left to stand for more than 72 hours. A restriction on the type of plants that can be used may be necessary due to some plants' water intolerance. Furthermore, if water is left standing for longer than 72 hours mosquitoes and other insects may start to breed.

The appropriate planting soil should be backfilled into the excavated bioretention area. Planting soils should be sandy loam, loamy sand, or loam texture with a clay content ranging from 10 to 25 percent.

Generally the soil should have infiltration rates greater than 0.5 inches (1.25 centimeters) per hour, which is typical of sandy loams, loamy sands, or loams. The pH of the soil should range between 5.5 and 6.5, where pollutants such as organic nitrogen and phosphorus can be adsorbed by the soil and microbial activity can flourish. Additional requirements for the planting soil include a 1.5 to 3 percent organic content and a maximum 500 ppm concentration of soluble salts.

Soil tests should be performed for every 500 cubic yards (382 cubic meters) of planting soil, with the exception of pH and organic content tests, which are required only once per bioretention area (EPA, 1999). Planting soil should be 4 inches (10.1 centimeters) deeper than the bottom of the largest root ball and 4 feet (1.2 meters) altogether. This depth will provide adequate soil for the plants' root systems to become established, prevent plant damage due to severe wind, and provide adequate moisture capacity. Most sites will require excavation in order to obtain the recommended depth.

Planting soil depths of greater than 4 feet (1.2 meters) may require additional construction practices such as shoring measures (EPA, 1999). Planting soil should be placed in 18 inches or greater lifts and lightly compacted until the desired depth is reached. Since high canopy trees may be destroyed during maintenance the bioretention area should be vegetated to resemble a terrestrial forest community ecosystem that is dominated by understory trees. Three species each of both trees and shrubs are recommended to be planted at a rate of 2500 trees and shrubs per hectare (1000 per acre). For instance, a 15 foot (4.6 meter) by 40 foot (12.2 meter) bioretention area (600 square feet or 55.75 square meters) would require 14 trees and shrubs. The shrub-to-tree ratio should be 2:1 to 3:1.

Trees and shrubs should be planted when conditions are favorable. Vegetation should be watered at the end of each day for fourteen days following its planting. Plant species tolerant of pollutant loads and varying wet and dry conditions should be used in the bioretention area.

The designer should assess aesthetics, site layout, and maintenance requirements when selecting plant species. Adjacent non-native invasive species should be identified and the designer should take measures, such as providing a soil breach to eliminate the threat of these species invading the bioretention area. Regional landscaping manuals should be consulted to ensure that the planting of the bioretention area meets the landscaping requirements established by the local authorities. The designers should evaluate the best placement of vegetation within the bioretention area. Plants should be placed at irregular intervals to replicate a natural forest. Trees should be placed on the perimeter of the area to provide shade and shelter from the wind. Trees and shrubs can be sheltered from damaging flows if they are placed away from the path of the incoming runoff. In cold climates, species that are more tolerant to cold winds, such as evergreens, should be placed in windier areas of the site.

Following placement of the trees and shrubs, the ground cover and/or mulch should be established. Ground cover such as grasses or legumes can be planted at the beginning of the growing season. Mulch should be placed immediately after trees and shrubs are planted. Two to 3 inches (5 to 7.6 cm) of commercially-available fine shredded hardwood mulch or shredded hardwood chips should be applied to the bioretention area to protect from erosion.

Maintenance

The primary maintenance requirement for bioretention areas is that of inspection and repair or replacement of the treatment area's components. Generally, this involves nothing more than the routine periodic maintenance that is required of any landscaped area. Plants that are appropriate for the site, climatic, and watering conditions should be selected for use in the bioretention cell. Appropriately selected plants will aide in reducing fertilizer, pesticide, water, and overall maintenance requirements. Bioretention system components should blend over time through plant and root growth, organic decomposition, and the development of a natural soil horizon. These biologic and physical processes over time will lengthen the facility's life span and reduce the need for extensive maintenance.

Routine maintenance should include a biannual health evaluation of the trees and shrubs and subsequent removal of any dead or diseased vegetation (EPA, 1999). Diseased vegetation should be treated as needed using preventative and low-toxic measures to the extent possible. BMPs have the potential to create very attractive habitats for mosquitoes and other vectors because of highly organic, often heavily vegetated areas mixed with shallow water. Routine inspections for areas of standing water within the BMP and corrective measures to restore proper infiltration rates are necessary to prevent creating mosquito and other vector habitat. In addition, bioretention BMPs are susceptible to invasion by aggressive plant species such as cattails, which increase the chances of water standing and subsequent vector production if not routinely maintained.

In order to maintain the treatment area's appearance it may be necessary to prune and weed. Furthermore, mulch replacement is suggested when erosion is evident or when the site begins to look unattractive. Specifically, the entire area may require mulch replacement every two to three years, although spot mulching may be sufficient when there are random void areas. Mulch replacement should be done prior to the start of the wet season.

New Jersey's Department of Environmental Protection states in their bioretention systems standards that accumulated sediment and debris removal (especially at the inflow point) will normally be the primary maintenance function. Other potential tasks include replacement of dead vegetation, soil pH regulation, erosion repair at inflow points, mulch replenishment, unclogging the underdrain, and repairing overflow structures. There is also the possibility that the cation exchange capacity of the soils in the cell will be significantly reduced over time. Depending on pollutant loads, soils may need to be replaced within 5-10 years of construction (LID, 2000).

Cost

Construction Cost

Construction cost estimates for a bioretention area are slightly greater than those for the required landscaping for a new development (EPA, 1999). A general rule of thumb (Coffman, 1999) is that residential bioretention areas average about $3 to $4 per square foot, depending on soil conditions and the density and types of plants used. Commercial, industrial and institutional site costs can range between $10 to $40 per square foot, based on the need for control structures, curbing, storm drains and underdrains.

Retrofitting a site typically costs more, averaging $6,500 per bioretention area. The higher costs are attributed to the demolition of existing concrete, asphalt, and existing structures and the replacement of fill material with planting soil. The costs of retrofitting a commercial site in Maryland, Kettering Development, with 15 bioretention areas were estimated at $111,600.

In any bioretention area design, the cost of plants varies substantially and can account for a significant portion of the expenditures. While these cost estimates are slightly greater than those of typical landscaping treatment (due to the increased number of plantings, additional soil excavation, backfill material, use of underdrains etc.), those landscaping expenses that would be required regardless of the bioretention installation should be subtracted when determining the net cost.

Perhaps of most importance, however, the cost savings compared to the use of traditional structural stormwater conveyance systems makes bioretention areas quite attractive financially. For example, the use of bioretention can decrease the cost required for constructing stormwater conveyance systems at a site. A medical office building in Maryland was able to reduce the amount of storm drain pipe that was needed from 800 to 230 feet - a cost savings of $24,000 (PGDER, 1993). And a new residential development spent a total of approximately $100,000 using bioretention cells on each lot instead of nearly $400,000 for the traditional stormwater ponds that were originally planned (Rappahanock,). Also, in residential areas, stormwater management controls become a part of each property owner's landscape, reducing the public burden to maintain large centralized facilities.

Maintenance Cost

The operation and maintenance costs for a bioretention facility will be comparable to those of typical landscaping required for a site. Costs beyond the normal landscaping fees will include the cost for testing the soils and may include costs for a sand bed and planting soil.

References and Sources of Additional Information

Coffman, L.S., R. Goo and R. Frederick, 1999: Low impact development: an innovative alternative approach to stormwater management. Proceedings of the 26th Annual Water Resources Planning and Management Conference ASCE, June 6-9, Tempe, Arizona.

Davis, A.P., Shokouhian, M., Sharma, H. and Minami, C., "Laboratory Study of Biological Retention (Bioretention) for Urban Stormwater Management," *Water Environ. Res.*, 73(1), 5-14 (2001).

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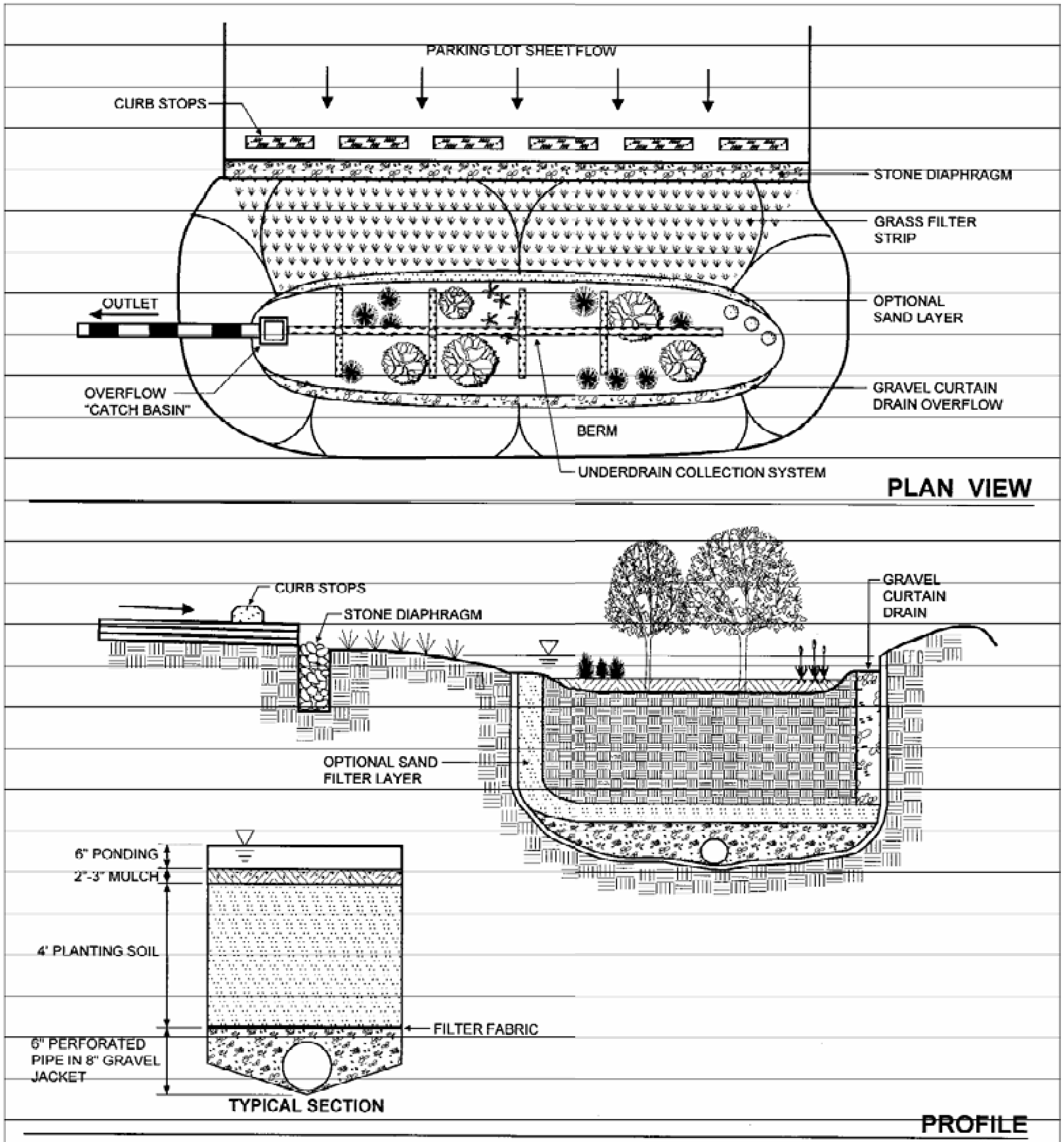
Kim, H., Seagren, E.A., and Davis, A.P., "Engineered Bioretention for Removal of Nitrate from Stormwater Runoff," *WEFTEC 2000 Conference Proceedings on CDROM Research Symposium, Nitrogen Removal*, Session 19, Anaheim CA, October 2000.

Hsieh, C.-h. and Davis, A.P. "Engineering Bioretention for Treatment of Urban Stormwater Runoff," *Watersheds 2002, Proceedings on CDROM Research Symposium*, Session 15, Ft. Lauderdale, FL, Feb. 2002.

Prince George's County Department of Environmental Resources (PGDER), 1993. Design Manual for Use of *Bioretention in Stormwater Management*. Division of Environmental Management, Watershed Protection Branch. Landover, MD.

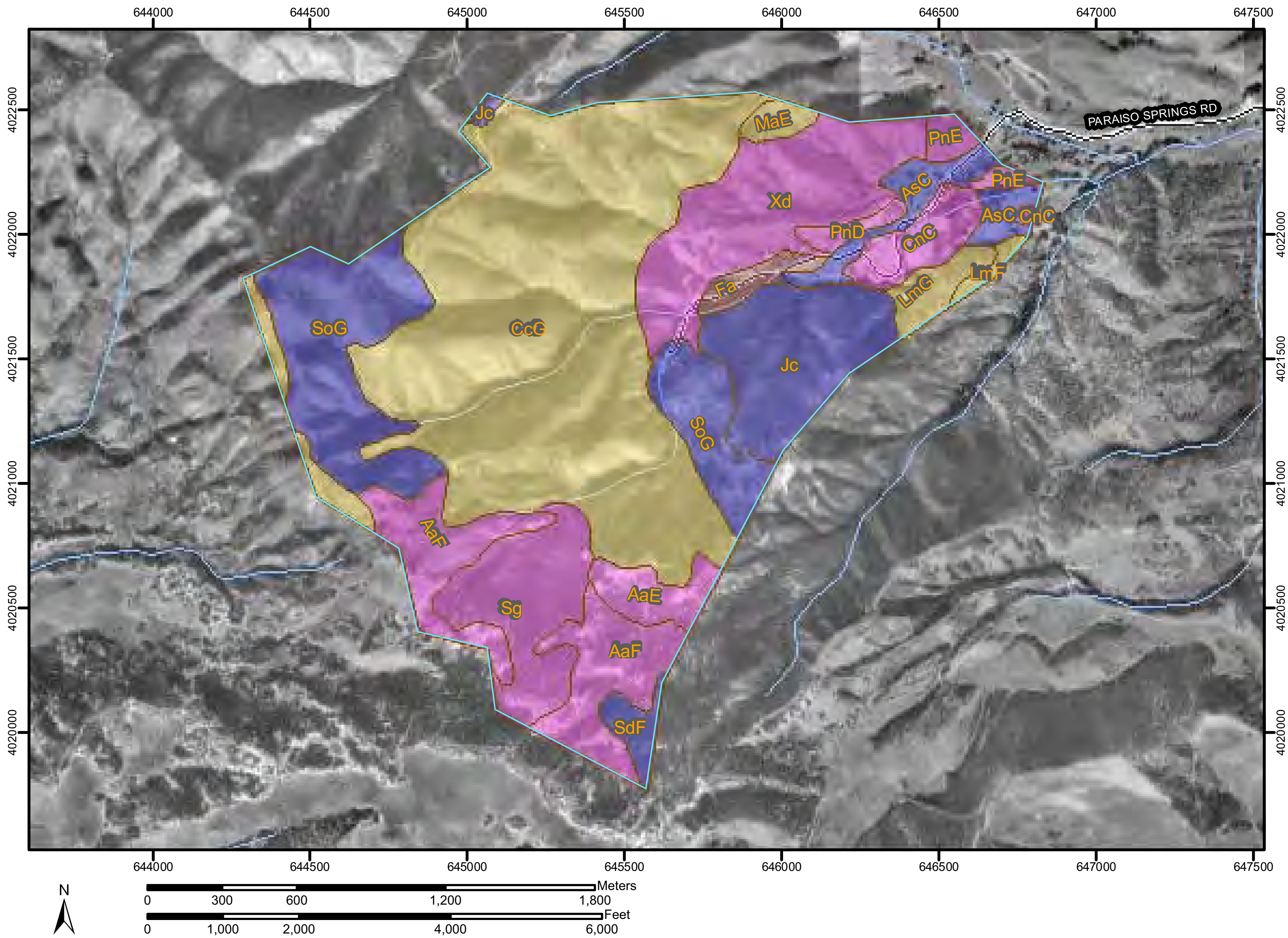
U.S. EPA Office of Water, 1999. Stormwater Technology Fact Sheet: Bioretention. EPA 832-F-99-012.

Weinstein, N. Davis, A.P. and Veeramachaneni, R. "Low Impact Development (LID) Stormwater Management Approach for the Control of Diffuse Pollution from Urban Roadways," *5th International Conference Diffuse/Nonpoint Pollution and Watershed Management Proceedings*, C.S. Melching and Emre Alp, Eds. 2001 International Water Association



Schematic of a Bioretention Facility (MDE, 2000)

Attachment 8
Web Soil Survey 2.0 Output for the Project Site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AsC
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PnD
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CnC
LmF
Fa
LmG
SoG
CcG
Jc
SoG
AaF
AaE
Sg
AaF
SdF
N
Meters
0
300
600
1,200
1,800
Feet
0
1,000
2,000
4,000
6,000

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Units

Soil Ratings
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Political Features

Municipalities
- Cities
- Urban Areas

Water Features
- Oceans
- Streams and Canals

Transportation
- Rails

Roads
- Interstate Highways
- US Routes
- State Highways
- Local Roads
- Other Roads

MAP INFORMATION

Original soil survey map sheets were prepared at publication scale. Viewing scale and printing scale, however, may vary from the original. Please rely on the bar scale on each map sheet for proper map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: UTM Zone 10N

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Monterey County, California
Survey Area Data: Version 7, Dec 10, 2007

Date(s) aerial images were photographed: 5/13/1994

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Hydrologic Soil Group— Summary by Map Unit — Monterey County, California				
Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
AaE	Alo silty clay, 15 to 30 percent slopes	D	23.2	2.0%
AaF	Alo silty clay, 30 to 50 percent slopes	D	102.2	8.8%
AsC	Arroyo Seco gravelly sandy loam, 5 to 9 percent slopes	B	36.7	3.1%
CcG	Cieneba fine gravelly sandy loam, 30 to 75 percent slopes	C	467.3	40.0%
CnC	Cropley silty clay, 2 to 9 percent slopes	D	29.5	2.5%
Fa	Fluvents, stony	A	12.1	1.0%
Jc	Junipero-Sur complex	B	89.5	7.7%
LmF	Los Osos clay loam, 30 to 50 percent slopes	C	5.3	0.5%
LmG	Los Osos clay loam, 50 to 75 percent slopes	C	17.2	1.5%
MaE	McCoy clay loam, 15 to 30 percent slopes	C	9.0	0.8%
PnD	Placentia sandy loam, 9 to 15 percent slopes	D	8.0	0.7%
PnE	Placentia sandy loam, 15 to 30 percent slopes	D	11.5	1.0%
SdF	San Benito clay loam, 30 to 50 percent slopes	B	12.6	1.1%
Sg	Santa Lucia-Reliz association	D	73.7	6.3%
SoG	Sheridan coarse sandy loam, 30 to 75 percent slopes	B	150.2	12.9%
Xd	Xerorthents, dissected	D	119.3	10.2%
Totals for Area of Interest (AOI)			1,167.4	100.0%

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: *None Specified*

Tie-break Rule: Lower

MONTEREY COUNTY

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CURTIS V. WEEKS
GENERAL MANAGER

STREET ADDRESS
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November 24, 2010

Jacqueline Onciano, Planning & Building Services Manager
Monterey County Resource Management Agency
Planning Department
168 W. Alisal Street, 2nd Floor
Salinas, CA 93901

SUBJECT: Paraiso Springs Resort (PLN 040183) Response to Preliminary Engineering Reports for Paraiso Hot Springs Resort, prepared by CH2MHILL, dated August 2010.

Dear Ms. Onciano:

After reviewing the subject reports the Monterey County Water Resources Agency (Agency) has the following comments:

Water Demand

The *Estimated Potable Water Demand and Potable Water Source Technical Memorandum* contained within the subject reports did not include the following assumptions in the water balance calculations:

- Pre-project water use and Pre-project recharge
- Post-project water use for the spa facility
- Post-project recharge

The Agency recommends the Paraiso Springs Resort water balance follow the water balance template prepared for the Omni Subdivision (PC 020344). The revised water balance analyses should be included DEIR.

Drainage Analysis & Drainage Plan

According to the *Hydrology and Hydraulic Analysis and Erosion Control Measures Technical Memorandum* contained within the subject reports, detention ponds are not proposed and stormwater runoff will be mitigated through the use of retention/infiltration facilities. Therefore, the project does not comply with the Agency's standard design policy that requires stormwater detention facilities designed to limit the 100-year post-development runoff rate to the 10-year pre-development rate.

If stormwater retention facilities are proposed, the design criteria should be approved by the Agency prior to the preparation of the preliminary drainage calculations and preliminary drainage plan. Additionally, a geologic report should be included in the DEIR analyzing the suitability of subsurface materials for stormwater retention, and the potential impacts to geologic hazards should be analyzed.

The memorandum did not include information regarding the proposed stormwater retention design criteria, preliminary drainage calculations, or a preliminary drainage plan. These items should be included in the DEIR.

Stream Setback

The Draft EIR should include a site plan showing all proposed development setback 50 feet from *top-of-bank* (as defined in Monterey County Code Chapter 16.16) of the watercourse referred to in the ADEIR as the "Paraiso Springs drainage". If development is proposed within 50 feet of the *top-of-bank*, the DEIR should address the two provisions outlined in Chapter 16.16.050K of the Monterey County Code.

The Agency requests the opportunity to review the water balance analyses, preliminary drainage analysis, preliminary drainage plan, and the stream setback plan prior to the release of the DEIR. If you have any questions, please feel free to contact me at (831) 755-4860.

Sincerely,

Jennifer Bodensteiner

Jennifer Bodensteiner, CFM
Water Resources Hydrologist
Floodplain Management and Development Review Section

Paraiso Springs Resort – Drainage Analysis and Drainage Plan Comments

PREPARED FOR:	John Thompson/Thompson Holdings, LLC
PREPARED BY:	Meabon Burns, PE (CA No. C 71053)/CH2M HILL
COPIES:	David Von Rueden, PE/CH2M HILL File
DATE:	May 2, 2012
PROJECT NUMBER:	434834.03

The purpose of this Technical Memorandum (TM) is to provide responses to Monterey County Water Resources Agency (MCWRA) review comments on the Memorandum titled *Paraiso Springs Resort – Response to Hydrology and Hydraulic Analysis and Erosion Control Measures Review Comments* dated October 28, 2008. MCWRA Comments were provided in a letter from Jennifer Bodensteiner to Jacqueline Onciano dated November 24, 2010. A copy of this letter is included in Attachment 1 for reference.

Response to Drainage Analysis and Drainage Plan Comments

The comments indicate that MCWRA standard design policy "requires stormwater detention facilities designed to limit the 100-year post-development runoff rate to the 10-year pre-development runoff rate." It was further clarified during a conference call on February 12, 2012 that this standard design policy is for a 2-hour storm event.

This TM presents the preliminary design of a detention basin for the Project that is sized to comply with the MCWRA standard design policy. A hydrologic analysis was developed to support this preliminary design utilizing data from the 2008 Memorandum.

Hydrologic Analysis

A hydrologic analysis was developed to comply with the MCWRA's standard design policy that the 100-year post-development runoff rate must be limited to the 10-year pre-development runoff rate for a 2 hour storm event. This analysis was conducted using the Rational Method to calculate peak storm runoff

$$Q = KCiA$$

where Q is the peak runoff rate, K is 1.0 in U.S customary units, C is the runoff coefficient, i is the average rainfall intensity for a specific return period and duration (t_c), and A is the drainage area (Mays, 2001).

Rainfall intensity, i, was calculated using the equation and data provided on Plate 25 (MCDPW, 1977) for return periods of 10-years and 100-years as required by the standard design policy. The duration, also known as time of concentration (t_c), used in the rainfall intensity calculations was developed using the US Soil Conservation Service (SCS) lag equation, which is an empirical equation that requires the longest flow path, SCS curve number (CN), and average watershed slope as inputs (Mays 2001).

Analysis Results

The results of the revised hydrologic analysis are shown below. Calculations are included in Attachment 2. Supporting documentation for these calculations is included in Attachment 3 through Attachment 6.

Table 1 summarizes the 10-year pre-development runoff rates by subbasin.

TABLE 1
10-Year Pre-Development Runoff Rates

Subbasin	C (-)	*i* (in/hr)	A (acres)	Q (cfs)
N	0.41	0.33	17.5	2.4
S	0.41	0.28	17.8	2.0
V	0.41	0.29	44.2	4.6
			TOTAL	9.0

in/hr = inches per hour
cfs = cubic feet per second

Table 2 summarizes the 100-year post-development runoff rates by subbasin.

TABLE 2
100-Year Post-Development Runoff Rates

Subbasin	C (-)	*i* (in/hr)	A (acres)	Q (cfs)
N	0.74	0.60	17.5	7.3
S	0.70	0.50	17.8	7.1
V	0.62	0.49	44.2	12.4
			TOTAL	26.8

in/hr = inches per hour
cfs = cubic feet per second

Table 3 compares the runoff volume that will need to be detained onsite to comply with the MCWRA standard design policy.

TABLE 3
Onsite Detention Volume Required for Compliance

Table Head	2-Hour Volume (CF)	2 Hour Volume (MG)	2 Hour Volume (ac-ft)
100-year Post-Development	192,740	1.5	4.4
10-year Pre-Development	64,820	0.5	1.5
Difference	127,920	1.0	2.9

CF = cubic feet
MG = million gallons
ac-ft = acre feet

Based on this analysis, the Project will include a detention basin sized to hold a minimum of 2.9 acre-feet. The detention basin will be approximately 100 feet by 100 feet at the bottom with side slopes of 2:1 and a depth of 10 feet. The proposed location for the detention basin is shown on the site map in Attachment 3.

References

Mays, Larry W. 2001. *Water Resources Engineering*. John Wiley & Sons, Inc. 1st ed.

Monterey County Department of Public Works (MCDPW). 24 October 1977. *Standard Details Rainfall Intensities Chart, Plate 25.*

Monterey County Water Resources Agency (MCWRA). Jennifer Bodensteiner, CFM. 24 November 2010. *Paraiso Springs Resort (PLN 040183) Response to Preliminary Engineering Reports for Paraiso Hot Springs Resort, prepared by CH2M HILL, dated August 2010.*

Natural Resources Conservation Service (NRCS), United States Department of Agriculture. 12 April 2011. Web Soil Survey 2.3. < http://websoilsurvey.nrcs.usda.gov/>. 2 May 2012.

Attachments

1. MCWRA Comment Letter
2. Hydrologic Analysis Calculations
3. Project Site Map and Subbasin Delineation
4. Proposed Developed Area Calculations
5. Web Soil Survey 2.3 Output for the Project Site
6. Curve Number Determination

Attachment 1
MCWRA Comment Letter

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November 24, 2010

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If stormwater retention facilities are proposed, the design criteria should be approved by the Agency prior to the preparation of the preliminary drainage calculations and preliminary drainage plan. Additionally, a geologic report should be included in the DEIR analyzing the suitability of subsurface materials for stormwater retention, and the potential impacts to geologic hazards should be analyzed.

The memorandum did not include information regarding the proposed stormwater retention design criteria, preliminary drainage calculations, or a preliminary drainage plan. These items should be included in the DEIR.

Stream Setback

The Draft EIR should include a site plan showing all proposed development setback 50 feet from *top-of-bank* (as defined in Monterey County Code Chapter 16.16) of the watercourse referred to in the ADEIR as the "Paraiso Springs drainage". If development is proposed within 50 feet of the *top-of-bank*, the DEIR should address the two provisions outlined in Chapter 16.16.050K of the Monterey County Code.

The Agency requests the opportunity to review the water balance analyses, preliminary drainage analysis, preliminary drainage plan, and the stream setback plan prior to the release of the DEIR. If you have any questions, please feel free to contact me at (831) 755-4860.

Sincerely,

Jennifer Bodensteiner

Jennifer Bodensteiner, CFM
Water Resources Hydrologist
Floodplain Management and Development Review Section

Attachment 2
Hydrologic Analysis Calculations

PARAISO SPRINGS RESORT

Pre-Development Subbasin Details

Subbasin [a]	Area (SF)	Area (acres)	Longest Flow Path, L (LF)	Max Elev	Min Elev	Slope, S (%)	CN [b]
N	762,317	17.50	2,933	1,110	880	7.8%	69
S	776,457	17.83	3,564	1,193	880	8.8%	72
V	1,927,175	44.24	3,546	1,295	1,005	8.2%	62
Watershed	3,465,949	79.57					

a. Subbasins are delineated in Attachment 3

b. Composite CN based on Table 8.7.3 (Mays, 2001), see Attachment 6

PARAISO SPRINGS RESORT

Post-Development Subbasin Details

Subbasin [a]	Area (SF)	Area (acres)	Longest Flow Path, L (LF)	Max Elev	Min Elev	Slope, S (%)	CN [b]	Impervious [c] (SF)	Impervious [c] (acres)	Impervious [c] (% area)
N	762,317	17.50	2933	1110	880	7.8%	85	384,320	8.82	50%
S	776,457	17.83	3564.00	1193.00	880.00	8.8%	87	332,675	7.64	43%
V	1,927,175	44.24	3781	1315	1005	8.2%	74	445,100	10.22	23%
Watershed	3,465,949	79.57						1,162,095	26.68	34%

a. Subbasins are delineated in Attachment 3

b. Composite CN based on Table 8.7.3 (Mays, 2001), See Attachment 6

c. All developed areas are assumed to be impervious, see Attachment 4

PARAISO SPRINGS RESORT

Time of Concentration (t_c) Calculations

SCS Lag Equation

t_c = (100*L^0.8*[(1000/CN)-9]^0.7)/(1900*S^0.5)

Table 15.2.4 (Mays, 2001)

10-Year Pre-Development

Subbasin	L (LF)	CN	S (%)	t_c (min)
N	2,933	76	7.8%	307
S	3,564	67	8.8%	431
V	3,546	62	8.2%	503

100-Year Post-Development

Subbasin	L (LF)	CN	S (%)	t_c (min)
N	2,933	85	7.8%	228
S	3,564	87	8.8%	234
V	3,546	74	8.2%	365

In the rational method each sewer is designed individually and independently (except for the computation of sewer flow time) and the corresponding rainfall intensity i is computed repeatedly for the area drained by the sewer. For a given sewer, all the different areas drained by this sewer have the same i. Thus, as the design progresses towards the downstream sewers, the drainage area increases and usually the time of concentration increases accordingly. This increasing t_c in turn gives a decreasing i that should be applied to the entire area drained by the sewer.

Inlet times, or times of concentration for the case of no upstream sewers, can be computed using a number of methods, some of which are presented in Table 15.2.4. The longest time of concentration among the times for the various flow routes in the drainage area is the critical time of concentration used.

Table 15.2.4 Summary of Time of Concentration Formulas

Method and Date	Formula for t_c (min)	Remarks
Kirpich (1940)	$t_c = 0.0078L^{0.77}S^{-0.385}$ L = length of channel/ditch from headwater to outlet, ft S = average watershed slope, ft/ft	Developed from SCS data for seven rural basins in Tennessee with well-defined channel and steep slopes (3% to 10%); for overland flow on concrete or asphalt surfaces multiply t_c by 0.4; for concrete channels multiply by 0.2; no adjustments for overland flow on bare soil or flow in roadside ditches.
California Culverts Practice (1942)	$t_c = 60(11.9L^3/H)^{0.385}$ L = length of longest watercourse, mi H = elevation difference between divide and outlet, ft	Essentially the Kirpich formula; developed from small mountainous basins in California (U.S. Bureau of Reclamation, 1973, 1987).
Izzard (1946)	$t_c = \dfrac{41.025(0.0007i+c)L^{0.33}}{S^{0.333}i^{0.667}}$ i = rainfall intensity, in/h c = retardance coefficient L = length of flow path, ft S = slope of flow path, ft/ft	Developed in laboratory experiments by Bureau of Public Roads for overland flow on roadway and turf surfaces; values of the retardance coefficient range from 0.0070 for very smooth pavement to 0.012 for concrete pavement to 0.06 for dense turf; solution requires iteration; product i times L should be < 500.
Federal Aviation Administration (1970)	$t_c = 1.8(1.1-C)L^{0.50}/S^{0.333}$ C = rational method runoff coefficient L = length of overland flow, ft S = surface slope, %	Developed from airfield drainage data assembled by the Corps of Engineers; method is intended for use on airfield drainage problems, but has been used frequently for overland flow in urban basins.
Kinematic wave formulas (Morgali and Linsley (1965); Aron and Erborge (1973))	$t_c = \dfrac{0.94L^{0.6}n^{0.6}}{(i^{0.4}S^{0.3})}$ L = length of overland flow, ft n = Manning roughness coefficient i = rainfall intensity in/h S = average overland slope ft/ft	Overland flow equation developed from kinematic wave analysis of surface runoff from developed surfaces; method requires iteration since both i (rainfall intensity) and t_c are unknown; superposition of intensity–duration–frequency curve gives direct graphical solution for t_c.
SCS lag equation (U.S. Soil Conservation Service (1975))	$t_c = \dfrac{100L^{0.8}[(1000/\mathrm{CN})-9]^{0.7}}{1900S^{0.5}}$ L = hydraulic length of watershed (longest flow path), ft CN = SCS runoff curve number S = average watershed slope, %	Equation developed by SCS from agricultural watershed data; it has been adapted to small urban basins under 2000 acres; found generally good where area is completely paved; for mixed areas it tens to overestimate; adjustment factors are applied to correct for channel improvement and impervious area; the equation assumes that t_c = 1.67 × basin lag.

PARAISO SPRINGS RESORT

Rainfall Intensity Calculations

From Plate 25 (MCDPW, 1977)

2 year i	0.5	in/hr
10 year i	0.74	in/hr
100 year i	1.1	in/hr

$i_t = 7.75 * i / \text{sqrt}(t_c)$

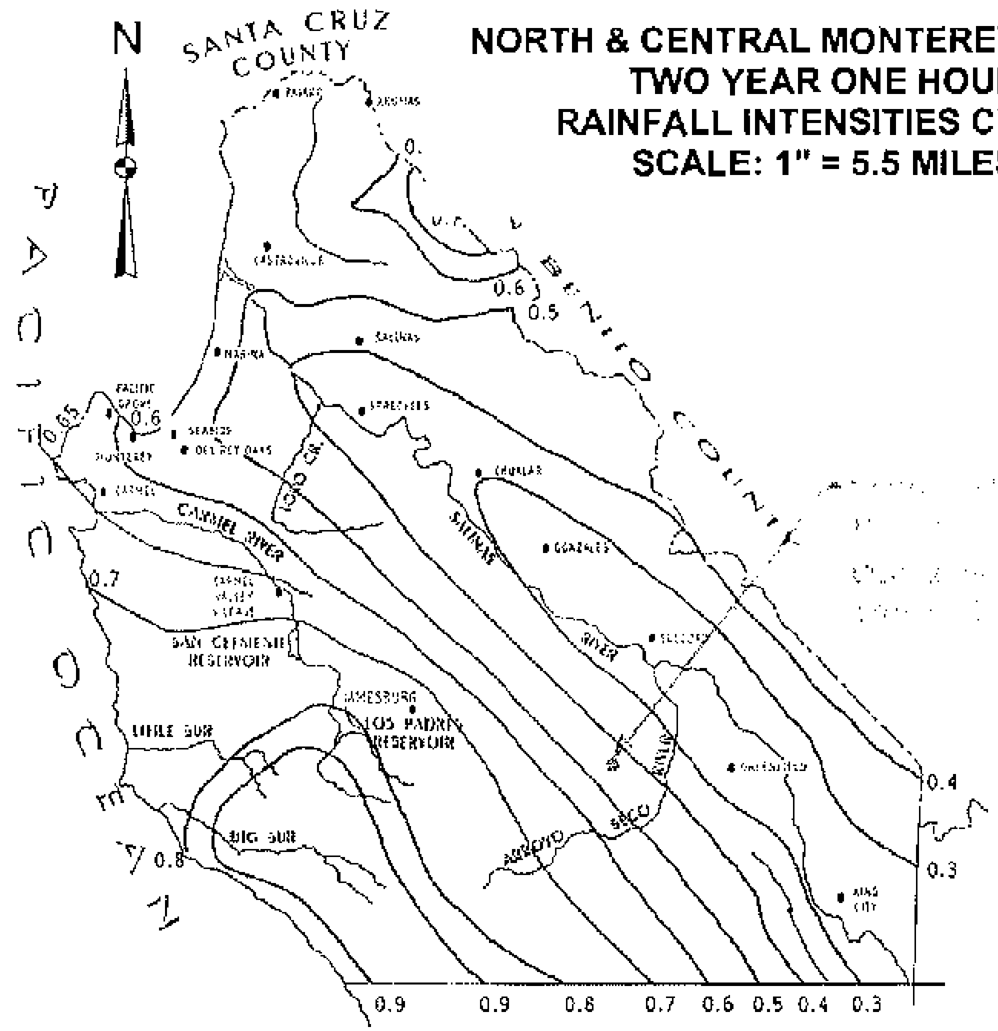
10-Year Pre-Development

Subbasin	i (in/hr)	t_c (min)	i_t (in/hr)
N	0.74	307	0.33
S	0.74	431	0.28
V	0.74	503	0.26

100-Year Post-Development

Subbasin	i (in/hr)	t_c (min)	i_t (in/hr)
N	1.1	228	0.57
S	1.1	234	0.56
V	1.1	365	0.45

NORTH & CENTRAL MONTEREY COUNTY
TWO YEAR ONE HOUR
RAINFALL INTENSITIES CHART
SCALE: 1" = 5.5 MILES



NOTE:

1. Intensities for particular location in the Southern part of the County available from County Surveyors Office.
2. Conversion Factors:
   Intensity of a 10-year design storm equals 2-year design storm times 1.48
   Intensity of a 25-year design storm equals 2-year design storm times 1.73
   Intensity of a 50-year design storm equals 2-year design storm times 1.92
   Intensity of a 100-year design storm equals 2-year design storm times 2.22
3. The maximum Intensity (I_t) for storms of various in duration is determined by the formula: $I_t = 7.75i/\sqrt{t}$ in which variables are as follows:
   I_t = maximum intensity of storm of t minutes duration
   i = one hour rainfall intensity from above chart and note 2
   t = time in minutes shortest time it takes storm runoff to flow from farthest point in the drainage area to the point in question
4. **Example:** Find maximum intensity of 20 minute storm in Chualar, expected to occur on the average of once in 25 years.
   Solution: From chart 0.3/hr intensity for 2-year design storm.
   From note 2, 0.3 times 1.73 equals 0.52"/hr the maximum intensity of a 25-year one hour design storm.
   From note 3, $I_t = 7.75i/\sqrt{t} = (7.75)(0.52)/\sqrt{20} = 0.90$"/hr. Therefore, the maximum 20 minute intensity of a storm that on the average would occur once every 25 years would be 0.90"/hr.

MONTEREY COUNTY	DEPT. OF PUBLIC WORKS
STANDARD DETAILS RAINFALL INTENSITIES CHART	
APPROVED	DATE
REVISED	PLATE NO. 25

PARAISO SPRINGS RESORT

Rational Method

Q = KCitA   K = 1 for US customary units

	C	Notes
10-yr Pre	0.41	10-yr forest/woodland; steep, over 7%
100-yr Post	0.95	100-yr Asphaltic
	0.53	100-yr Fair condition (grass over 50% to 75% of the area); steep, over 7%

Table 15.2.3 (Mays, 2001)

10-Year Pre-Development

Subbasin	C	i_t (in/hr)	A (acres)	Q (cfs)
N	0.41	0.33	17.50	2.35
S	0.41	0.28	17.83	2.02
V	0.41	0.26	44.24	4.64
				9.00

100-Year Post-Development

Subbasin	% Impervious	Weighted C
N	50%	0.74
S	43%	0.71
V	23%	0.63

C = % Impervious * 0.95 + (1 - % Impervious) * 0.53

Sub Basin	C	i_t (in/hr)	A (acres)	Q (cfs)
N	0.74	0.57	17.50	7.34
S	0.71	0.56	17.83	7.06
V	0.63	0.45	44.24	12.38
				26.77

Table 15.2.2 Technical Items and Limitations to Consider in Storm Sewer Design (*continued*)

Minimum size of pipe	12–24 in (0.3–0.6 m)
Vertical alignment at manholes:	
Different size pipe	Match crown of pipe or 80 to 85% depth lines
Same size pipe	Minimum of 0.1–0.2 ft (0.03–0.06 m) in invert drop
Minimum depth of soil cover	12–24 in (0.3–0.6 m)
Final hydraulic design	Check design for surcharge and junction losses by using backwater analysis
Location of inlets	In street where the allowable gutter flow capacity is exceeded

Source: Urbonas and Roesner (1993).

15.2.2 Rational Method Design

From an engineering viewpoint the design can be divided into two main aspects: runoff prediction and pipe sizing. The rational method, which can be traced back to the mid-nineteenth century, is still probably the most popular method used for the design of storm sewers (Yen and Akan, 1999). Although criticisms have been raised of its adequacy, and several other more advanced methods have been proposed, the rational method, because of its simplicity, is still in continued use for sewer design when high accuracy of runoff rate is not essential.

Using the rational method, the storm runoff peak is estimated by the rational formula

$$Q = KCiA \qquad (15.2.1)$$

where the peak runoff rate Q is in ft^3/s (m^3/s), K is 1.0 in U.S. customary units (0.28 for SI units), C is the runoff coefficient (Table 15.2.3), i is the average rainfall intensity in in/hr (mm/hr) from intensity-duration frequency relationships for a specific return period and duration t_c in min, and A is the area of the tributary drainage area in acres (km^2). The duration is taken as the time of concentration t_c of the drainage area.

Table 15.2.3 Runoff Coefficients for Use in the Rational Method

	Return Period (years)						
Character of Surface	2	5	10	25	50	100	500
Developed							
Asphaltic	0.73	0.77	0.81	0.86	0.90	0.95	1.00
Concrete/roof	0.75	0.80	0.83	0.88	0.92	0.97	1.00
Grass areas (lawns, parks, etc.)							
Poor condition (grass cover less than 50% of the area)							
Flat, 0–2%	0.32	0.34	0.37	0.40	0.44	0.47	0.58
Average, 2–7%	0.37	0.40	0.43	0.46	0.49	0.53	0.61
Steep, over 7%	0.40	0.43	0.45	0.49	0.52	0.55	0.62
Fair condition (grass cover 50% to 75% of the area)							
Flat, 0–2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2–7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Good condition (grass cover larger than 75% of the area)							
Flat, 0–2%	0.21	0.23	0.25	0.29	0.32	0.36	0.49
Average, 2–7%	0.29	0.32	0.35	0.39	0.42	0.46	0.56
Steep, over 7%	0.34	0.37	0.40	0.44	0.47	0.51	0.58

Table 15.2.3 Runoff Coefficients for Use in the Rational Method (*continued*)

Character of Surface	Return Period (years)						
	2	5	10	25	50	100	500
Undeveloped							
Cultivated land							
Flat, 0–2%	0.31	0.34	0.36	0.40	0.43	0.47	0.57
Average, 2–7%	0.35	0.38	0.41	0.44	0.48	0.51	0.60
Steep, over 7%	0.39	0.42	0.44	0.48	0.51	0.54	0.61
Pasture/range							
Flat, 0–2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2–7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Forest/woodlands							
Flat, 0–2%	0.30	0.25	0.28	0.31	0.35	0.39	0.48
Average, 2–7%	0.31	0.34	0.26	0.40	0.43	0.47	0.56
Steep, over 7%	0.35	0.39	0.41	0.45	0.48	0.52	0.58

Note: The values in the table are the standards used by the City of Austin, Texas.

Source: Chow, Maidment, and Mays (1988).

In urban areas, the drainage area usually consists of subareas or subcatchments of substantially different surface characteristics. As a result, a composite analysis is required that must take into account the various surface characteristics. The areas of the subcatchments are denoted by A_j and the runoff coefficients for each subcatchment are denoted by C_j. Then the peak runoff is computed using the following form of the rational formula:

$$Q = Ki\sum_{j=1}^{m} C_j A_j \tag{15.2.2}$$

where m is the number of subcatchments drained by a sewer.

The *rainfall intensity* i is the average rainfall rate considered for a particular drainage basin or subbasin. The intensity is selected on the basis of design rainfall duration and design frequency of occurrence. The design duration is equal to the time of concentration for the drainage area under consideration. The frequency of occurrence is a statistical variable that is established by design standards or chosen by the engineer as a design parameter.

The *time of concentration* t_c used in the rational method is the time associated with the peak runoff from the watershed to the point of interest. Runoff from a watershed usually reaches a peak at the time when the entire watershed is contributing; in this case, the time of concentration is the time for a drop of water to flow from the remotest point in the watershed to the point of interest. Runoff may reach a peak prior to the time the entire watershed is contributing. A trial-and-error procedure can be used to determine the critical time of concentration. The time of concentration to any point in a storm drainage system is the sum of the inlet time t_0 and the flow time t_f in the upstream sewers connected to the catchment, that is,

$$t_c = t_0 + t_f \tag{15.2.3}$$

where the flow time is

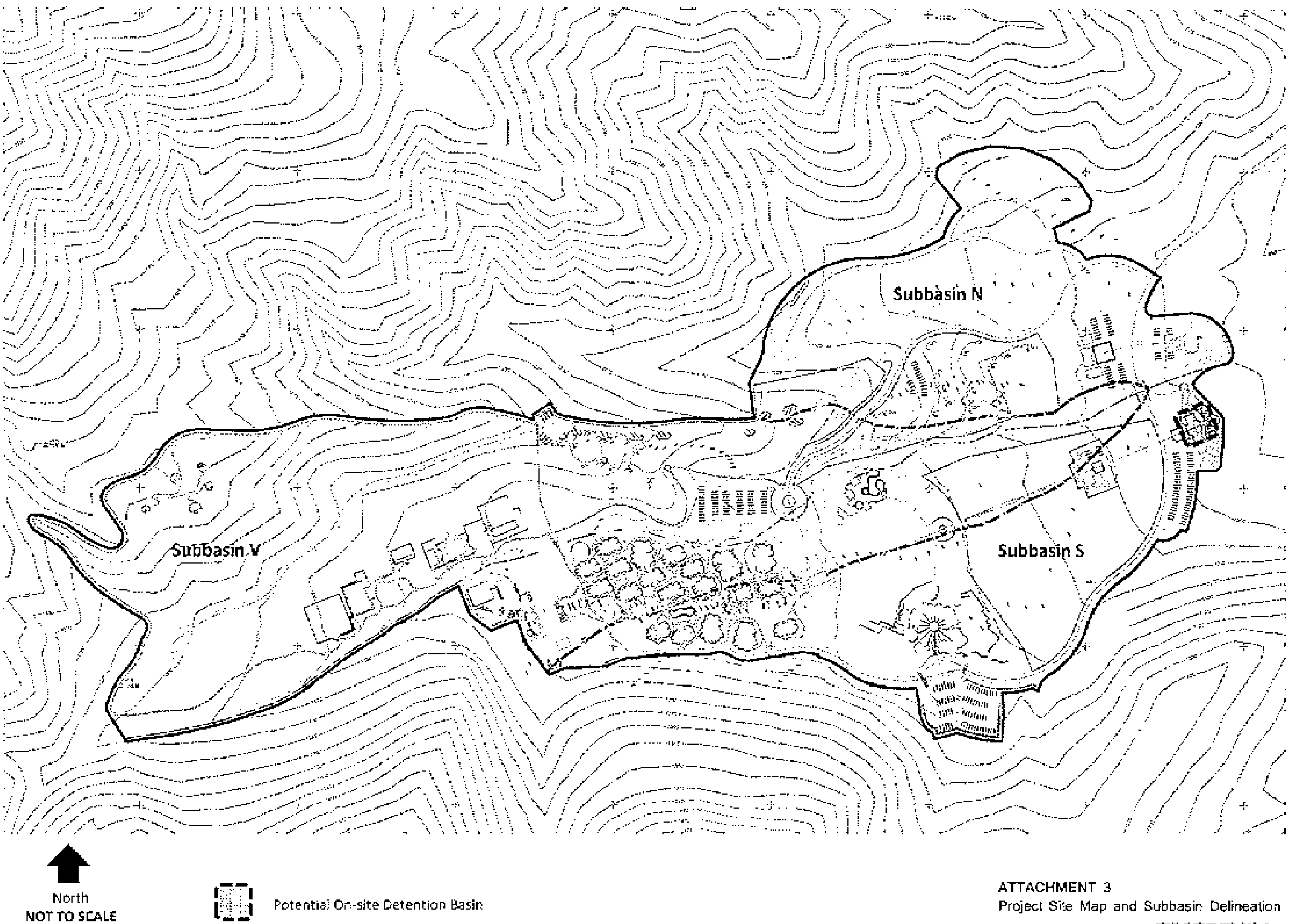
$$t_f = \sum \frac{L_j}{V_j} \tag{15.2.4}$$

where L_j is the length of the jth pipe along the flow path in ft (m) and V_j is the average flow velocity in the pipe in ft/s (m/s). The inlet time t_0 is the longest time of overland flow of water in a catchment to reach the storm sewer inlet draining the catchment.

PARAISO SPRINGS RESORT
Runoff Rates and Volumes

Return Period	Q (cfs)	Q (CF/hr)	2 hour (CF)	2 hour (MG)	2 hour (ac-ft)
100-Year Post-Development	26.77	96,370	192,739	1.44	4.42
10-Year Pre-Development	9.00	32,411	64,822	0.48	1.49
Difference	**17.77**	**63,959**	**127,917**	**0.96**	**2.94**

Attachment 3
Project Site Map and Subbasin Delineation



ATTACHMENT 3
Project Site Map and Subbasin Delineation

CH2MHILL®

Attachment 4
Proposed Developed Area Calculations

CH2MHILL.

Job No. 434834.03
Sheet No. 1 of 4
Date 5/1/2012
Computed By M. Burns
Checked By

Job Name Paraiso Springs Resort
Subject Proposed Developed Areas

Subbasin N

4-plex condos	10 condos x 2,800 SF = 28,000 SF
Paths	1,420 SF
Roads	25' wide x (1,450' + 4,450') long = 147,500 SF
Parking	3,000 SF + 10,000 SF = 13,000 SF
Homes	180,000 SF
Misc buildings	9,200 SF + 5,200 SF = 14,400 SF
TOTAL	28,000 + 1420 + 147,500 + 13,000 + 180,000 + 14,400
	= 384,320 SF

CH2MHILL.

Job Name Paraiso Springs Resort

Subject Proposed Developed Areas

Job No. 434834.03

Sheet No. 2 of 4

Date 5/1/2012

Computed By

Checked By

Subbasin S

Parking	27,800SF + 13,500SF + 17,300SF = 58,600 SF
Roads	25' wide x (430' + 1,200' + 185' + 800') long = 65,375 SF
Paths	3,000 SF
Misc. walkways	6' wide x 1,300' long = 7,800 SF
Bungalows	10 bungalows x 1,700 SF = 17,000 SF
Misc. buildings	15,300SF + 132,500 SF + 12,800 SF + 20,300SF = 180,900 SF
TOTAL	58,600 + 65,375 + 3,000 + 7,800 + 17,000 + 180,900
	= 332,675 SF

CH2MHILL.

Job Name: Paraiso Springs Resort	Job No. 434834.03
Subject: Proposed Developed Areas	Sheet No. 3 of 4
	Date 5/1/2012
	Computed By M. Burns
	Checked By

Subbasin V

Paths	6' wide x (1,900' + 8,100' + 3,500') long + 1,000 SF = 82,000 SF
Roads	25' wide x 1,550' long + 1,900 SF = 40,650 SF
Parking	13,800 SF + 29,000 SF = 42,800 SF
Pools	3,600 SF
4-Plex Condos	20 condos x 2,800 SF = 56,000 SF
Bungalows	16 bungalows x 1700 SF = 27,200 SF
Misc buildings	1,500 SF + 35,100 SF + 85,700 SF + 38,500 SF + 26,700 SF + 5,350 SF = 192,850 SF

TOTAL

82,000 + 40,650 + 42,800 + 3,600 + 56,000 + 27,200 + 192,850

= 445,100 SF

CH2MHILL.

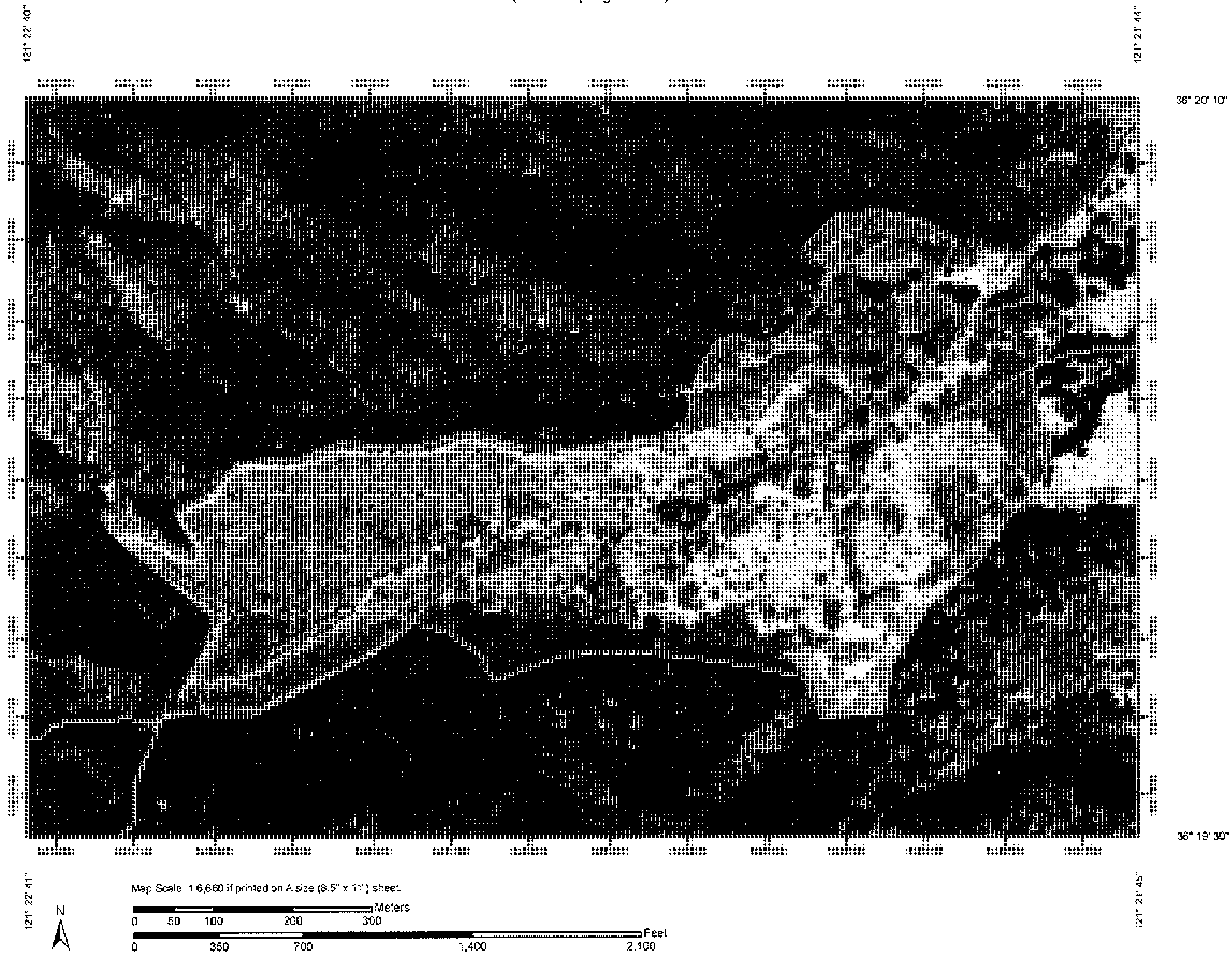
Job Name Paraiso Springs Resort
Subject Proposed Developed Areas

Job No. 434834.03
Sheet No. 4 of 4
Data 5/1/2012
Computed By M. Burns
Checked By

Subbasin	Proposed Developed Areas*
N	384,320 SF
S	332,675 SF
V	445,100 SF
TOTAL	1,162,095 SF

*Proposed developed areas calculated based on the Paraiso Springs Resort Vesting Tenative Map dated Nov 11, 2009.

Attachment 5
Web Soil Survey 2.3 Output for the Project Site


121° 22' 40"
121° 21' 44"
36° 20' 10"
36° 20' 10"
36° 19' 40"
36° 19' 30"
121° 22' 41"
121° 21' 45"
Map Scale: 1:6,660 if printed on A size (8.5" x 11") sheet.
N
0 50 100 200 300 Meters
0 350 700 1,400 2,100 Feet

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Map Units

Soil Ratings

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Political Features

Cities

Water Features

Streams and Canals

Transportation

- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

MAP INFORMATION

Map Scale: 1:6,660 if printed on A size (8.5" × 11") sheet.

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for accurate map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: UTM Zone 10N NAD83

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Monterey County, California
Survey Area Data: Version 9, Apr 14, 2009

Date(s) aerial images were photographed: 6/13/2005; 6/28/2005

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Hydrologic Soil Group— Summary by Map Unit — Monterey County, California (CA053)				
Map unit symbol	**Map unit name**	**Rating**	**Acres in AOI**	**Percent of AOI**
AsC	Arroyo Seco gravelly sandy loam, 5 to 9 percent slopes	B	17.8	19.3%
CnC	Cropley silty clay, 2 to 9 percent slopes	D	25.3	27.5%
Fa	Fluvents, stony	A	11.0	11.9%
Jc	Junipero-Sur complex	B	2.6	2.8%
LmG	Los Osos clay loam, 50 to 75 percent slopes	C	4.1	4.5%
PnD	Placentia sandy loam, 9 to 15 percent slopes	D	8.0	8.7%
PnE	Placentia sandy loam, 15 to 30 percent slopes	D	0.4	0.4%
Xd	Xerorthents, dissected	D	23.0	24.9%
Totals for Area of Interest			**92.2**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rata of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or wall drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. Thesa soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: *None Specified*

Tie-break Rule: Higher

Attachment 6
Composite Curve Number Calculations

CH2MHILL.

Job Name: Paraiso Springs Resort
Subject: Composite CN Calculations

Job No. 434843.03

Date: 5/2/2012
Computed By: M Burns
Checked By:

Subbasin	Hydrologic Soil Group*	Percentage of Area*
N	B	50%
	D	50%
S	B	25%
	C	25%
	D	50%
V	A	25%
	B	25%
	D	50%

* Hydrologic Soil group and percentage for each subbasin were determined using site specific mapping available through the NRCS Web Soil Survey (accessed 5/2/2012).

CH2MHILL.

Job Name Paraiso Springs Resort
Subject Composite CN Calculations

Job No. 434843.03

Date 5/2/2012
Computed By M Burns
Checked By

Pre-Development, All Subbasins

Cover = Forestland — grass or orchards — evergreens or deciduous, Table 8.7.3 (Mays, 2001), Good condition

Hydrologic Soil Group	Curve Number, CN
A	32
B	58
C	72
D	79

Subbasin	Composite CN Calculation
N	$0.50 \times 58 + 0.50 \times 79 = 68.5$ use 69
S	$0.25 \times 58 + 0.25 \times 72 + 0.50 \times 79 = 72$
V	$0.25 \times 32 + 0.25 \times 58 + 0.50 \times 79 = 62$

CH2MHILL.

Job Name: Paraiso Springs Resort	Job No. 434843.03
Subject: Composite CN Calculations	Sheet No. 3 of 4
	Date 5/2/2012
	Computed By M. Burns
	Checked By

Post-Development, Subbasin N

50% impervious — Assumes all developed areas within the Subbasin are impervious

Cover = average of Row houses, town houses, and residential with lot size of 1/8 acre or less (65% impervious) and Residential with average lot size of 1/4 acre (38% impervious) for an average of 51.5% impervious, Table 8.7.3 (Mays, 2001)

Hydrologic Soil Group	Average CN
A	$\frac{1}{2}(77+61) = 69$
B	$\frac{1}{2}(85+75) = 80$
C	$\frac{1}{2}(90+83) = 86.5$ use 87
D	$\frac{1}{2}(92+87) = 89.5$ use 90

Composite CN $= 0.50 \times 80 + 0.50 \times 90 = 85$

Post-Development, Subbasin S

43% impervious — Assumes all developed areas within the Subbasin are impervious

cover = same as used for Subbasin N

Composite CN $= 0.25 \times 80 + 0.25 \times 87 + 0.50 \times 90 = 86.75$
use 87

CH2MHILL.

Job Name Paraiso Springs Resort

Subject Composite CN Calculations

Job No. 434843.03

Sheet No. 4 of 4

Date 5/2/2012

Computed By M. Burns

Checked By

Post-Development, Subbasin V

23% impervious

Assumes all developed areas are impervious

Cover = Residential with an average lot size of 1/2 acre (25% impervious), Table 8.7.3 (Mays, 2001)

Hydrologic Soil Group	CN
A	54
B	70
C	80
D	85

Composite CN = 0.25x 54 + 0.25x 70 + 0.00x 85 = 73.5

use 74

The values of CN for various land uses on these soil types are given in Table 8.7.3. For a watershed made up of several soil types and land uses, a composite CN can be calculated.

Minimum infiltration rates for the various soil groups are:

Group	*Minimum Infiltration Rate (in/hr)*
A	0.30 – 0.45
B	0.15 – 0.30
C	0 – 0.05

Table 8.7.3 Runoff Curve Numbers (Average Watershed Condition, $I_a = 0.2S$)

Land Use Description			Curve Numbers for Hydrologic Soil Group A	B	C	D
Fully developed urban areas[a] (vegetation established)						
Lawns, open spaces, parks, golf courses, cemeteries, etc.						
Good condition; grass cover on 75% or more of the area			39	61	74	80
Fair condition; grass cover on 50% to 75% of the area			49	69	79	84
Poor condition; grass cover on 50% or less of the area			68	79	86	89
Paved parking lots, roofs, driveways, etc.			98	98	98	98
Streets and roads						
Paved with curbs and storm sewers			98	98	98	98
Gravel			76	85	89	91
Dirt			72	82	87	89
Paved with open ditches			83	89	92	93
		Average % impervious[b]				
Commercial and business areas		85	89	92	94	95
Industrial districts		72	81	88	91	93
Row houses, town houses, and residential with lot sizes 1/8 acre or less		65	77	85	90	92
Residential: average lot size						
1/4 acre		38	61	75	83	87
1/3 acre		30	57	72	81	86
1/2 acre		25	54	70	80	85
1 acre		20	51	68	79	84
2 acre		12	46	65	77	82
Developing urban areas[c] (no vegetation established)						
Newly graded area			77	86	91	94
Cover Land Use	Treatment of Practice	Hydrologic Condition[d]				
Cultivated agricultural land						
Fallow	Straight row		77	86	91	94
	Conservation tillage	Poor	76	85	90	93
	Conservation tillage	Good	74	83	88	90
Row crops	Straight row	Poor	72	81	88	91
	Straight row	Good	67	78	85	89
	Conservation tillage	Poor	71	80	87	90
	Conservation tillage	Good	64	75	82	85

Table 8.7.3 Runoff Curve Numbers (*continued*)

Cover		Hydrologic Condition[d]	Curve Numbers for Hydrologic Soil Group			
Land Use	Treatment of Practice		A	B	C	D
	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	Contoured and conservation tillage	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured and terraces	Poor	66	74	80	82
	Contoured and terraces	Good	62	71	78	81
	Contoured and terraces and conservation tillage	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	Straight row	Poor	65	76	84	88
	Straight row	Good	63	75	83	87
	Conservation tillage	Poor	64	75	83	86
	Conservation tillage	Good	60	72	80	84
	Contoured	Poor	63	74	82	85
	Contoured	Good	61	73	81	84
	Contoured and conservation tillage	Poor	62	73	81	84
		Good	60	72	80	83
	Contoured and terraces	Poor	61	72	79	82
	Contoured and terraces	Good	59	70	78	81
	Contoured and terraces and conservation tillage	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded legumes or rotation meadow[e]	Straight row	Poor	66	77	85	89
	Straight row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
	Contoured and terraces	Poor	63	73	80	83
	Contoured and terraces	Good	51	67	76	80
Noncultivated agricultural land Pasture or range	No mechanical treatment	Poor	68	79	86	89
	No mechanical treatment	Fair	49	69	79	84
	No mechanical treatment	Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
Meadow		—	30	58	71	78
Forestland—grass or orchards—evergreen or deciduous		Poor	55	73	82	86
		Fair	44	65	76	82
		Good	32	58	72	79
Brush		Poor	48	67	77	83
		Good	20	48	65	73
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		—	59	74	82	86
Forest–range Herbaceous		Poor		79	86	92
		Fair		71	80	89
		Good		61	74	84

Table 8.7.3 Runoff Curve Numbers (*continued*)

Cover		Hydrologic Condition[d]	Curve Numbers for Hydrologic Soil Group			
Land Use	Treatment of Practice		A	B	C	D
Oak–aspen		Poor		65	74	
		Fair		47	57	
		Good		30	41	
Juniper–grass		Poor		72	83	
		Fair		58	73	
		Good		41	61	
Sage–grass		Poor		67	80	
		Fair		50	63	
		Good		35	48	

[a] For land uses with impervious areas, curve numbers are computed assuming that 100% of runoff from impervious areas is directly connected to the drainage system. Pervious areas (lawn) are considered to be equivalent to lawns in good condition and the impervious areas have a *CN* of 98.

[b] Includes paved streets.

[c] Use for the design of temporary measures during grading and construction. Impervious area percent for urban areas under development vary considerably. The user will determine the percent impervious. Then using the newly graded area *CN* and Figure 8.7.1*a* or *b*, the composite *CN* can be computed for any degree of development.

[d] For conservation tillage poor hydrologic condition, 5% to 20% of the surface is covered with residue (less than 750-lb/acre row crops or 300-lb/acre small grain).

For conservation tillage good hydrologic condition, more than 20% of the surface is covered with residue (greater than 750-lb/acre row crops or 300-lb/acre small grain).

[e] Close-drilled or broadcast.

For noncultivated agricultural land:
Poor hydrologic condition has less than 25% ground cover density.
Fair hydrologic condition has between 25% and 50% ground cover density.
Good hydrologic condition has more than 50% ground cover density.

For forest–range:
Poor hydrologic condition has less than 30% ground cover density.
Fair hydrologic condition has between 30% and 70% ground cover density.
Good hydrologic condition has more than 70% ground cover density.

Source: U.S. Department of Agriculture Soil Conservation Service (1986).

8.7.3 Curve Numbers

Table 8.7.3 gives the curve numbers for average watershed conditions, $I_a = 0.2S$, and antecedent moisture condition II. For watersheds consisting of several subcatchments with different *CN*s, the area-averaged composite *CN* can be computed for the entire watershed. This analysis assumes that the impervious areas are directly connected to the watershed drainage system (Figure 8.7.1*a*). If the percent imperviousness is different from the value listed in Table 8.7.3 or if the impervious areas are not directly connected, then Figures 8.7.1*a* or *b*, respectively can be used. The pervious *CN* used in these figures is equivalent to the open-space *CN* in Table 8.7.3. If the total impervious area is less than 30 percent, Figure 8.7.1*b* is used to obtain a composite *CN*. For natural desert landscaping and newly graded areas, Table 8.7.3 gives only the *CN*s for pervious areas.

Paraiso Springs Resort (PLN 040183) – Stream Setback Plan

PREPARED FOR: John Thompson/Thompson Holdings, LLC

COPY TO: file

PREPARED BY: David Von Rueden P.E. (#26428)/CH2MHILL

DATE: April 20, 2012

PROJECT NUMBER: 434834

Introduction

This Technical Memorandum (TM) is in response to a request from the Monterey County Water Resources Agency (MCWRA), as outlined in their November 24, 2010 letter to Jacqueline Onciano/Planning Department, regarding development setback from the Paraiso Springs watercourse. The subject watercourse is an unnamed intermittent drainage swale/stream that traverses the Project site, from west to east. Please refer to the *Paraiso Springs Resort – Response to Hydrology and Hydraulic Analysis and Erosion Control Measures Review Comments Technical Memorandum*, prepared by CH2MHILL and dated October 28, 2008, for additional information about this drainage feature.

Watercourse Setback Delineation

Monterey County Code Chapter 16.16.050K specifies a 50-foot setback from a watercourse for all proposed development. Please see the attached Site Plan (4 pages total) for an annotated map showing approximate watercourse top-of-bank locations, and the 50-foot setback line on either side of the watercourse. Please note that the top-of-bank has been delineated using aerial topography. The watercourse is not clearly defined along its entire length throughout the Project site, because at several locations, it is currently confined to culverts. This analysis only focused on portions of the development where a defined channel exists. All of the existing culverts will be removed from the watercourse as part of the Project.

As shown on the attached Site Plan, the proposed development would encroach into the 50-foot setback zone at several locations. Therefore, we have analyzed the significant encroachments relative to the two provisions outlined in the previously-noted County Code. The locations studied are labeled on the Site Plan as Sections A-A, B-B and C-C. Section A-A is representative of the setback encroachment from time-share-condominiums near the downstream end of the watercourse. Section B-B is located near the center of the development, where Hotel units encroach into the setback zone. Section C-C is located further upstream and indicative of the setback encroachment from the proposed spa and fitness facilities.

Watercourse Capacity

Provision 1 of County Code Chapter 16.16.050K requires that development within a setback zone not significantly reduce capacity of an existing watercourse, nor otherwise adversely affect other properties.

The capacity of the existing watercourse was initially evaluated by CH2MHILL and summarized in the TM entitled *Paraiso Springs Resort: Existing Hydrologic and Hydraulic Site Conditions*, dated July 15, 2005. This document described the flow capacity of the existing watercourse as approximately 4,000 cfs. This capacity exceeded the approximately 400 cfs of runoff from a 100-yr storm event. Subsequently, CH2MHILL re-evaluated the watershed using a more accurate HEC-HMS model and documented the post-Project 100-yr runoff rate as 316 cfs, in their 2008 TM. Based on the previous analysis and the information presented in the following section, the existing watercourse should have adequate capacity to covey the anticipated 100-yr post-project flow rate. The proposed development will not constrict or significantly reduce the existing watercourse capacity. No adverse impacts on other properties are anticipated by the proposed development.

Erosion Protection

Provision 2 of County Code Chapter 16.16.050K requires that the new development be safe from flow related erosion and not cause erosion hazards.

To address this issue, we propose to use Rock Slope Protection (RSP) in the watercourse, at all locations where building or critical roadway construction would encroach into the 50-foot setback zone. Please refer to the attached typical watercourse sections A-A, B-B and C-C for conceptual RSP installation details at critical Project locations. The conceptual RSP design is based upon the California Bank and Shore Rock Slope Protection Design Manual, published by Caltrans. Pertinent pages from this Manual are attached. The approximate stream depths and velocities were calculated manually, using King's Handbook of Hydraulics. These preliminary calculations are also attached. Table 1 summarizes the key design parameters of the RSP design.

Table 1

	Flow 1 Data (Q cfs)	Channel Roughness (*n*)	Average Channel Slope (s)	Channel Bottom Width (b-ft)	Channel Sideslope (z)	100-Yr Water Depth (D-ft)	Stream Velocity[1] (fps)	RSP Class[2]	RSP Thickness[3] (ft)
Section A-A	316	0.03	0.100	10	4:1	1.4	9.7	Light	3
Section B-B	316	0.03	0.053	7	2:1	2.0	9.7	Light	3
Section C-C	316	0.03	0.071	20	4:1	1.2	7.3	Light	3

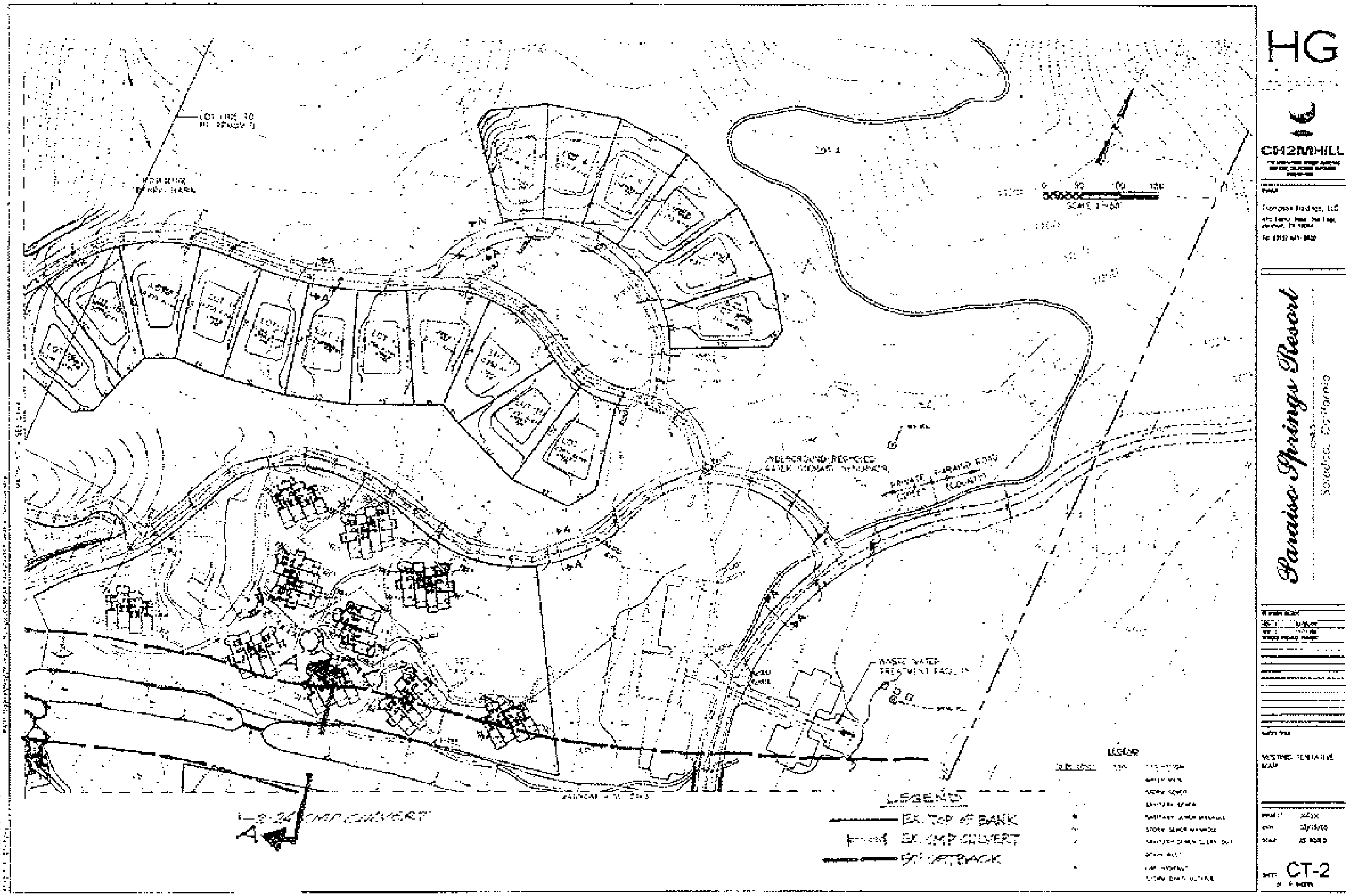
[1]Velocity shown is 67% of calculated average channel velocity, applicable for parallel flow per CALTRANS Manual.

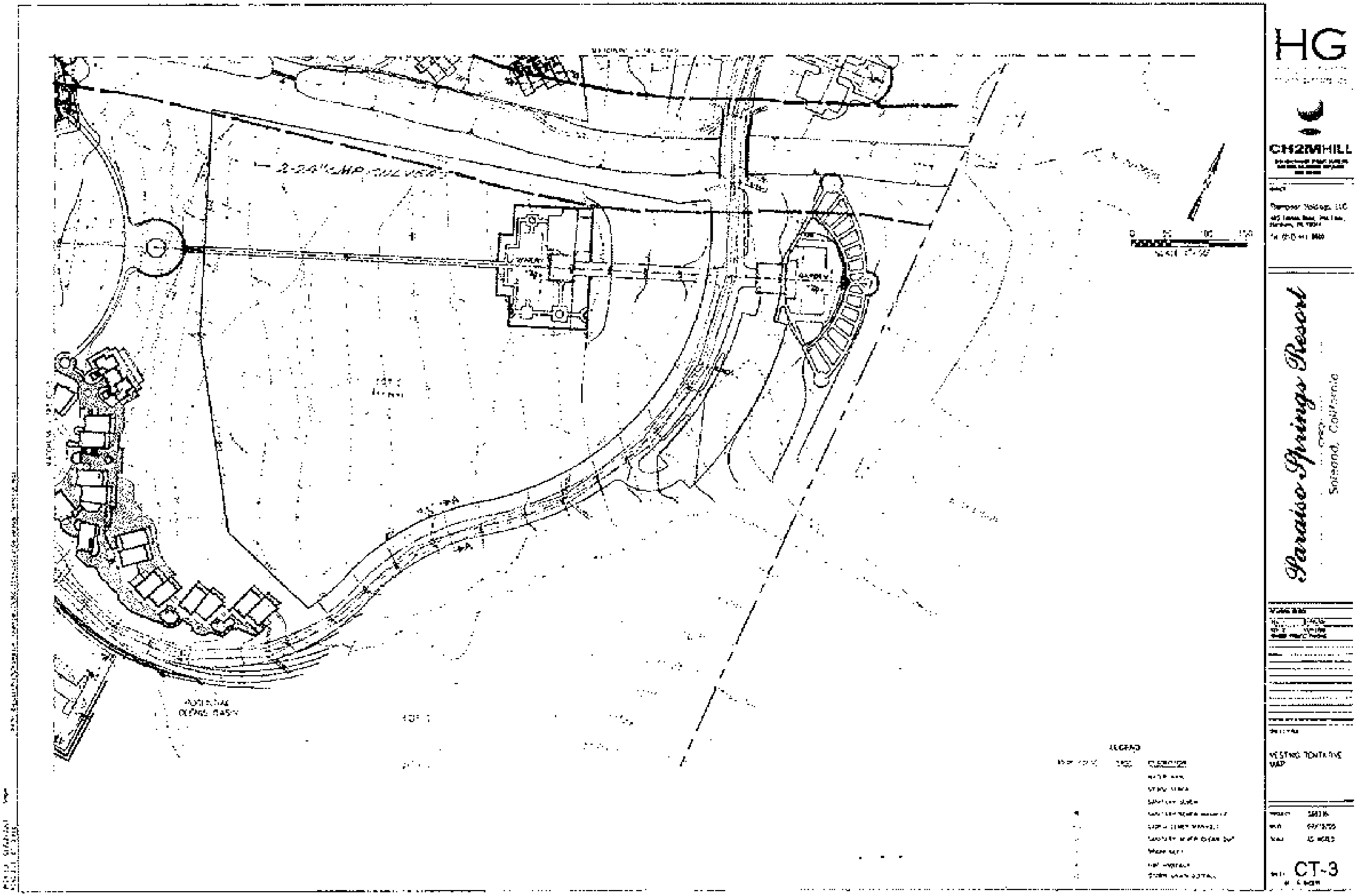
[2] RSP class is based on CALTRANS standard specifications, Section 72.

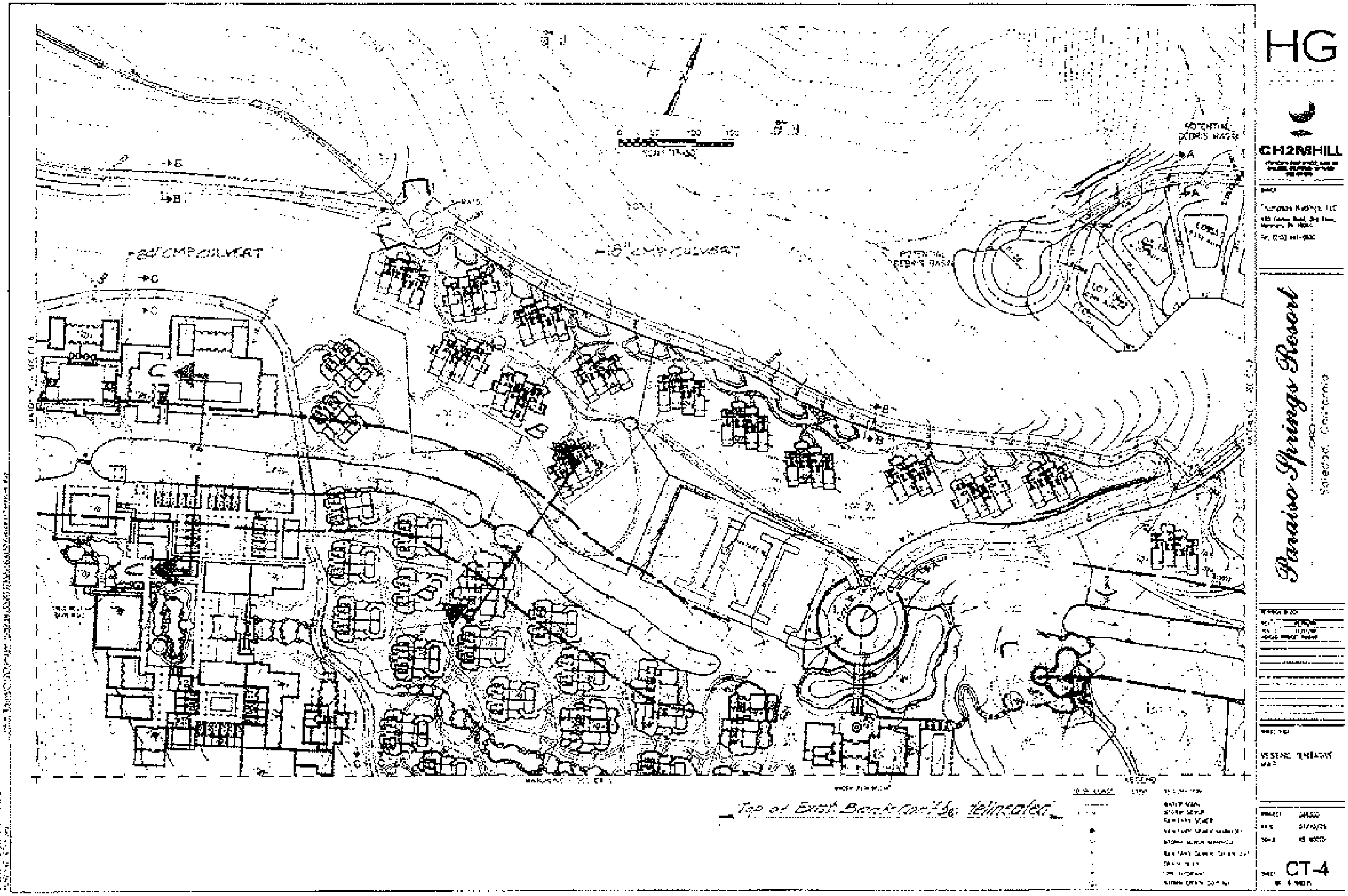
[3] RSP section includes placement of a geotextile fabric, between soil surface and rock.

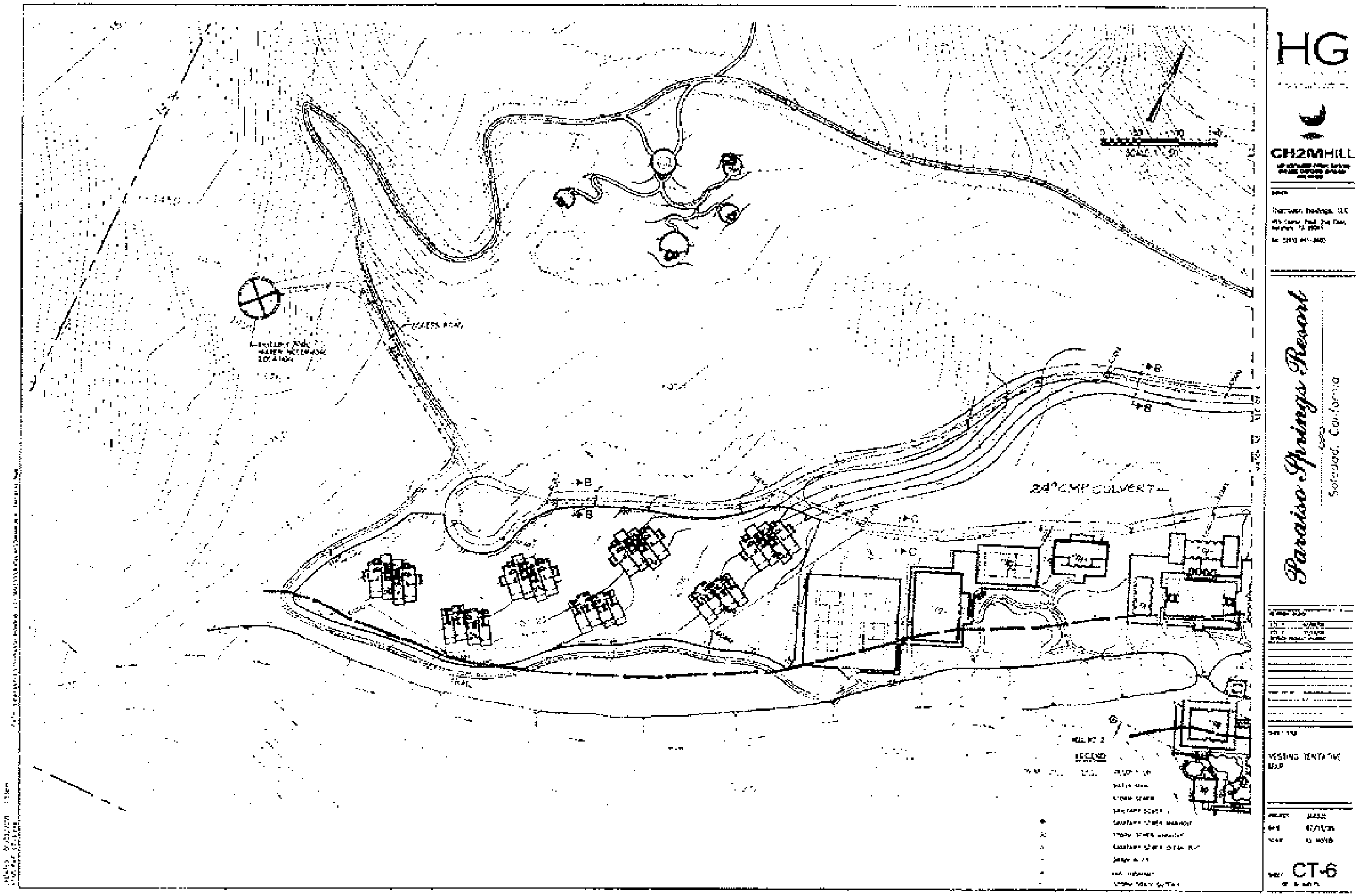
Summary

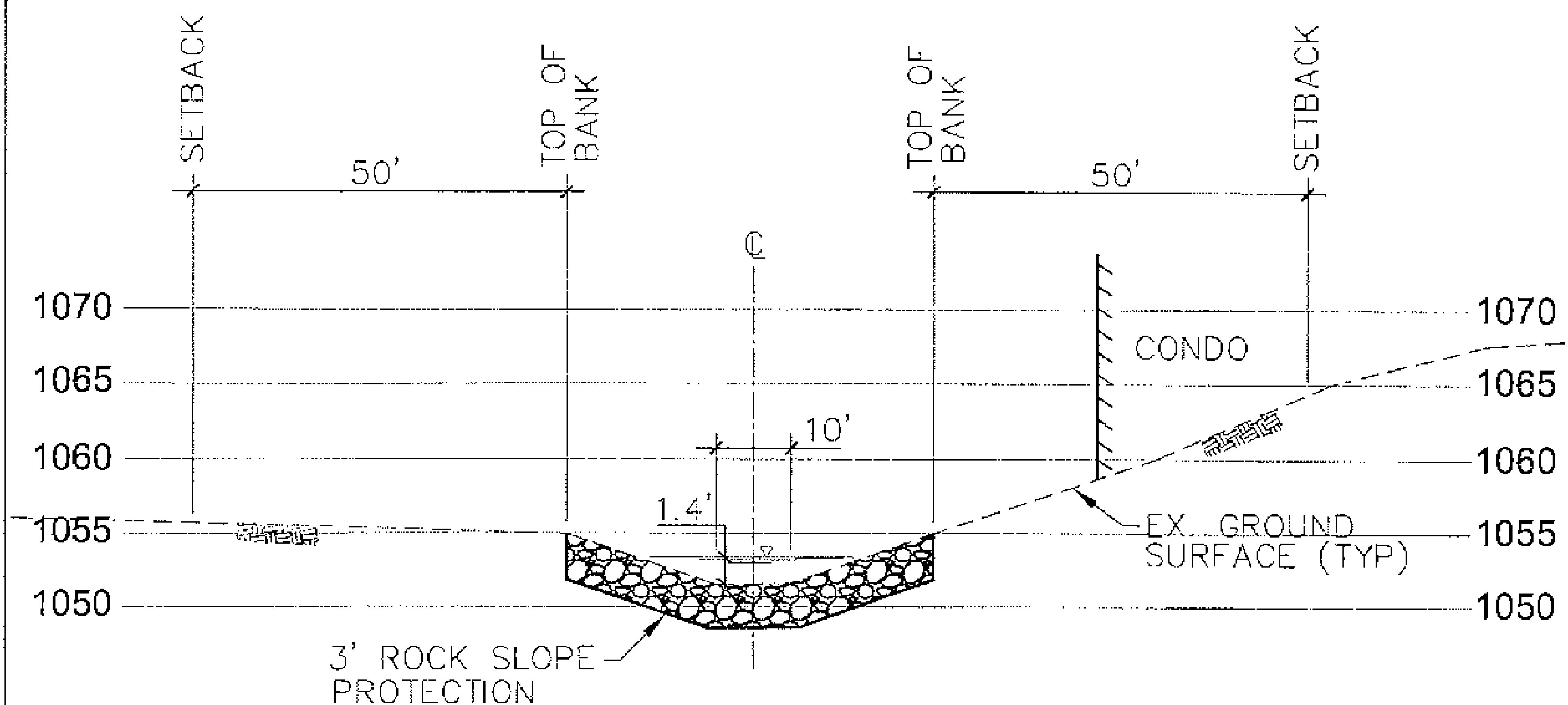
It is anticipated that during Project design, the Site Plan will be refined to reduce setback zone encroachments. At locations where setback encroachments cannot be avoided, erosion control measures as described herein will be constructed within the existing watercourse for erosion protection and to preserve 100-yr flow capacity.


HG
CH2MHILL
Paradise Springs Resort
LEGEND
EX. TOP OF BANK
EX. CMP CULVERT
50' SETBACK
WASTE WATER TREATMENT FACILITY
PRIVATE
COUNTY
CT-2


HG
CH2MHILL
Paraiso Springs Resort
Soledad, California
VESTING TENTATIVE MAP
CT-3
LEGEND
2-24" CMP CULVERT
0 50 100 150


HG
CH2MHILL
Paraiso Springs Resort
Soledad, California
CT-4
24" CMP CULVERT
18" CMP CULVERT
POTENTIAL DEBRIS BASIN
Top of Exist. Bank (on'd) delineated
LEGEND


HG
CH2MHILL
Paraiso Springs Resort
Soledad, California
24"CMP CULVERT
ACCESS ROAD
LEGEND
VESTING TENTATIVE MAP
CT-6

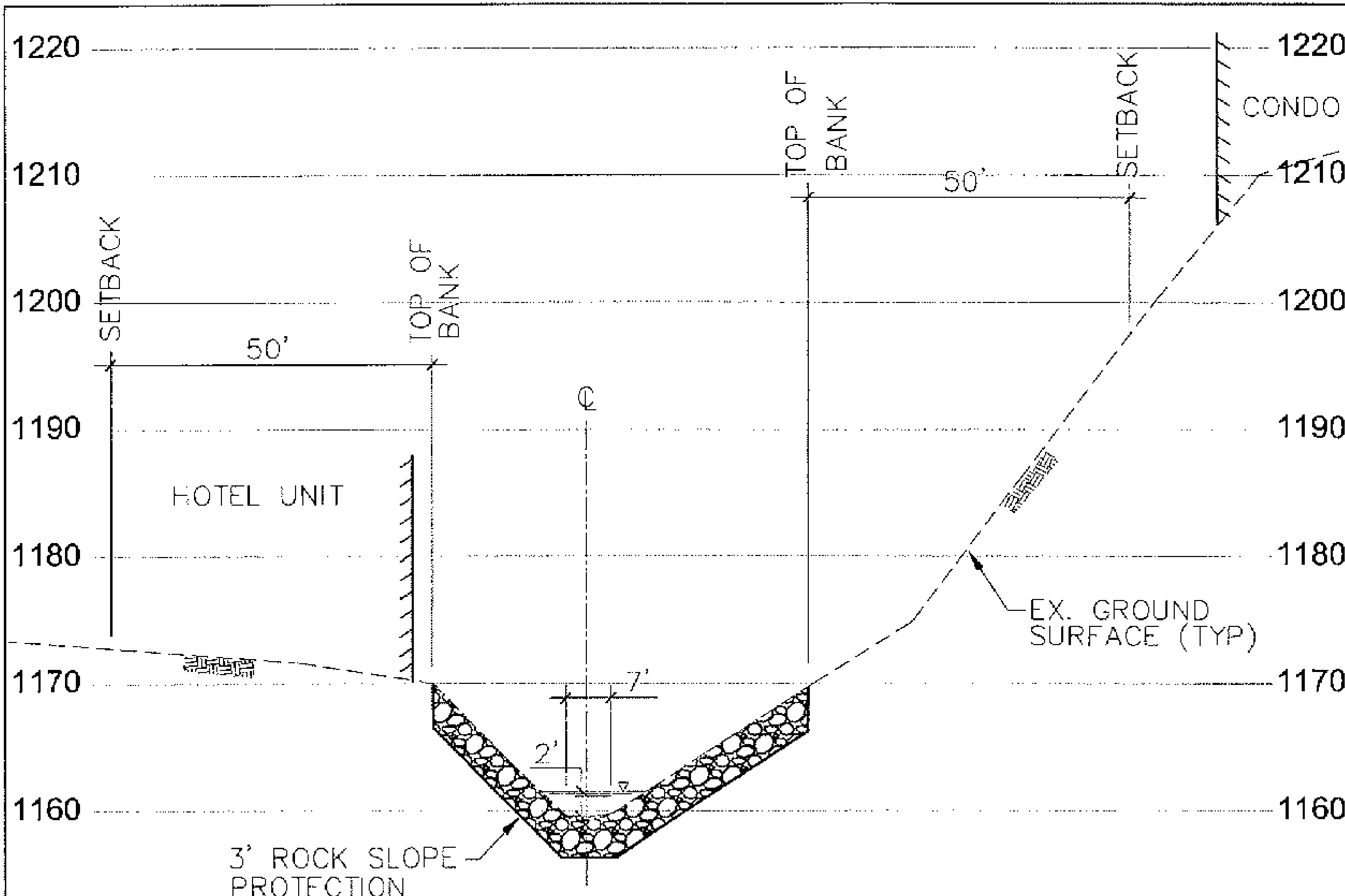


SECTION A-A

SCALE HORZ. = 1"=20'
VERT. = 1"=10'

CH2MHILL

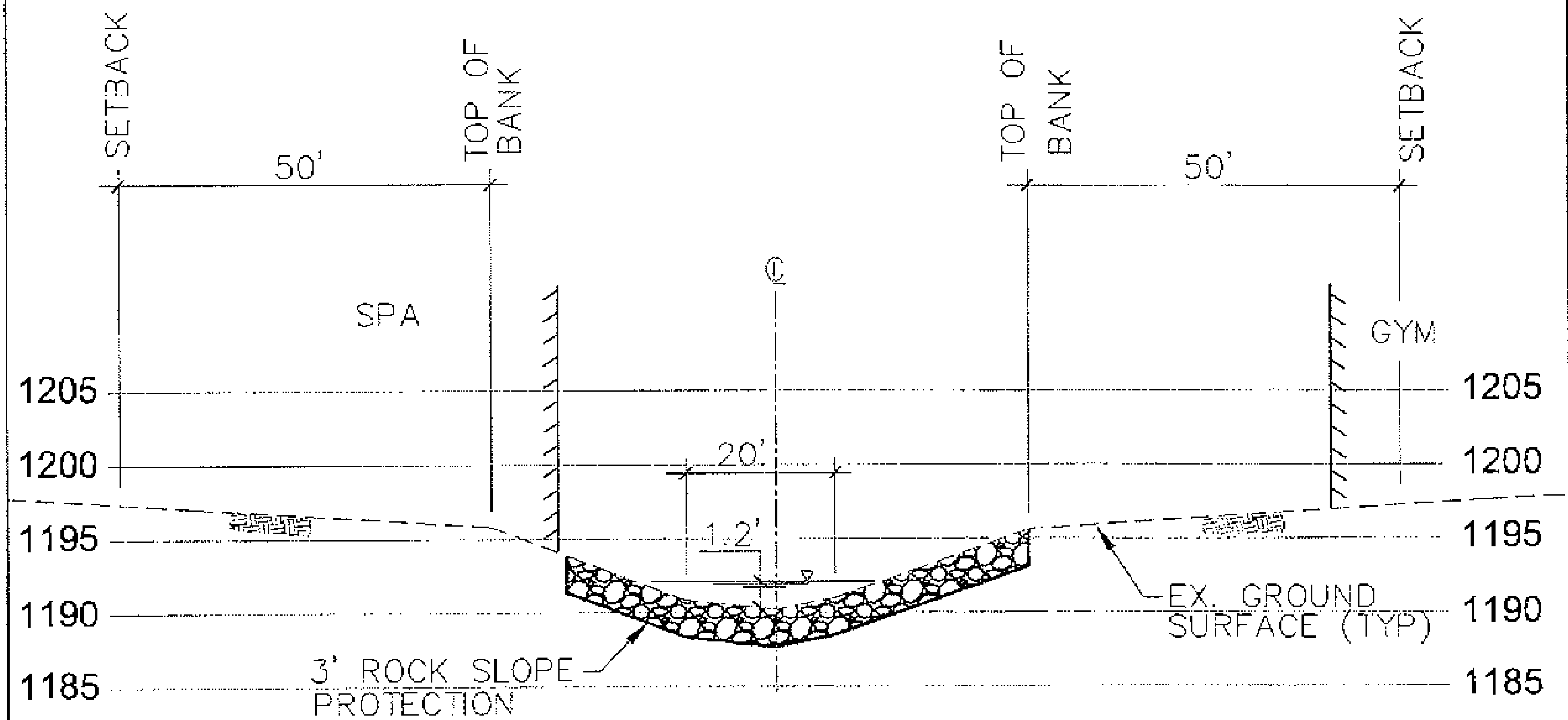
1737 NORTH FIRST STREET, SUITE 300
SAN JOSE, CALIFORNIA 95112-4524
(408) 436-4909



SECTION B-B

SCALE HORZ. = 1"=20'
VERT. = 1"=10'

CH2MHILL
1737 NORTH FIRST STREET, SUITE 300
SAN JOSE, CALIFORNIA 95112-4524
(408) 436-4909



SECTION C-C

SCALE HORZ. = 1"=20'
VERT. = 1"=10'

CH2MHILL
1737 NORTH FIRST STREET, SUITE 300
SAN JOSE, CALIFORNIA 95112-4524
(408) 436-4909

- Determine RSP Size & Thickness
  - Use King's Handbook to determine approx. stream depth.
  - Use Caltrans's Bank & Shore Protection Manual to size RSP.
  - $Q_{100} = 316$ cfs

① Section A-A

$n = 0.03$, $s = 0.100$ (from topo)
$b = 10'$, $z = 4.1$

$$Q = \frac{K'}{n} b^{8/3} s^{1/2}$$

$$K' = \frac{Q(n)}{(b^{8/3})(s^{1/2})} = \frac{316(0.03)}{(10)^{8/3}(0.100)^{1/2}} = 0.06459$$

$D/b = 0.14$ (from Handbook Table 7-11)

$D = 0.14(10) = 1.4'$

$A = 22 \text{ ft}^2$

$V = Q/A = 316/22 = 14.4$ fps

Size RSP: $SG = 2.65$, $a = 14°$

$$W = \frac{0.00002\, V^6\, SG}{(SG-1)^3 \sin^3(\rho - a)}$$ (from Caltrans' Manual)

$$W = \frac{0.00002\,(14.4 \times 0.67)^6 (2.65)}{(2.65-1)^3 (\sin^3 70° - 14°)} = 16.7^{\#}$$

Use Class Light, $T = 2.5'$ (say 3' w/ geotextile

(2) Section C-C $\quad n = 0.03,\ s = 0.071,\ b = 20'$, $z = 4:1$

$$K' = \frac{(316)(0.03)}{(20)^{8/3}(0.071)^{1/2}} = 0.01207$$

$D/b = 0.06$ (from Handbook Table 7-11)

$D = 1.2'$, $A = 29^{\#}$

$V = 316/29 = 10.9$ fps

Size RSP: $\quad SG = 2.65,\ \alpha = 14°$

$$W = \frac{0.00002\,(10.9 \times 0.67)^6\,(2.65)}{(2.65-1)^3 \sin^3(70-14)}$$

$W = 3.2^{\#}$

Use same RSP as for Section A-A

(3) Section B-B $\quad n = 0.03,\ s = 0.053,\ b = 7'$, $z = 2:1$

$$K' = \frac{(316)(.03)}{(7)^{8/3}(0.053)^{1/2}} = 0.2297$$

$D/b = .29$ (from Handbook) Table 7-11

$D = 2.0$, $A = 22^{\#}$

$V = 316/22 = 14.4$ fps

Size RSP: $\quad SG = 2.65,\ \alpha = 26.6°$

$$W = \frac{(0.00002)(14.4 \times .67)^6(2.65)}{(2.65-1)^3 \sin^3(90-26.6)} = 29.3^{\#}$$

Use same RSP as for Section A-A

Table 7-11. Values of K' in Formula $Q = \frac{K'}{n} b^{8/3} s^{1/2}$ for Trapezoidal Channels

D = depth of water b = bottom width of channel

	$\frac{D}{b}$	Side slopes of channel, ratio of horizontal to vertical									
		Vertical	¼-1	½-1	¾-1	1-1	1½-1	2-1	2½-1	3-1	4-1
	.01	.00068	.00068	.00069	.00069	.00069	.00069	.00069	.00069	.00070	.00070
	.02	.00213	.00215	.00216	.00217	.00218	.00220	.00221	.00222	.00223	.00225
	.03	.00414	.00419	.00423	.00426	.00428	.00433	.00436	.00439	.00443	.00449
	.04	.00660	.00670	.00679	.00685	.00691	.00700	.00708	.00716	.00723	.00736
	.05	.00946	.00964	.00979	.00991	.01002	.01019	.01033	.01047	.01060	.01084
C-C	.06	.0127	.0130	.0132	.0134	.0136	.0138	.0141	.0143	.0145	.0150
	.07	.0162	.0166	.0170	.0173	.0175	.0180	.0183	.0187	.0190	.0197
	.08	.0200	.0205	.0211	.0215	.0219	.0225	.0231	.0236	.0240	.0250
	.09	.0241	.0249	.0256	.0262	.0267	.0275	.0282	.0289	.0296	.0310
	.10	.0284	.0294	.0304	.0311	.0318	.0329	.0339	.0349	.0358	.0375
	.11	.0329	.0343	.0354	.0364	.0373	.0387	.0400	.0413	.0424	.0445
	.12	.0376	.0393	.0408	.0420	.0431	.0450	.0466	.0482	.0497	.0527
A-A	.13	.0425	.0446	.0464	.0480	.0493	.0516	.0537	.0556	.0575	.0613
	.14	.0476	.0502	.0524	.0542	.0559	.0587	.0612	.0636	.0659	.0706
	.15	.0528	.0560	.0586	.0608	.0627	.0662	.0692	.0721	.0749	.0805
	.16	.0582	.0619	.0650	.0676	.0700	.0740	.0777	.0811	.0845	.0912
	.17	.0638	.0680	.0716	.0748	.0775	.0823	.0866	.0907	.0947	.1026
	.18	.0695	.0744	.0786	.0822	.0854	.0910	.0960	.1008	.1055	.1148
	.19	.0753	.0809	.0857	.0899	.0936	.1001	.1059	.1115	.1169	.1277
	.20	.0812	.0876	.0931	.0979	.1021	.1096	.1163	.1227	.1290	.1414
	.21	.0873	.0945	.101	.106	.111	.120	.127	.135	.142	.156
	.22	.0934	.1015	.109	.115	.120	.130	.139	.147	.155	.171
	.23	.0997	.1087	.117	.124	.130	.141	.150	.160	.169	.187
	.24	.1061	.1161	.125	.133	.140	.152	.163	.173	.184	.204
	.25	.1125	.1236	.133	.142	.150	.163	.176	.188	.199	.222
	.26	.119	.131	.142	.152	.160	.175	.189	.202	.215	.241
	.27	.126	.139	.151	.162	.171	.188	.203	.218	.232	.260
	.28	.132	.147	.160	.172	.182	.201	.217	.234	.249	.281
B-B	.29	.139	.155	.170	.182	.194	.214	.232	.250	.268	.302
	.30	.146	.163	.179	.193	.205	.228	.246	.267	.287	.324
	.31	.153	.172	.189	.204	.218	.242	.264	.285	.306	.347
	.32	.160	.180	.199	.215	.230	.256	.281	.304	.327	.371
	.33	.167	.189	.209	.227	.243	.271	.298	.323	.348	.396
	.34	.174	.198	.219	.238	.256	.287	.316	.343	.370	.423
	.35	.181	.207	.230	.251	.269	.303	.334	.363	.392	.450
	.36	.189	.216	.241	.263	.283	.319	.353	.385	.416	.478
	.37	.196	.225	.252	.275	.297	.336	.372	.406	.440	.507
	.38	.203	.234	.263	.288	.312	.353	.392	.429	.465	.537
	.39	.211	.244	.274	.301	.326	.371	.413	.452	.491	.568
	.40	.218	.253	.286	.315	.341	.389	.434	.476	.518	.600
	.41	.226	.263	.297	.328	.357	.408	.456	.501	.546	.633
	.42	.233	.273	.309	.342	.373	.427	.478	.526	.574	.668
	.43	.241	.283	.321	.357	.389	.447	.501	.553	.603	.703
	.44	.248	.293	.334	.371	.405	.467	.525	.580	.633	.740
	.45	.256	.303	.346	.386	.422	.488	.549	.607	.664	.777

Table 7-11. Values of K' in Formula $Q = \frac{K'}{n} b^{8/3} s^{1/2}$ for Trapezoidal Channels (*Continued*)

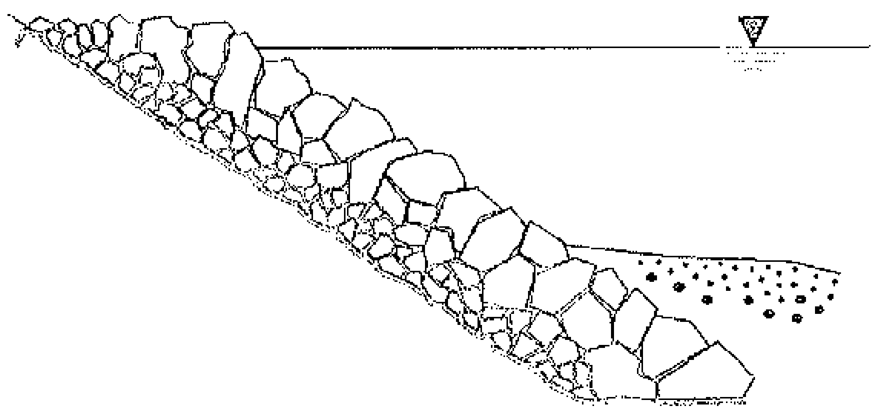
D = depth of water b = bottom width of channel

$\frac{D}{b}$	Side slopes of channel, ratio of horizontal to vertical									
	Vertical	¼-1	½-1	¾-1	1-1	1½-1	2-1	2½-1	3-1	4-1
.46	.264	.313	.359	.401	.439	.509	.574	.636	.696	.816
.47	.271	.323	.372	.416	.457	.531	.599	.665	.729	.856
.48	.279	.334	.385	.432	.474	.553	.625	.695	.762	.897
.49	.287	.344	.398	.447	.493	.575	.652	.725	.797	.939
.50	.295	.355	.412	.463	.511	.598	.679	.757	.833	.983
.51	.303	.366	.425	.480	.530	.622	.707	.789	.869	1.03
.52	.311	.377	.439	.496	.549	.646	.736	.822	.907	1.07
.53	.319	.388	.453	.513	.569	.671	.765	.856	.945	1.12
.54	.327	.399	.467	.531	.589	.696	.795	.891	.984	1.17
.55	.335	.410	.482	.548	.609	.722	.826	.926	1.025	1.22
.56	.343	.422	.497	.566	.630	.748	.857	.968	1.07	1.27
.57	.351	.433	.511	.584	.651	.775	.889	1.000	1.11	1.33
.58	.359	.445	.526	.602	.673	.802	.922	1.035	1.15	1.37
.59	.367	.456	.542	.621	.694	.830	.956	1.077	1.20	1.42
.60	.375	.468	.557	.640	.717	.858	.990	1.117	1.24	1.46
.61	.383	.480	.573	.659	.739	.887	1.02	1.16	1.29	1.54
.62	.391	.492	.588	.678	.762	.916	1.06	1.20	1.33	1.60
.63	.399	.504	.604	.698	.786	.946	1.10	1.24	1.38	1.66
.64	.408	.516	.620	.718	.809	.977	1.13	1.28	1.43	1.72
.65	.416	.529	.637	.738	.833	1.008	1.17	1.33	1.48	1.79
.66	.424	.541	.653	.759	.857	1.04	1.21	1.37	1.53	1.85
.67	.433	.553	.670	.780	.882	1.07	1.25	1.42	1.59	1.91
.68	.441	.566	.687	.801	.907	1.10	1.29	1.47	1.64	1.98
.69	.449	.579	.704	.822	.933	1.14	1.33	1.51	1.69	2.05
.70	.457	.592	.722	.844	.959	1.17	1.37	1.56	1.75	2.12
.71	.466	.604	.739	.866	.985	1.21	1.41	1.61	1.81	2.19
.72	.474	.617	.757	.889	1.012	1.24	1.46	1.66	1.86	2.26
.73	.483	.631	.775	.911	1.039	1.28	1.50	1.71	1.92	2.34
.74	.491	.644	.793	.934	1.067	1.31	1.54	1.77	1.98	2.41
.75	.499	.657	.811	.957	1.095	1.35	1.59	1.82	2.05	2.49
.76	.508	.670	.830	.981	1.12	1.39	1.63	1.87	2.11	2.57
.77	.516	.684	.849	1.005	1.15	1.43	1.68	1.93	2.17	2.65
.78	.525	.698	.868	1.029	1.18	1.46	1.73	1.99	2.24	2.73
.79	.533	.711	.887	1.054	1.21	1.50	1.78	2.04	2.30	2.81
.80	.542	.725	.906	1.078	1.24	1.54	1.83	2.10	2.37	2.90
.81	.550	.739	.925	1.10	1.27	1.58	1.88	2.16	2.44	2.98
.82	.559	.753	.945	1.13	1.30	1.62	1.93	2.22	2.51	3.07
.83	.567	.767	.965	1.15	1.33	1.67	1.98	2.28	2.58	3.16
.84	.576	.781	.985	1.18	1.36	1.71	2.03	2.34	2.65	3.25
.85	.585	.796	1.006	1.21	1.40	1.75	2.09	2.41	2.72	3.35
.86	.593	.810	1.03	1.23	1.43	1.79	2.14	2.47	2.80	3.44
.87	.602	.825	1.05	1.26	1.46	1.84	2.19	2.54	2.87	3.54
.88	.610	.839	1.07	1.29	1.49	1.88	2.25	2.60	2.95	3.63
.89	.619	.854	1.09	1.31	1.53	1.93	2.31	2.67	3.03	3.73
.90	.628	.869	1.11	1.34	1.56	1.98	2.36	2.74	3.11	3.83

State of California
Department of Transportation
Engineering Service Center
Office of Structural Foundations
Transportation Laboratory

CALIFORNIA BANK AND SHORE ROCK SLOPE PROTECTION DESIGN

Practitioner's Guide and Field Evaluations of Riprap Methods



Final Report No. FHWA-CA-TL-95-10
Caltrans Study No. F90TL03

Third Edition - Internet
October 2000

Prepared in Cooperation with the US Department of Transportation
Federal Highway Administration

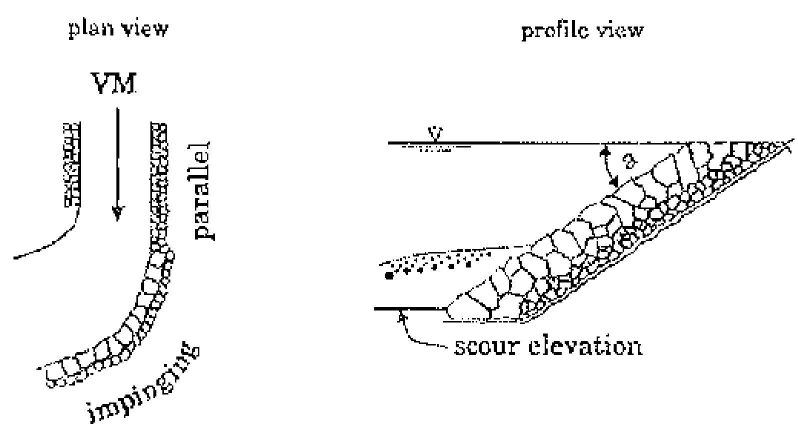
5-1-C. Determine Minimum Stone Weight. Solve Equation 1 for W in US customary units. To get values in System International (SI), metric units, first divide the weight of minimum stable rock, W in pounds by 2.2 to get W in kilograms, then divide by 1000 to get W in tonnes. Use W later in section 5-1-D.
See Figure 5-1 for key variables in Equation 1.

Equation 1.
$$W = \frac{0.00002}{(SG - 1)^3} \quad \frac{V^6}{SIN^3 (r - a)} \quad SG$$

W = theoretical **minimum rock mass (size or weight)** which resists forces of flowing water and remains stable on slope of stream or river bank, POUNDS.
V = velocity to which bank is exposed, FEET PER SECOND.
for PARALLEL flow multiply average channel velocity VM by 0.67 (2/3)
for IMPINGING flow multiply average channel velocity VM by 1.33 (4/3)
SG = specific gravity of the rock.
r = 70 DEGREES (for randomly placed rubble, a constant).
a = outside slope face angle with horizontal, DEGREES.

In profile, the lower elevation limit of riverbank RSP is based on expected scour (determined by experience, measurements, or scour equations). The upper elevation limit is based on design high water, although it may be set higher.

Figure 5-1. Key Variables in Equation 1



STANDARD Rock SIZE or Rock MASS or Rock WEIGHT		GRADING OF ROCK SLOPE PROTECTION **PERCENTAGE LARGER THAN**											
		RSP-Classes [A]											
		Method A Placement					Method B Placement						
		RSP-Classes other than Backing									Backing No.		
US unit		8 ton	4 ton	2 ton	1 ton	1/2 ton	1 ton	1/2 ton	1/4 ton	Light	1 [B]	2	3
	SI unit	8 T	4 T	2 T	1 T	1/2 T	1 T	1/2 T	1/4 T	Light	1 [B]	2	3
16 ton	14.5 tonne	0-5											
8 ton	7.25 tonne	50-100	0-5										
4 ton	3.6 tonne	95-100	50-100	0-5									
2 ton	1.8 tonne		95-100	50-100	0-5		0-5						
1 ton	900 kg			95-100	50-100	0-5	50-100	0-5					
1/2 ton	450 kg				95-100	50-100	----	50-100	0-5				
1/4 ton	220 kg					95-100	95-100	-----	50-100	0-5			
200 lb	90 kg							95-100	----	50-100	0-5		
75 lb	34 kg								95-100	-----	50-100	0-5	
25 lb	11 kg									95-100	90-100	25-75	0-5
5 lb	2.2 kg											90-100	25-75
1 lb	0.4 kg												90-100

[A] US customary names (units) of RSP-Classes listed above SI names, example US is "2 ton" metric is "2 T".
[B] "Facing" has same gradation as "Backing No. 1". To conserve space "Facing" is not shown .

Example for determining RSP-Class of outside layer. By using Equation 1, if the calculated W=135 kg (minimum stable rock size):
1. Enter table at left and select closest value of STANDARD Rock SIZE which is greater than calculated W, in this case 220 kg
2. Trace to right and locate "50-100" entry 3. Trace upward and read column heading "1/4 T", then 1/4 T is first trial RSP-Class.

Table 5-1. Guide for Determining RSP-Class of Outside Layer

Table 5-3. Minimum Layer Thickness SI metric (US customary)		
RSP-Class Layer	Method of Placement	Minimum Thickness
8 T (8 ton)	A	2.60 meters (8.5 feet)
4 T (4 ton)	A	2.07 meters (6.8 feet)
2 T (2 ton)	A	1.65 meters (5.4 feet)
1 T (1 ton)	A	1.31 meters (4.3 feet)
1/2 T (1/2 ton)	A	1.04 meters (3.4 feet)
1 T (1 ton)	B	1.65 meters (5.4 feet)
1/2 T (1/2 ton)	B	1.31 meters (4.3 feet)
1/4 T (1/4 ton)	B	1.00 meters (3.3 feet)
Light	B	760 millimeters (2.5 feet)
Facing	B	550 millimeters (1.8 feet)
Backing No. 1	B	550 millimeters (1.8 feet)
Backing No. 2	B	380 millimeters (1.25 feet)
Backing No. 3	B	230 millimeters (0.75 feet)

For total thickness, add each layer thickness. Use zero thickness for the RSP-fabric. Before adopting values in Table 5-3, consult with a materials engineer about rock sources, quality, shapes, and specific gravity. Calculate new thickness values if the shape factor is not spherical and specific gravity is not reasonably close to 2.65. "Minimum Thickness" values were calculated by starting with US customary units, hard-converting to a value in feet, then soft-converting to SI metric values.

5-1-G. Review Hydraulic Calculations at Site With RSP and Possibility of Vegetation. This step of the layered design process is required to help assure future success of the revetment under changed channel dimensions, roughness coefficients, and other permit/agreement requirements. Examples are: filling voids among RSP with soil and/or covering RSP with soil then planting local species, and/or enhancing fish habitat by placing large-sized rock along the toe. Discuss site hydraulics with people of permit agencies and feasible revegetation efforts. Historically, sites with no prior vegetation are usually not revegetated, especially when subjected to scouring velocities or high wave attack.

CH2M HILL Engineers, Inc.
1737 North First Street
Suite 300
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95112-4524
Tel 408.436.4936
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February 14, 2013

434834

John Thompson
Thompson Holdings, LLC
P.O. Box 2015
Horsham, PA 19044

Subject: Paraiso Springs Resort – PLN040183
Stream Channel Modification
Response to Comments from Monterey County

Dear John:

We have reviewed the February 2, 2013 email from John Ford of the Monterey County Planning Department, regarding the subject Project and offer the following responses relative to information requested. The questions from the email have been included below in italics, for ease of reference. Our response immediately follows each question as listed below.

1. *Q. An engineered plan showing the existing contours along the stream channel, including the existing grades and the proposed changes to the channel including removal of the culvert, recontouring of the channel and improvements for the crossing locations*

   A. Existing elevation contours along the stream channel are shown on the Project Site Plan included on the Tentative Map, previously submitted to the County on 5/18/12. Proposed changes to the stream channel include the following:

- Removal of existing small diameter metal culverts at four locations (see attached Site Plan). As part of this work, the stream bed and banks would be reconstructed to match the existing channel section adjacent to these work areas and to a stable side slope per geotechnical recommendations. Disturbed channel areas would be revegetated with native grasses via hydroseeding.

- Portions of the Project will encroach into the 50-ft top of bank setback zone and rock slope protection will be installed in the channel to prevent erosion, per County Code 16.16.050K. The general location and installation details are included in the Stream Setback Plan Technical Memorandum, dated 4/20/12, previously submitted to the County.
- New stream channel crossings for new roadways are proposed at two locations. These locations and roadway widths are shown on the Tentative Map. The crossings are planned to be clear-span concrete slab bridges on pile foundations. The bridge spans would be approximately 50 ft long. Rock slope protection would be installed on the channel banks beneath and approximately 25 ft upstream and downstream of the bridge abutments for erosion and scour protection. Disturbed channel areas would be revegetated with native grasses via hydroseeding. Another similar type bridge will cross the proposed pond immediately north of the main Hotel building (see attached Site Plan).

2. *Q. The potential impacts to wetlands need to be identified. This would include impacts in the existing stream channel where modifications are proposed and areas of riparian vegetation along the eastern stretch of the channel, particularly where the stream crossing is shown. WRA did the wetland delineation, but did not evaluate the impacts associated with any activities around the stream. They indicate there will be no impacts to the stream channel. Right now there is no assessment of the potential impact on the channel for either minimal improvements (channelization and stream crossings) or for the re-circulating stream*.

   A. Wetland impacts will be addressed by others.

3. *Q. If the recirculating stream is to be analyzed, we need to know where the water will come from, the volume of water used, and the source of the water. Please define the extent of the recirculating stream, and what improvements are needed to the channel to accommodate the stream. How will the recirculating stream affect water supply? The biologist should assess what impact the recirculating stream will have on the vegetation along the stream channel there is a portion of the stream that goes through oak woodland.*

   A. A pond is proposed as a landscape feature located between the Hotel and Hotel parking lot, as shown on the Tentative Map. The pond would have a surface area of approximately 15,000 to 20,000 sf and a depth of 5-10 ft. It would be constructed in an area where the stream currently is contained in an existing culvert and would be connected to the existing stream channel at the westerly and easterly ends of the pond. The stream connections are anticipated to be graded transitions and armored with landscape-type amenities, such as boulders. The water source for the pond would be natural springs water piped from the spa overflow. As the springs flow constantly, the pond would fill and then spill excess water down the existing stream channel, as is the current condition. Because springs water would be used to fill the pond, no effect on the Project water supply is anticipated. A pond liner is anticipated to control seepage and retain water volume. The pond would likely include an aeration system to maintain water quality.

Thank you.

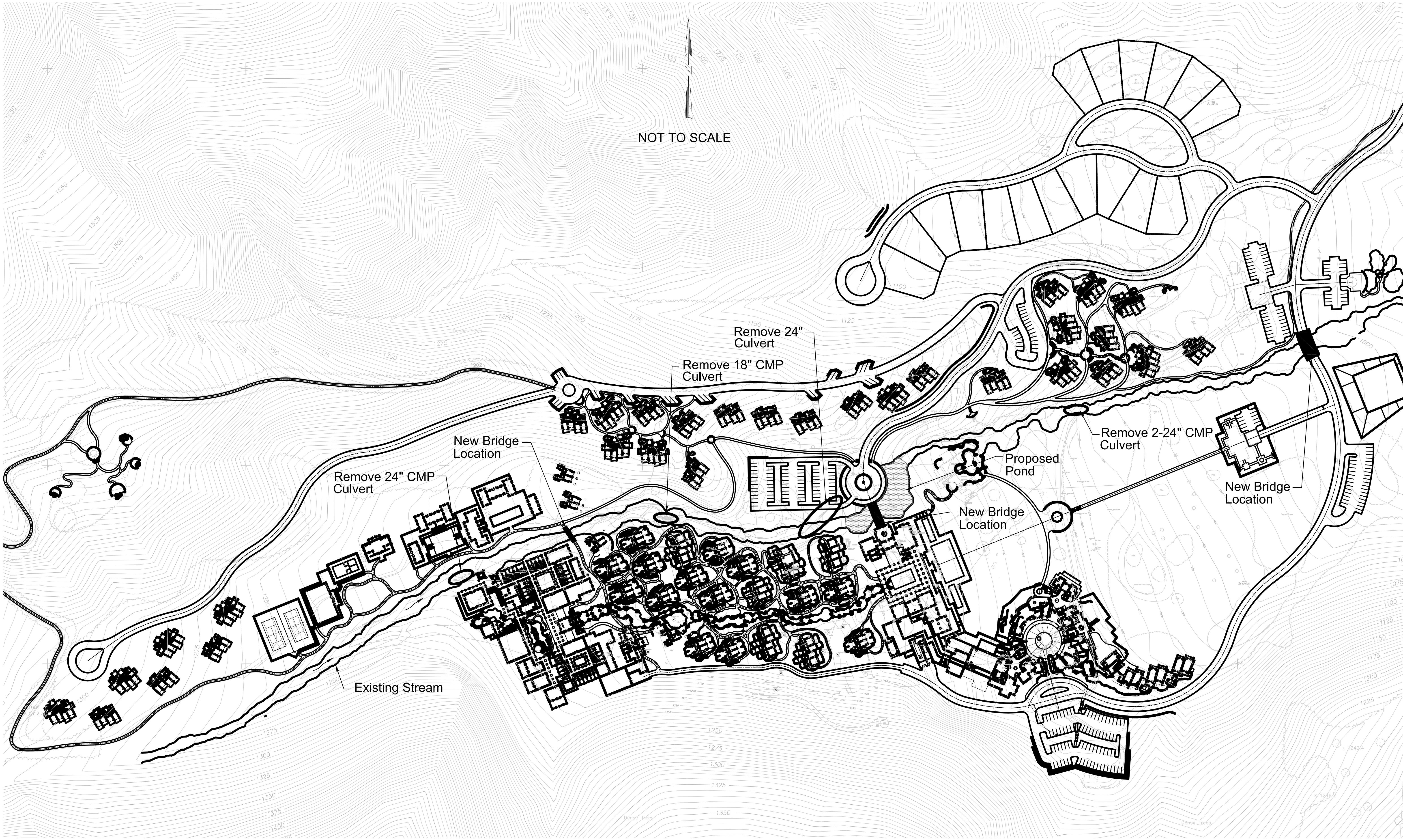
Sincerely,

CH2M HILL Engineers, Inc.

David Von Rueden
Sr. Project Manager

Attachments

c: file



Paraiso Springs Resort
Site Plan

SCALE: NOT TO SCALE DATE: 02/12/2013

Paraiso Springs Resort (PLN 040183) – Stream Setback Plan

PREPARED FOR: John Thompson/Thompson Holdings, LLC

COPY TO: file

PREPARED BY: David Von Rueden P.E. (#26428)/CH2MHILL

DATE: April 20, 2012

PROJECT NUMBER: 434834

Introduction

This Technical Memorandum (TM) is in response to a request from the Monterey County Water Resources Agency (MCWRA), as outlined in their November 24, 2010 letter to Jacqueline Onciano/Planning Department, regarding development setback from the Paraiso Springs watercourse. The subject watercourse is an unnamed intermittent drainage swale/stream that traverses the Project site, from west to east. Please refer to the *Paraiso Springs Resort – Response to Hydrology and Hydraulic Analysis and Erosion Control Measures Review Comments Technical Memorandum*, prepared by CH2MHILL and dated October 28, 2008, for additional information about this drainage feature.

Watercourse Setback Delineation

Monterey County Code Chapter 16.16.050K specifies a 50-foot setback from a watercourse for all proposed development. Please see the attached Site Plan (4 pages total) for an annotated map showing approximate watercourse top-of-bank locations, and the 50-foot setback line on either side of the watercourse. Please note that the top-of-bank has been delineated using aerial topography. The watercourse is not clearly defined along its entire length throughout the Project site, because at several locations, it is currently confined to culverts. This analysis only focused on portions of the development where a defined channel exists. All of the existing culverts will be removed from the watercourse as part of the Project.

As shown on the attached Site Plan, the proposed development would encroach into the 50-foot setback zone at several locations. Therefore, we have analyzed the significant encroachments relative to the two provisions outlined in the previously-noted County Code. The locations studied are labeled on the Site Plan as Sections A-A, B-B and C-C. Section A-A is representative of the setback encroachment from time-share-condominiums near the downstream end of the watercourse. Section B-B is located near the center of the development, where Hotel units encroach into the setback zone. Section C-C is located further upstream and indicative of the setback encroachment from the proposed spa and fitness facilities.

Watercourse Capacity

Provision 1 of County Code Chapter 16.16.050K requires that development within a setback zone not significantly reduce capacity of an existing watercourse, nor otherwise adversely affect other properties.

The capacity of the existing watercourse was initially evaluated by CH2MHILL and summarized in the TM entitled *Paraiso Springs Resort: Existing Hydrologic and Hydraulic Site Conditions*, dated July 15, 2005. This document described the flow capacity of the existing watercourse as approximately 4,000 cfs. This capacity exceeded the approximately 400 cfs of runoff from a 100-yr storm event. Subsequently, CH2MHILL re-evaluated the watershed using a more accurate HEC-HMS model and documented the post-Project 100-yr runoff rate as 316 cfs, in their 2008 TM. Based on the previous analysis and the information presented in the following section, the existing watercourse should have adequate capacity to covey the anticipated 100-yr post-project flow rate. The proposed development will not constrict or significantly reduce the existing watercourse capacity. No adverse impacts on other properties are anticipated by the proposed development.

Erosion Protection

Provision 2 of County Code Chapter 16.16.050K requires that the new development be safe from flow related erosion and not cause erosion hazards.

To address this issue, we propose to use Rock Slope Protection (RSP) in the watercourse, at all locations where building or critical roadway construction would encroach into the 50-foot setback zone. Please refer to the attached typical watercourse sections A-A, B-B and C-C for conceptual RSP installation details at critical Project locations. The conceptual RSP design is based upon the California Bank and Shore Rock Slope Protection Design Manual, published by Caltrans. Pertinent pages from this Manual are attached. The approximate stream depths and velocities were calculated manually, using King's Handbook of Hydraulics. These preliminary calculations are also attached. Table 1 summarizes the key design parameters of the RSP design.

Table 1

	Flow 1 Data (Q cfs)	Channel Roughness (n)	Average Channel Slope (s)	Channel Bottom Width (b-ft)	Channel Sideslope (z)	100-Yr Water Depth (D-ft)	Stream Velocity[1] (fps)	RSP Class[2]	RSP Thickness[3] (ft)
Section A-A	316	0.03	0.100	10	4:1	1.4	9.7	Light	3
Section B-B	316	0.03	0.053	7	2:1	2.0	9.7	Light	3
Section C-C	316	0.03	0.071	20	4:1	1:2	7.3	Light	3

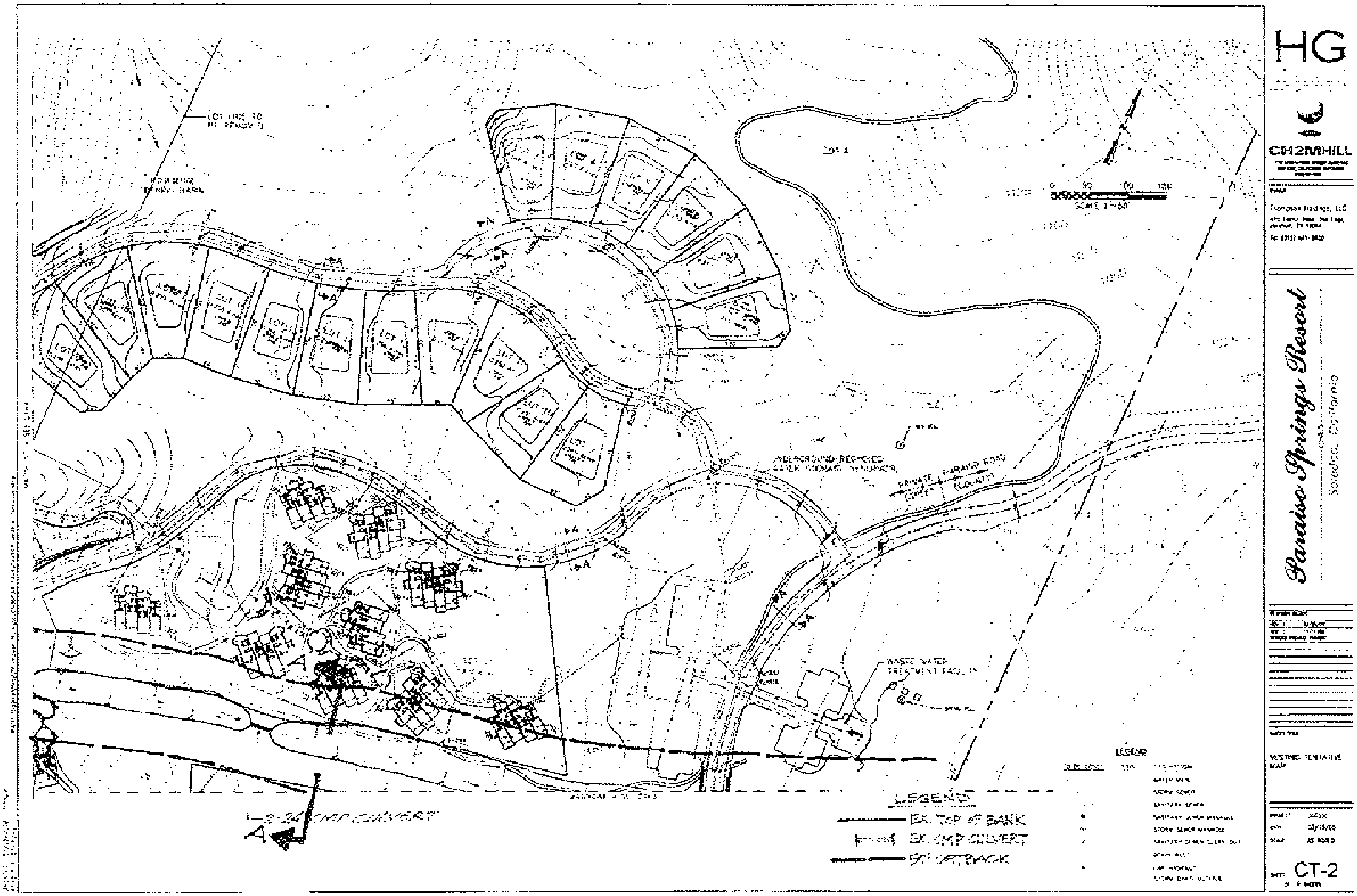
[1] Velocity shown is 67% of calculated average channel velocity, applicable for parallel flow per CALTRANS Manual.

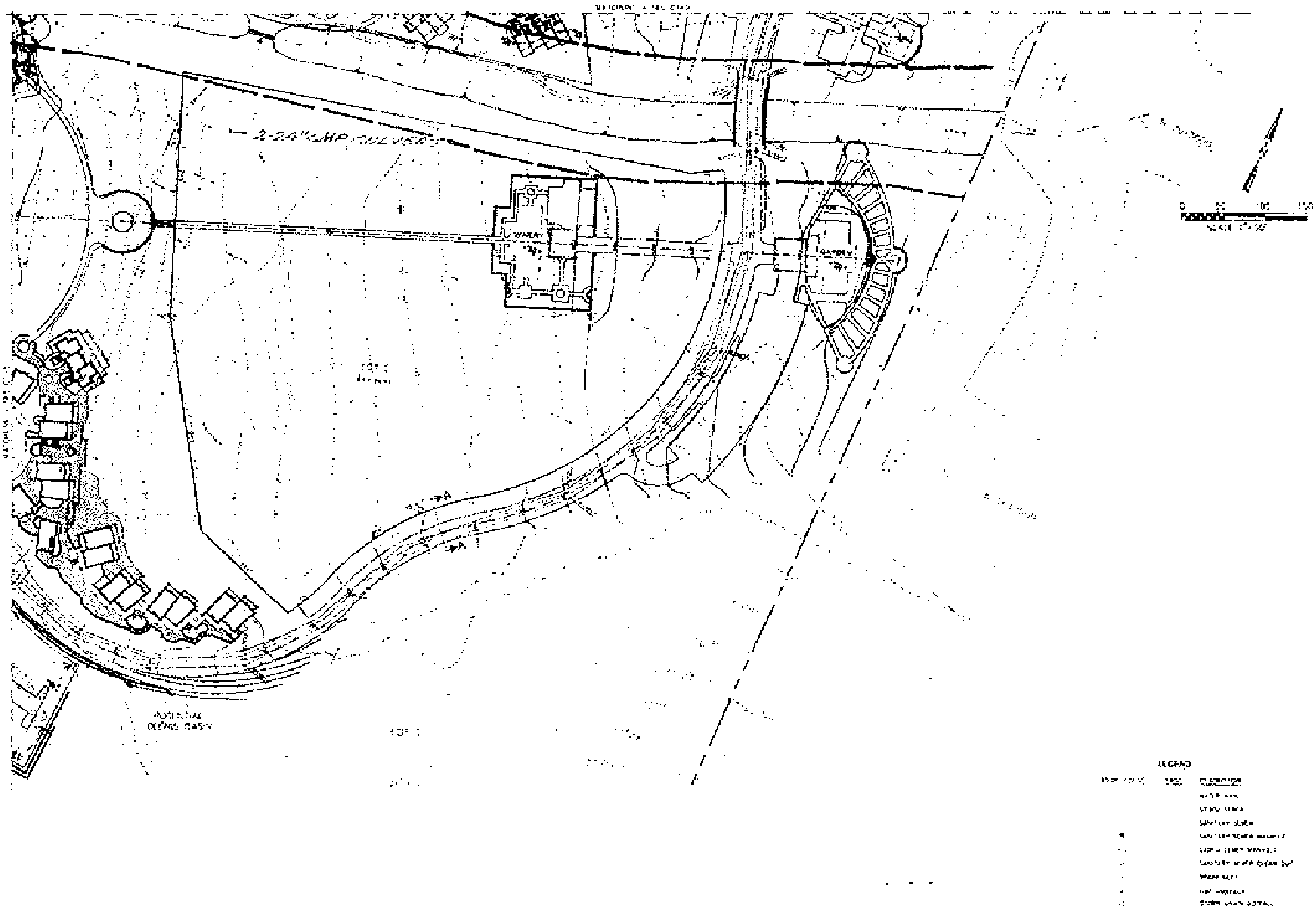
[2] RSP class is based on CALTRANS standard specifications, Section 72.

[3] RSP section includes placement of a geotextile fabric, between soil surface and rock.

Summary

It is anticipated that during Project design, the Site Plan will be refined to reduce setback zone encroachments. At locations where setback encroachments cannot be avoided, erosion control measures as described herein will be constructed within the existing watercourse for erosion protection and to preserve 100-yr flow capacity.

HG
CH2MHILL
Paradise Springs Resort
UNDERGROUND REDUCED WATER STORAGE RESERVOIR
PRIVATE PARADISE ROAD (COUNTY)
WASTE WATER TREATMENT FACILITY
LEGEND
EX. TOP OF BANK
EX. CMP CULVERT
50' SETBACK
24" CMP CULVERT
LEGEND
CT-2



HG

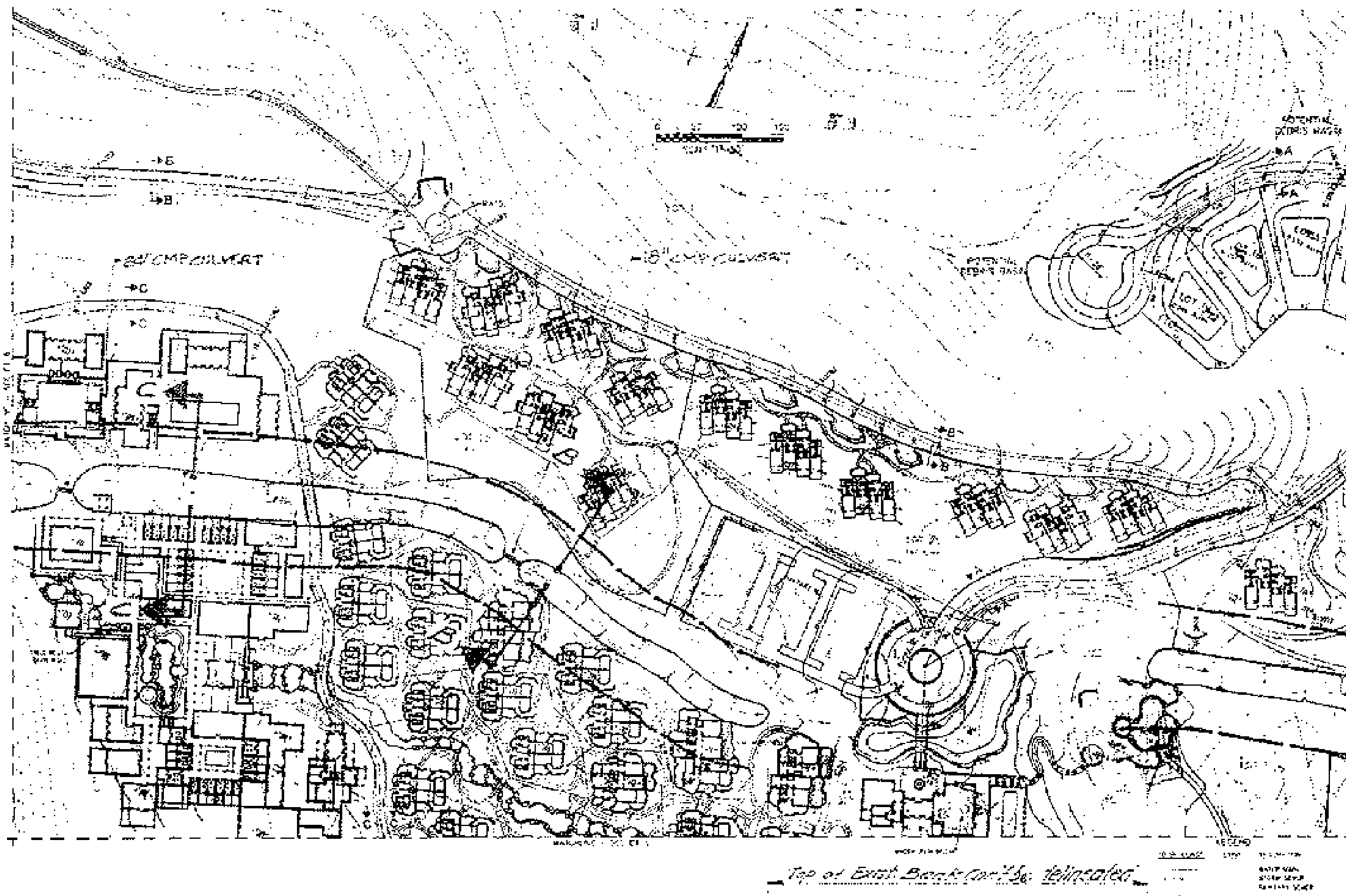
CH2MHILL

Paraiso Springs Resort

Soledad, California

VESTING TENTATIVE MAP

CT-3



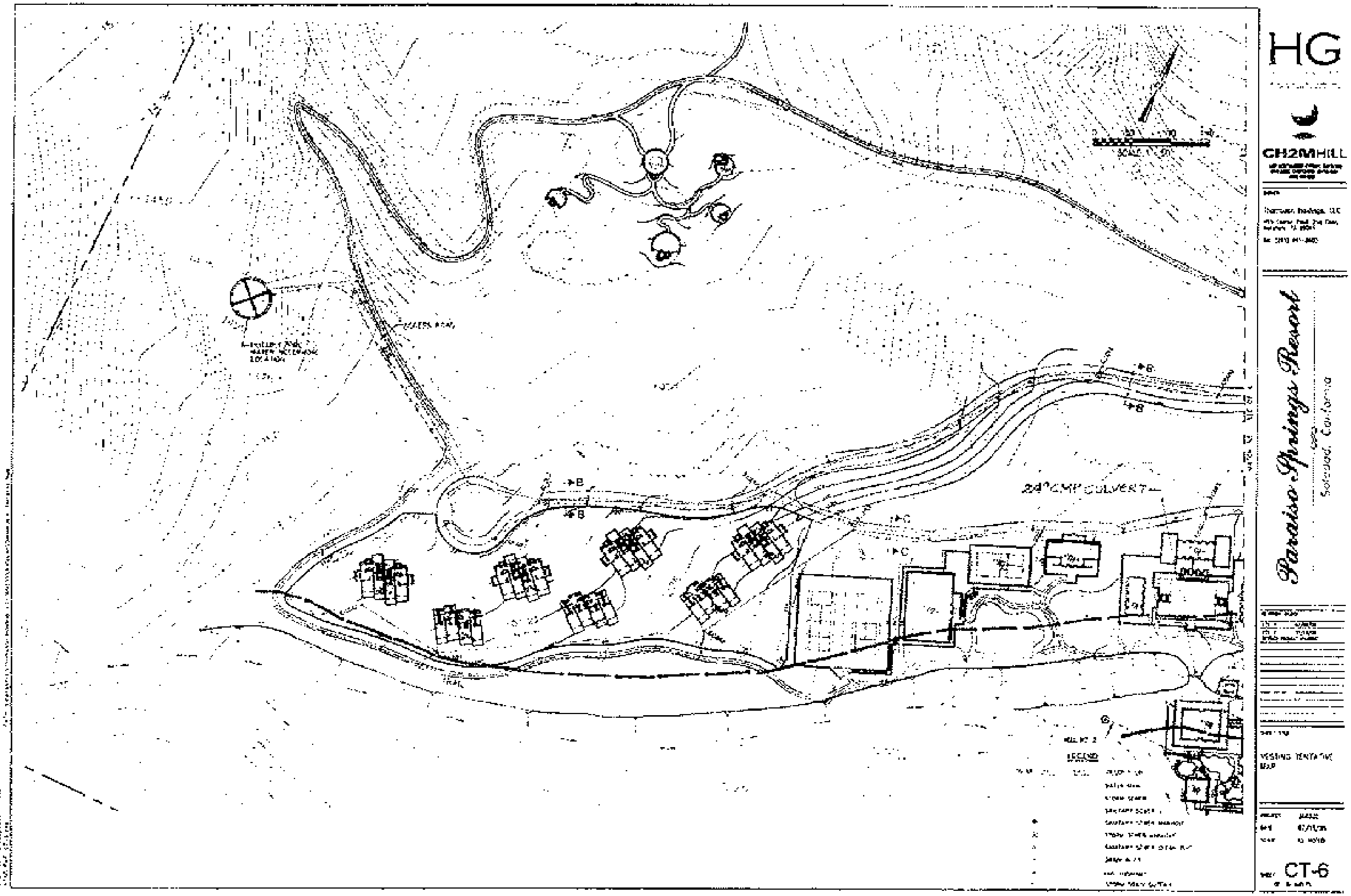
HG

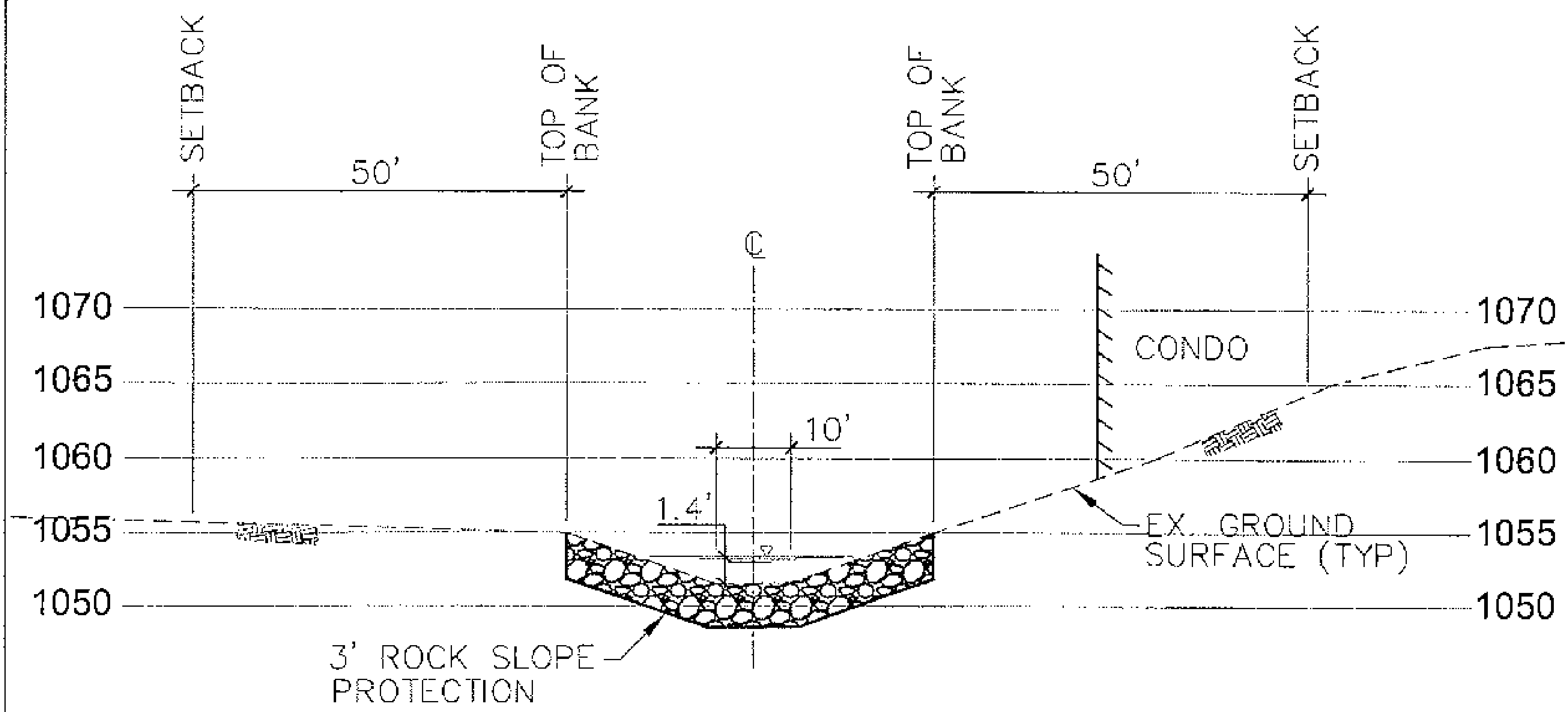
CH2MHILL

Paraiso Springs Resort

Soledad, California

CT-4

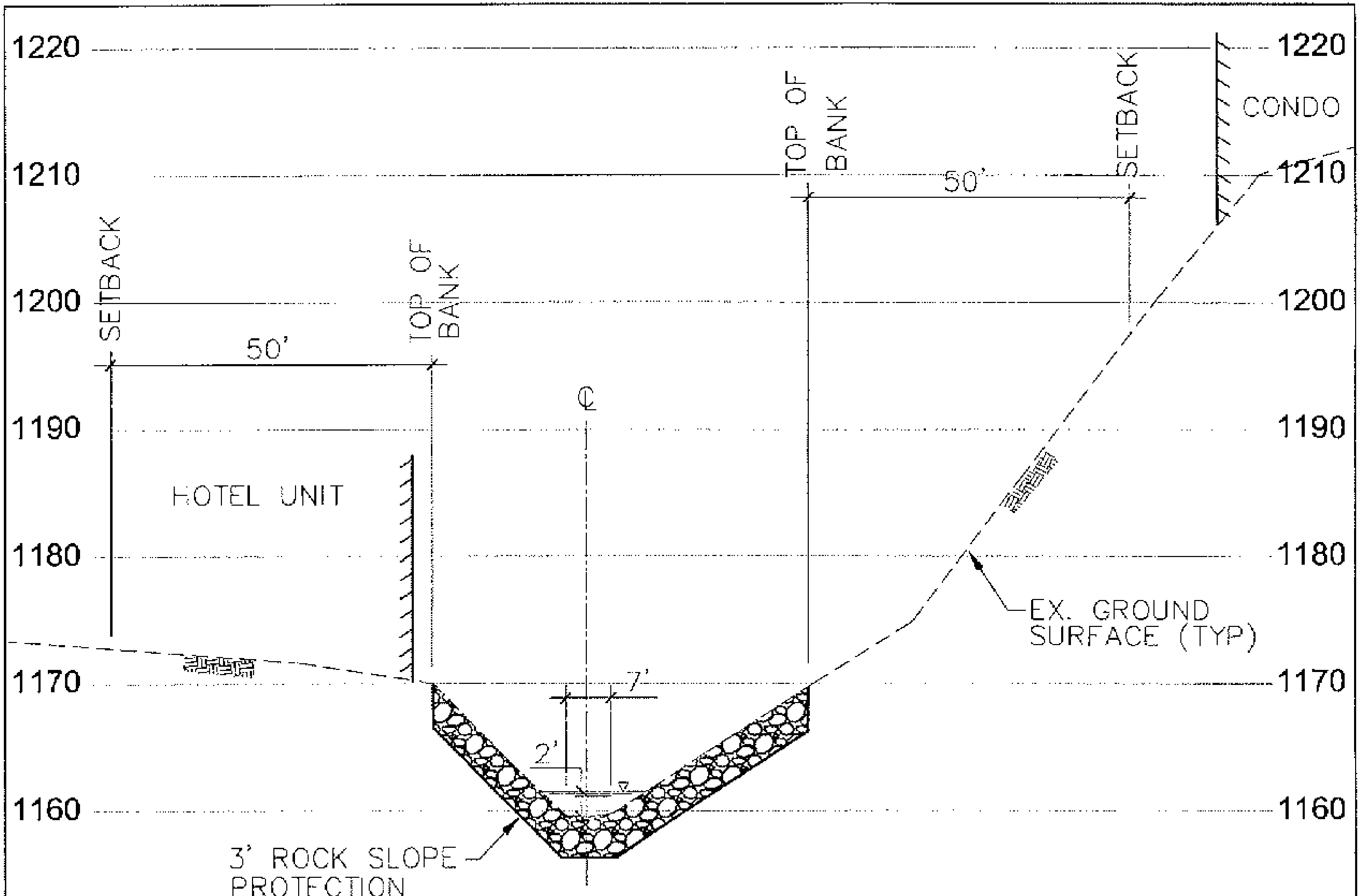

HG
CH2MHILL
Paraiso Springs Resort
Soledad, California
24"CMP CULVERT
ACCESS ROAD
LEGEND
VESTING TENTATIVE MAP
CT-6



SECTION A-A

SCALE HORZ. = 1"=20'
VERT. = 1"=10'

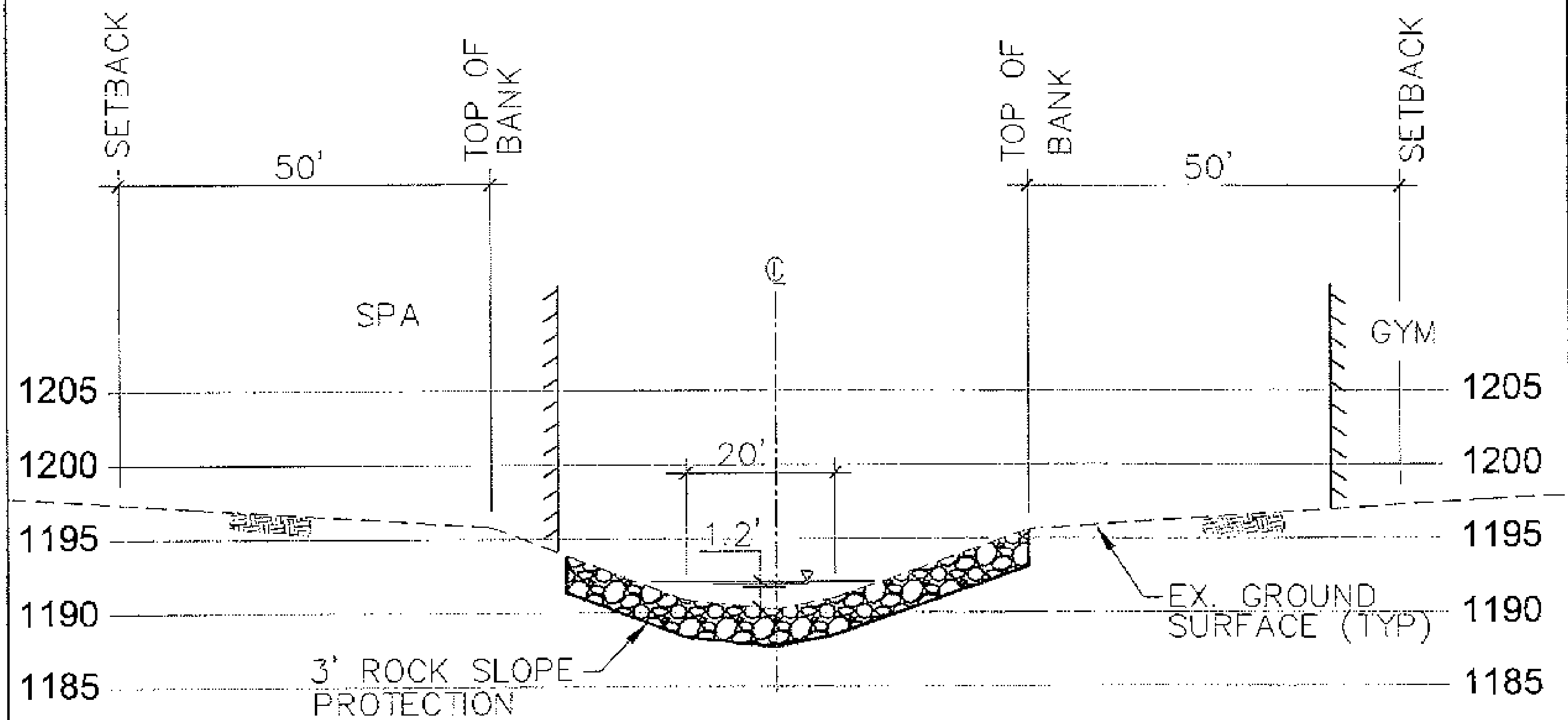
CH2MHILL
1737 NORTH FIRST STREET, SUITE 300
SAN JOSE, CALIFORNIA 95112-4524
(408) 436-4909



SECTION B-B

SCALE HORZ. = 1"=20'
VERT. = 1"=10'

CH2MHILL
1737 NORTH FIRST STREET, SUITE 300
SAN JOSE, CALIFORNIA 95112-4524
(408) 436-4909



SECTION C-C

SCALE HORZ. = 1"=20'
VERT. = 1"=10'

CH2MHILL
1737 NORTH FIRST STREET, SUITE 300
SAN JOSE, CALIFORNIA 95112-4524
(408) 436-4909

- Determine RSP Size & Thickness
  - Use King's Handbook to determine approx. stream depth.
  - Use Caltrans' Bank & Shore Protection Manual to size RSP.
  - $Q_{100} = 316$ cfs

(1) <u>Section A-A</u> $n = 0.03$, $s = 0.100$ (from topo)
$b = 10'$, $z = 4.1$

$$Q = \frac{K'}{n} b^{8/3} s^{1/2}$$

$$K' = \frac{Q(n)}{(b^{8/3})(s^{1/2})} = \frac{316(0.03)}{(10)^{8/3}(0.100)^{1/2}} = 0.06459$$

$D/b = 0.14$ (from Handbook Table 7-11)

$D = 0.14(10) = 1.4'$

$A = 22 \text{ ft}^2$

$V = Q/A = 316/22 = 14.4$ fps

Size RSP: $SG = 2.65$, $a = 14°$

$$W = \frac{0.00002\, V^6\, SG}{(SG-1)^3 \sin^3(\rho - a)}$$ (from Caltrans' Manual)

$$W = \frac{0.00002\,(14.4 \times 0.67)^6 (2.65)}{(2.65-1)^3 (\sin^3 70° - 14°)} = 16.7^{\#}$$

Use Class Light, $T = 2.5'$ (say 3' w/ geotextile

(2) Section C-C $n = 0.03$, $s = 0.071$, $b = 20'$, $z = 4:1$

$$K' = \frac{(316)(0.03)}{(20)^{8/3}(0.071)^{1/2}} = 0.01207$$

$D/b = 0.06$ (from Handbook Table 7-11)

$D = 1.2'$, $A = 29^{\#}$

$V = 316/29 = 10.9$ fps

Size RSP: $SG = 2.65$, $a = 14°$

$$W = \frac{0.00002\,(10.9 \times 0.67)^6\,(2.65)}{(2.65-1)^3 \sin^3(70-14)}$$

$W = 3.2^{\#}$

Use same RSP as for Section A-A

(3) Section B-B $n = 0.03$, $s = 0.053$, $b = 7'$, $z = 2:1$

$$K' = \frac{(316)(.03)}{(7)^{8/3}(0.053)^{1/2}} = 0.0297$$

$D/b = .29$ (from Handbook) Table 7-11

$D = 2.0$, $A = 22^{\#}$

$V = 316/22 = 14.4$ fps

Size RSP: $SG = 2.65$, $a = 26.6°$

$$W = \frac{(0.00002)(14.4 \times .67)^6(2.65)}{(2.65-1)^3 \sin^3(90-26.6)} = 24.3^{\#}$$

Use same RSP as for Section A-A

Table 7-11. Values of K' in Formula $Q = \frac{K'}{n} b^{8/3} s^{1/2}$ for Trapezoidal Channels

D = depth of water b = bottom width of channel

	$\frac{D}{b}$	Vertical	¼-1	½-1	¾-1	1-1	1½-1	2-1	2½-1	3-1	4-1
		Side slopes of channel, ratio of horizontal to vertical									
	.01	.00068	.00068	.00069	.00069	.00069	.00069	.00069	.00069	.00070	.00070
	.02	.00213	.00215	.00216	.00217	.00218	.00220	.00221	.00222	.00223	.00225
	.03	.00414	.00419	.00423	.00426	.00429	.00435	.00436	.00439	.00443	.00449
	.04	.00660	.00670	.00679	.00686	.00691	.00700	.00708	.00716	.00723	.00736
	.05	.00946	.00964	.00979	.00991	.01002	.01019	.01033	.01047	.01060	.01086
C-C	.06	.0127	.0130	.0132	.0134	.0136	.0138	.0141	.0143	.0145	.0150
	.07	.0162	.0166	.0170	.0173	.0175	.0180	.0183	.0187	.0190	.0197
	.08	.0200	.0206	.0211	.0215	.0219	.0225	.0231	.0236	.0240	.0250
	.09	.0241	.0249	.0256	.0262	.0267	.0275	.0282	.0289	.0296	.0310
	.10	.0284	.0294	.0304	.0311	.0318	.0329	.0339	.0349	.0358	.0375
	.11	.0329	.0343	.0354	.0364	.0373	.0387	.0400	.0413	.0424	.0448
	.12	.0376	.0393	.0408	.0420	.0431	.0450	.0466	.0482	.0497	.0527
A-A	.13	.0425	.0446	.0464	.0480	.0493	.0516	.0537	.0556	.0575	.0613
	.14	.0476	.0502	.0524	.0542	.0560	.0587	.0612	.0636	.0659	.0706
	.15	.0528	.0559	.0585	.0608	.0627	.0662	.0692	.0721	.0749	.0805
	.16	.0582	.0619	.0650	.0676	.0700	.0740	.0777	.0811	.0845	.0912
	.17	.0638	.0680	.0716	.0748	.0775	.0823	.0866	.0907	.0947	.1026
	.18	.0695	.0744	.0786	.0822	.0854	.0910	.0960	.1008	.1055	.1148
	.19	.0753	.0809	.0857	.0899	.0936	.1001	.1059	.1115	.1169	.1277
	.20	.0812	.0876	.0931	.0979	.1021	.1096	.1163	.1227	.1290	.1414
	.21	.0873	.0945	.101	.106	.111	.120	.127	.135	.142	.156
	.22	.0934	.1015	.109	.115	.120	.130	.139	.147	.155	.171
	.23	.0997	.1087	.117	.124	.130	.141	.150	.160	.169	.187
	.24	.1061	.1161	.125	.133	.140	.152	.163	.173	.184	.204
	.25	.1125	.1236	.133	.142	.150	.163	.176	.188	.199	.222
	.26	.119	.131	.142	.152	.160	.175	.189	.202	.215	.241
	.27	.126	.139	.151	.162	.171	.188	.203	.218	.232	.260
	.28	.132	.147	.160	.172	.182	.201	.217	.234	.249	.281
B-B	.29	.139	.155	.170	.182	.194	.214	.232	.250	.268	.302
	.30	.146	.163	.179	.193	.205	.228	.246	.267	.287	.324
	.31	.153	.172	.189	.204	.218	.242	.264	.285	.306	.347
	.32	.160	.180	.199	.215	.230	.256	.281	.304	.327	.371
	.33	.167	.189	.209	.227	.243	.271	.298	.323	.348	.396
	.34	.174	.198	.219	.238	.256	.287	.316	.343	.370	.423
	.35	.181	.207	.230	.251	.269	.303	.334	.363	.392	.450
	.36	.189	.216	.241	.263	.283	.319	.353	.385	.416	.478
	.37	.196	.225	.252	.275	.297	.336	.372	.406	.440	.507
	.38	.203	.234	.263	.288	.312	.353	.392	.429	.465	.537
	.39	.211	.244	.274	.301	.326	.371	.413	.452	.491	.568
	.40	.218	.253	.286	.315	.341	.389	.434	.476	.518	.600
	.41	.226	.263	.297	.328	.357	.408	.456	.501	.546	.633
	.42	.233	.273	.309	.342	.373	.427	.478	.526	.574	.668
	.43	.241	.283	.321	.357	.389	.447	.501	.553	.603	.703
	.44	.248	.293	.334	.371	.405	.467	.525	.580	.633	.740
	.45	.256	.303	.346	.386	.422	.488	.549	.607	.664	.777

Table 7-11. Values of K' in Formula $Q = \frac{K'}{n} b^{8/3} s^{1/2}$ for Trapezoidal Channels (*Continued*)

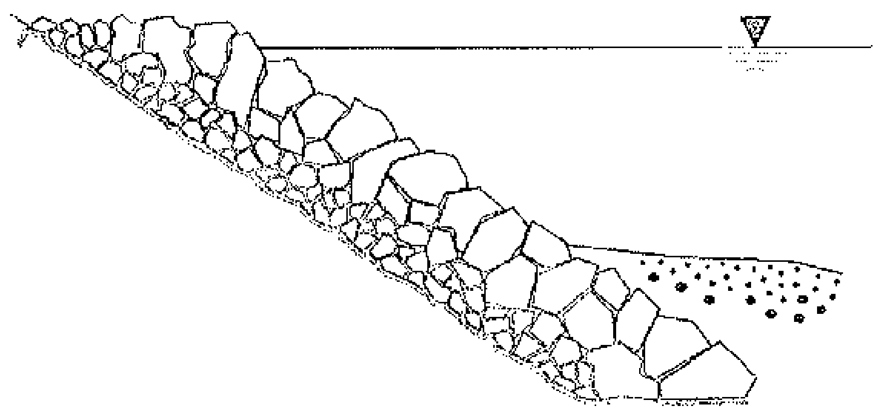
D = depth of water b = bottom width of channel

$\frac{D}{b}$	Vertical	¼-1	½-1	¾-1	1-1	1½-1	2-1	2½-1	3-1	4-1
	Side slopes of channel, ratio of horizontal to vertical									
.46	.264	.313	.359	.401	.439	.509	.574	.636	.696	.816
.47	.271	.323	.372	.416	.457	.531	.599	.665	.729	.856
.48	.279	.334	.385	.432	.474	.553	.625	.695	.762	.897
.49	.287	.344	.398	.447	.493	.576	.652	.725	.797	.939
.50	.295	.355	.412	.463	.511	.598	.679	.757	.833	.983
.51	.303	.366	.425	.480	.530	.622	.707	.789	.869	1.03
.52	.311	.377	.439	.496	.549	.646	.736	.822	.907	1.07
.53	.319	.388	.453	.513	.569	.671	.765	.856	.945	1.12
.54	.327	.399	.467	.531	.589	.696	.795	.891	.984	1.17
.55	.335	.410	.482	.548	.609	.722	.826	.926	1.025	1.22
.56	.343	.422	.497	.566	.630	.748	.857	.963	1.07	1.27
.57	.351	.433	.511	.584	.651	.775	.889	1.000	1.11	1.32
.58	.359	.445	.526	.602	.673	.802	.922	1.038	1.15	1.37
.59	.367	.456	.542	.621	.694	.830	.956	1.077	1.20	1.42
.60	.375	.468	.557	.640	.717	.858	.990	1.117	1.24	1.48
.61	.383	.480	.573	.659	.739	.887	1.02	1.16	1.29	1.54
.62	.391	.492	.588	.678	.762	.916	1.06	1.20	1.33	1.60
.63	.399	.504	.604	.698	.786	.946	1.10	1.24	1.38	1.66
.64	.408	.516	.620	.718	.809	.977	1.13	1.28	1.43	1.72
.65	.416	.529	.637	.738	.833	1.008	1.17	1.33	1.48	1.79
.66	.424	.541	.653	.759	.857	1.04	1.21	1.37	1.53	1.85
.67	.433	.553	.670	.780	.882	1.07	1.25	1.42	1.59	1.91
.68	.441	.566	.687	.801	.907	1.10	1.29	1.47	1.64	1.98
.69	.449	.579	.704	.822	.933	1.14	1.33	1.51	1.69	2.05
.70	.457	.592	.722	.844	.959	1.17	1.37	1.56	1.75	2.12
.71	.466	.604	.739	.866	.985	1.21	1.41	1.61	1.81	2.19
.72	.474	.617	.757	.888	1.012	1.24	1.46	1.66	1.86	2.26
.73	.483	.631	.775	.911	1.039	1.28	1.50	1.71	1.92	2.34
.74	.491	.644	.793	.934	1.067	1.31	1.54	1.77	1.98	2.41
.75	.499	.657	.811	.957	1.095	1.35	1.59	1.82	2.05	2.49
.76	.508	.670	.830	.981	1.12	1.39	1.63	1.87	2.11	2.57
.77	.516	.684	.849	1.005	1.15	1.43	1.68	1.93	2.17	2.65
.78	.525	.698	.868	1.029	1.18	1.46	1.73	1.99	2.24	2.73
.79	.533	.711	.887	1.054	1.21	1.50	1.78	2.04	2.30	2.81
.80	.542	.725	.906	1.078	1.24	1.54	1.83	2.10	2.37	2.90
.81	.550	.739	.925	1.10	1.27	1.58	1.88	2.16	2.44	2.98
.82	.559	.753	.945	1.13	1.30	1.62	1.93	2.22	2.51	3.07
.83	.567	.767	.965	1.15	1.33	1.67	1.98	2.28	2.58	3.16
.84	.576	.781	.985	1.18	1.36	1.71	2.03	2.34	2.65	3.25
.85	.585	.796	1.006	1.21	1.40	1.75	2.08	2.41	2.72	3.35
.86	.593	.810	1.03	1.23	1.43	1.79	2.14	2.47	2.80	3.44
.87	.602	.825	1.05	1.26	1.46	1.84	2.19	2.54	2.87	3.54
.88	.610	.839	1.07	1.29	1.49	1.88	2.25	2.60	2.95	3.63
.89	.619	.854	1.09	1.31	1.53	1.93	2.31	2.67	3.03	3.73
.90	.628	.869	1.11	1.34	1.56	1.98	2.36	2.74	3.11	3.83

State of California
Department of Transportation
Engineering Service Center
Office of Structural Foundations
Transportation Laboratory

CALIFORNIA BANK AND SHORE ROCK SLOPE PROTECTION DESIGN

Practitioner's Guide and Field Evaluations of Riprap Methods



Final Report No. FHWA-CA-TL-95-10
Caltrans Study No. F90TL03

Third Edition - Internet
October 2000

Prepared in Cooperation with the US Department of Transportation
Federal Highway Administration

5-1-C. Determine Minimum Stone Weight. Solve Equation 1 for W in US customary units. To get values in System International (SI), metric units, first divide the weight of minimum stable rock, W in pounds by 2.2 to get W in kilograms, then divide by 1000 to get W in tonnes. Use W later in section 5-1-D.
See Figure 5-1 for key variables in Equation 1.

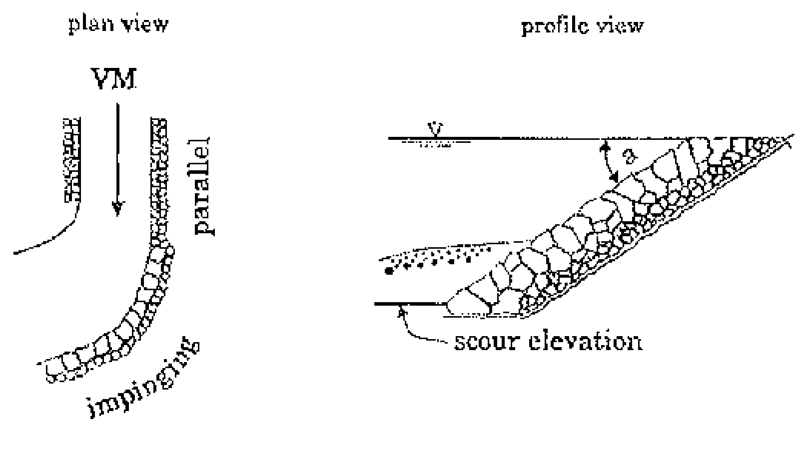
Equation 1.

$$W = \frac{0.00002}{(SG-1)^3} \; \frac{V^6}{SIN^3(r-a)} \; SG$$

W = theoretical **minimum rock mass (size or weight)** which resists forces of flowing water and remains stable on slope of stream or river bank, POUNDS.
V = velocity to which bank is exposed, FEET PER SECOND.
  for PARALLEL flow multiply average channel velocity VM by 0.67 (2/3)
  for IMPINGING flow multiply average channel velocity VM by 1.33 (4/3)
SG = specific gravity of the rock.
r = 70 DEGREES (for randomly placed rubble, a constant).
a = outside slope face angle with horizontal, DEGREES.

In profile, the lower elevation limit of riverbank RSP is based on expected scour (determined by experience, measurements, or scour equations). The upper elevation limit is based on design high water, although it may be set higher.

Figure 5-1. Key Variables in Equation 1



STANDARD Rock SIZE or Rock MASS or Rock WEIGHT		GRADING OF ROCK SLOPE PROTECTION **PERCENTAGE LARGER THAN**											
		RSP-Classes [A]											
		Method A Placement					Method B Placement						
		RSP-Classes other than Backing									Backing No.		
US unit		8 ton	4 ton	2 ton	1 ton	1/2 ton	1 ton	1/2 ton	1/4 ton	Light	1 [B]	2	3
	SI unit	8 T	4 T	2 T	1 T	1/2 T	1 T	1/2 T	1/4 T	Light	1 [B]	2	3
16 ton	14.5 tonne	0-5											
8 ton	7.25 tonne	50-100	0-5										
4 ton	3.6 tonne	95-100	50-100	0-5									
2 ton	1.8 tonne		95-100	50-100	0-5		0-5						
1 ton	900 kg			95-100	50-100	0-5	50-100	0-5					
1/2 ton	450 kg				95-100	50-100	----	50-100	0-5				
1/4 ton	220 kg					95-100	95-100	-----	50-100	0-5			
200 lb	90 kg							95-100	----	50-100	0-5		
75 lb	34 kg								95-100	-----	50-100	0-5	
25 lb	11 kg									95-100	90-100	25-75	0-5
5 lb	2.2 kg											90-100	25-75
1 lb	0.4 kg												90-100

[A] US customary names (units) of RSP-Classes listed above SI names, example US is "2 ton" metric is "2 T".
[B] "Facing" has same gradation as "Backing No. 1". To conserve space "Facing" is not shown .

Example for determining RSP-Class of outside layer. By using Equation 1, if the calculated W=135 kg (minimum stable rock size):
1. Enter table at left and select closest value of STANDARD Rock SIZE which is greater than calculated W, in this case 220 kg
2. Trace to right and locate "50-100" entry 3. Trace upward and read column heading "1/4 T", then 1/4 T is first trial RSP-Class.

Table 5-1. Guide for Determining RSP-Class of Outside Layer

Table 5-3. Minimum Layer Thickness SI metric (US customary)		
RSP-Class Layer	Method of Placement	Minimum Thickness
8 T (8 ton)	A	2.60 meters (8.5 feet)
4 T (4 ton)	A	2.07 meters (6.8 feet)
2 T (2 ton)	A	1.65 meters (5.4 feet)
1 T (1 ton)	A	1.31 meters (4.3 feet)
1/2 T (1/2 ton)	A	1.04 meters (3.4 feet)
1 T (1 ton)	B	1.65 meters (5.4 feet)
1/2 T (1/2 ton)	B	1.31 meters (4.3 feet)
1/4 T (1/4 ton)	B	1.00 meters (3.3 feet)
Light	B	760 millimeters (2.5 feet)
Facing	B	550 millimeters (1.8 feet)
Backing No. 1	B	550 millimeters (1.8 feet)
Backing No. 2	B	380 millimeters (1.25 feet)
Backing No. 3	B	230 millimeters (0.75 feet)

For total thickness, add each layer thickness. Use zero thickness for the RSP-fabric. Before adopting values in Table 5-3, consult with a materials engineer about rock sources, quality, shapes, and specific gravity. Calculate new thickness values if the shape factor is not spherical and specific gravity is not reasonably close to 2.65. "Minimum Thickness" values were calculated by starting with US customary units, hard-converting to a value in feet, then soft-converting to SI metric values.

5-1-G. Review Hydraulic Calculations at Site With RSP and Possibility of Vegetation. This step of the layered design process is required to help assure future success of the revetment under changed channel dimensions, roughness coefficients, and other permit/agreement requirements. Examples are: filling voids among RSP with soil and/or covering RSP with soil then planting local species, and/or enhancing fish habitat by placing large-sized rock along the toe. Discuss site hydraulics with people of permit agencies and feasible revegetation efforts. Historically, sites with no prior vegetation are usually not revegetated, especially when subjected to scouring velocities or high wave attack.