4.2 GEOLOGY AND SOILS

This section is based on the six phases of geologic and hydrologic characterization activities completed to date at the project site. The initial study was completed by Geotechnical Consultants, Inc. (GCI) for the County of San Diego and the U.S. Department of Interior (GCI, 1989). The second and third phases were completed by Geraghty and Miller (G&M, 1988, 1990). The fourth phase comprised the work of Woodward-Clyde Consultants completed in 1991 and reported in 1995 (WCC, 1995). The fifth phase was the hydrogeologic study completed by GeoLogic Associates (GLA, 1997), and the sixth phase addressed geotechnical issues (GLA, 1998 and 1999). Phase 6, which address geotechnical issues, is contained in Appendix F. In addition, GLA completed a Geophysical Study of Potential Borrow Areas (1998) which is also contained in Appendix F.

4.2.1 EXISTING CONDITIONS

Gregory Canyon is located in the central portion of the Peninsular Ranges geomorphic province, an area characterized by northwesterly trending mountains and intervening valleys. This geomorphic province extends from the Los Angeles Basin into Baja California, Mexico. Major drainage systems generally traverse the province in a westerly direction, and in northern San Diego County include, from north to south, the Santa Margarita, San Luis Rey and San Dieguito rivers. The proposed landfill is located in Gregory Canyon, a north-draining tributary canyon of the San Luis Rey River valley, the major east-west drainage in the northern part of San Diego County (Exhibit 3-2).

4.2.1.1 Topography

Regional topography in the Peninsular Range is characterized by considerable relief with relatively moderate to steep slopes. Most of the area is undergoing erosion and mass wasting, but the major river valleys have thick accumulations of sediments, technically referred to as alluvium. The alluvium undergoes cycles of deposition and erosion, depending on the water flow in the drainage system. Typically, the rivers are at low flows during the summer months and have variable flows during the winter rainy season.

The existing slopes on the lower area of Gregory Canyon are about 5:1 (horizontal-to-vertical ratio), become 2:1 to the east, and are 1:1 and steeper on the upper part of the eastern slope. The western flank of the canyon is defined by a rounded ridgeline, with rather uniform slopes at inclinations of 2:1 to 3:1.

East of Gregory Canyon, Gregory Mountain rises steeply to a maximum elevation of 1,844 feet above mean sea level (amsl). The western ridge rises to a maximum elevation of 940 feet amsl. The thalweg (i.e., the flow line) of the canyon itself drops in elevation from 920 ft amsl at the head of the canyon on the south to 320 feet amsl on its northern terminus into the San Luis Rey River.

4.2.1.2 Regional Stratigraphy

Pre-batholithic, metasedimentary and metavolcanic rocks[1] outcrop throughout the Peninsular Ranges. Examples include outcrops of Paleozoic limestone in Riverside County, and a thick sequence of altered Jurassic gneisses in the San Jacinto Mountains. In San Diego County, outcrops include the Triassic/Jurassic[2] Bedford Canyon sedimentary sequence and the overlying Jurassic Santiago Peak volcanics.

Late Cretaceous sedimentary rocks in the Camp Pendleton area include the largely non-marine Trabuco Formation, and the marine Williams Formation, which in the San Luis Rey and Encinitas areas, are grouped in the Lusardi and Point Loma Formations. Cretaceous rocks are not exposed in the immediate vicinity of the project site.

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[1] Geologists recognize three major rock families: igneous, sedimentary, and metamorphic. Igneous rocks are formed from the solidification of hot molten mass—magma—and are subcategorized as intrusive if they cool slowly and crystallize underground. Rocks formed this way include varieties such as granite, leucogranodiorite, tonalite, and gabbro. Volcanic rock is a kind of igneous rocks that forms when magma reaches the surface of the Earth, in the form of lava flows or clots of magma, and cools rapidly. Rocks formed this way include varieties such as rhyolite, andesite, basalt, and tuff. Sedimentary rocks are either made up of particles derived from the breakdown of pre-existing rocks, or by direct chemical precipitation of minerals from seawater. Examples of the former include sandstones, shales and conglomerates. Examples of the latter include rocks such as limestone and chert. Metamorphic rocks are formed by recrystallization of one of the other types of rock, in response to earth pressures, heat, and chemically active fluids. If the original rock type can still be recognized, then the metamorphic change is recognized by adding the prefix "meta" to the original rock family, such as in metasedimentary or metavolcanic. When recrystallization is advanced the rocks are given specific names to indicate their metamorphic origin, such as gneiss, schist, slate, amphibolite, quartzite, or marble.

[2] The terms Triassic, Jurassic, and Cretaceous refer to geologic periods in the Earth's history, as summarized in the following table:

	Era	Period	Epoch	Approx. Oldest Date (in millions of years before present)
Phanerozoic	Cenozoic	Quaternary	Holocene	0.011
			Pleistocene	2
		Tertiary	Pliocene	5
			Miocene	24
			Oligocene	37
			Eocene	58
			Paleocene	66
	Mesozoic	Cretaceous		144
		Jurassic		208
		Triassic		245
	Paleozoic	Permian		286
		Pennsylvanian		320
		Mississippian		360
		Devonian		408
		Silurian		438
		Ordovician		505
		Cambrian		570
	Precambrian			4,600

Post-Cretaceous rocks lie unconformably (i.e., younger strata were deposited after a period of erosion) on either the Cretaceous rocks or the crystalline basement, but are largely confined to coastal margins some distance from the project site.

In many instances, the crystalline rocks are covered by residual soils,[3] or colluvial, and alluvial deposits.[4] The colluvial deposits are typically located along the base of slopes and are formed as a result of the downslope movement of soil and rock by the force of gravity. The alluvial deposits are found to some degree in most drainages, with deposits of considerable thickness present in the major river valleys.

4.2.1.3 Site Stratigraphy

Various geologic units occur within the project site (Exhibit 4.2-1). In the lower portions of Gregory Canyon, a thin veneer of unconsolidated residual soils, colluvial, and alluvial deposits mantles a substrate of weathered tonalite. The topographic highs bounding the canyon are formed by igneous intrusive and metamorphic rocks with varying degrees of weathering. The following subsections describe in detail the geologic units that are exposed at the site.

Surficial soils

According to Woodward-Clyde (1995), the topsoil units encountered in the area vary in thickness from about six inches to three feet, and are composed of silty sand, silty sand with clay, and silty sand with cobbles and boulders.[5] In general, the steeper, upper slope areas of the canyon are expected to have slightly thinner soil accumulations than the intermediate or lower slope areas. Underlying the topsoil are residual soil horizons or weathered rocks.

Colluvium

Colluvium forms a veneer over most of the surface of the project site. In most instances it is formed by silty sand with rock fragments that range in size from gravel to very large boulders. Finer-grained deposits, largely devoid of rock fragments, were encountered in test pits located at the southern end of the canyon (Exhibit 4.2-2). In this area, older colluvium, consisting of clayey sand to sandy clay with varying contents of rock fragments and slight to moderate cementation was encountered.

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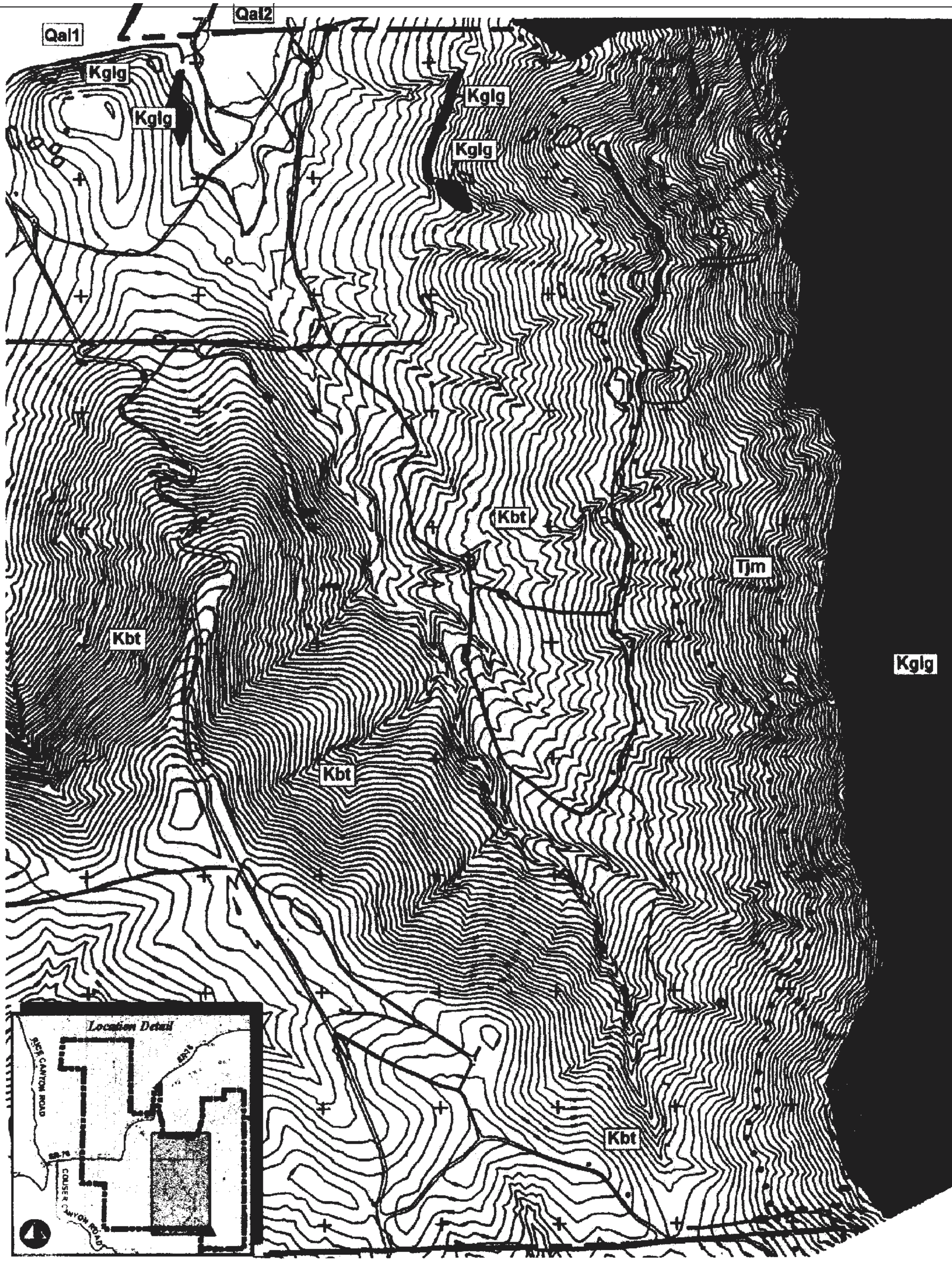
[3] Residual soil is the name given to soil that is formed in-place by the weathering of the underlying rocks.

[4] Colluvium is a general term applied to any loose soil material or rock fragments deposited by rainwash, sheetwash, or slow downslope creep under the influence of gravity. Alluvium is a general term applied to any soil material deposited by creeks, streams, and rivers.

[5] Sediments and soils are frequently classified by the size of their particles. The different sizes receive different names. The following table lists these names, together with a typical example of the size in question:

Name	Example
Clay	talcum powder
Silt	fine dust
Sand	sugar to pool salt
Gravel	peas to lemons
Cobbles	grapefruits to melons
Boulder	basketballs to VW bugs



Qal1 - San Luis Rey alluvium
Qal2 - Gregory Canyon alluvium
Kglg - Leucogranodiorite
Kbt - Tonalite
. . . . - Buried Geologic contact
Tjm - Metamorphic rocks

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N

0 500 FEET

Exhibit 4.2-1
Geologic Map of the Site

Sources: GeoLogic Associates, 1998; David Evans and Associates, Inc., 1999; PCR Services Corporation, 1999

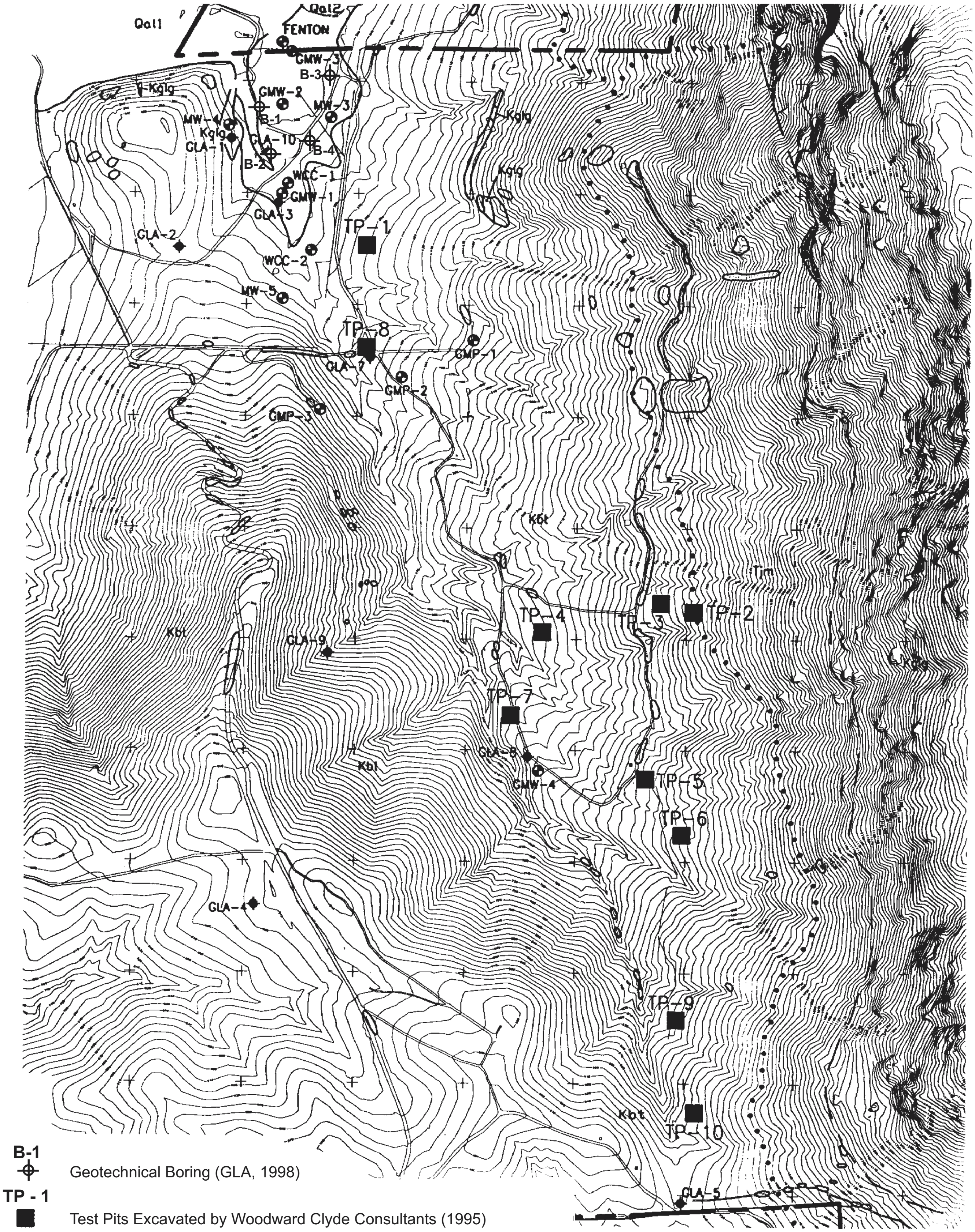

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Exhibit 4.2-2
Location of Test Pits

Sources: GeoLogic Associates, 1998; David Evans and Associates, Inc., 1999; PCR Services Corporation, 1999

Rock fragments exposed at the surface of the colluvial veneer vary from gravel- to boulder-size material. Boulders of leucogranodiorite, some in excess of 20 feet in maximum dimension, are present along much of the eastern sideslopes.

The thickness of the colluvial deposits in the proposed project area is highly variable with interpretations by Geragthy & Miller (1990) indicating thicknesses from 2 to 50 feet. In general, the upper slope areas are likely to be underlain by thin (less than 10 foot thick) colluvial deposits and surficial soils formed on highly weathered crystalline rock. Debris chutes and drainage channels are expected to be locally backfilled with colluvium of moderate thickness (10 to 20 feet). Colluvial deposits are expected to be thickest near the lower portion of the slopes.

Alluvium

Two alluvial units have been mapped at lower elevations near the mouth of Gregory Canyon (Exhibit 4.2-1). The younger subunit (Qal-1) consists of overbank deposits from the active San Luis Rey River channel, interbedded with channel deposits from the Gregory Canyon drainage.

These deposits are relatively thin and contain gravels, cobbles and boulders, supported by a sandy silt matrix. The older alluvial subunit (Qal-2) is a terrace remnant of older alluvium in the Gregory Canyon drainage.

The alluvial wedge pinches out to the south. The wedge thickens to the north until it eventually merges with the channel deposits of the San Luis Rey River. Well GMW-2 (Exhibit 4.2-2), near the mouth of the canyon, traversed through a 50-foot section of alluvial deposits before reaching the underlying bedrock.

Bedrock

Larsen (1948) used the term Bonsall Tonalite to describe the rocks underlying the western ridge of Gregory Canyon, and the term Indian Mountain Leucogranodiorite to describe the light-colored, bold outcrops of granitic rock underlying the eastern ridge. Larsen also mapped an intervening screen or wedge of metamorphic rock along the lower slopes of the eastern ridge, which he correlated with the sedimentary Triassic/Jurassic Bedford Canyon Formation. Rocks of this unit have relict volcanic textures, however, and are probably best correlated with the Jurassic Santiago Peak volcanics.

The contacts between all three units are intrusive, albeit different in nature. The main body of the leucogranodiorite is in intrusive contact with the metamorphic screen midway along the easterly slope of Gregory Canyon. The contact zone is narrow and abrupt where it can be observed, and has the characteristic features of sharp intrusive contact with apophyses[6] and dikes[7] extending from the leucogranodiorite into the metamorphic rock. No evidence of shearing is observed in the outcrop.

The intrusive contact between the tonalite and the metamorphic wedge is somewhat transitional because of the effects of partial melting. Mafic or intermediate magmas (gabbro to tonalite) are

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6 Apophysis is defined as a branch from a vein or fracture that has been filled by the injection of a larger intrusive body.

7 Dike is a tabular body of intrusive magma that cuts across the massive rocks.

emplaced at a relatively high temperature (1,200º to 900º C), so the contacts between them and the host rock tend to be anatectic (i.e., they are accompanied by partial melting of the pre-existing rock). The pre-metamorphic rock fabric can be completely obliterated by migmatization (i.e., development of a banded aspect in the rock as a result of partial melting), so along the contact zone it is not always easy to discriminate between the intrusion and the host rock. The intrusive nature of the contact has been documented at several field locations, however, and is characteristically irregular and intricate. No evidence of shearing is observed in the outcrop and the contact lacks the planar expression expected of a shear zone.

Metamorphic rocks (TJm)

The metamorphic rocks present along the easterly slopes of Gregory Canyon form a north-south-trending belt, or screen, of older rock that was intruded (i.e., the action of forcing magma between pre-existing rocks) by magma that formed intrusive rocks (Exhibit 4.2-1). Specifically, the magma that crystallized into the tonalite intruded and intermingled with the metamorphic rock that now forms the western boundary of this screen. Both of these units were subsequently intruded by the magma that crystallized into the leucogranodiorite to form the eastern boundary.

The metamorphic rock screen includes amphibolites and metavolcanic rocks that locally exhibit some migmatitic[8] structure that resembles gneissic banding. The rocks are generally dark blueish gray, hard, and only slightly weathered[9] with aphanitic to porphyroblastic[10] textures. Relict[11] porphyritic textures suggest a volcanic protolith[12] for some of the units.

Larsen (1948) correlated these metamorphic rocks with the Bedford Canyon Formation (a sequence of mildly metamorphosed sedimentary rocks represented by deformed slates, schists, quartzites and localized occurrences of marble), which is widespread in the Santa Ana Mountains. At Gregory Canyon, however, there are no outcrops of slates, quartzites or marbles, and there is a preponderance of metavolcanic rocks. It seems more reasonable to correlate the Gregory Canyon sequence with the Jurassic Santiago Peak volcanics, a unit composed of metavolcanic and metasedimentary rocks exposed elsewhere in San Diego County.

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8 Migmatitic texture is the name given to alternating dark and light colored bands in a metamorphic rock. The bands are formed in response to partial melting of the rock as it came in contact with magma. In contrast, gneissic banding is formed by segregation of dark and light colored minerals in the absence of partial melting or mixing with a magma. The dark and light bands in these two cases may look alike, but one forms only adjacent to a magma intrusion, whereas the other may be found over a large area.

9 Weathering is the collective name given to processes by which rocky materials exposed to atmospheric agents, at or near the earth's surface, change in color, texture, composition, firmness, or form. In intrusive rocks, the most common effects of weathering are the transformation of feldspar and plagioclase to clay, and the oxidation of minerals that contain iron. The weathered rocks are softer that the pristine rocks.

10 A rock is said to have aphanitic texture when the crystals that form it are too small to be observed by the naked eye. In contrast, the term porphyroblastic texture is applied to metamorphic rocks when a few large crystals, easily recognized by the naked eye, are set in a finely crystalline matrix. The same kind of texture in a volcanic rock is called porphyritic.

11 Relict texture is a "leftover" or remnant texture of the original. Recognition of a relict texture allows geologists to determine what a rock was before it underwent metamorphism.

12 A protolith is a parent rock from which a given metamorphic rock was formed by metamorphism.

Of the 196 acres of the proposed landfill footprint, 12 acres along the eastern side encroach over the outcrop of metamorphic rocks.

Tonalite (Kbt)

The tonalite[13] that underlies the western slope and the central portion of Gregory Canyon is an extensive rock unit in the area (Exhibit 4.2-1). Larsen (1948) referred to it as the Bonsall Tonalite. The tonalite is a dark gray, phaneritic rock, with medium- to coarse-crystallinity that varies in composition from tonalite to gabbro. Other common variations noted in the tonalite are the locally veined and streaked appearance and the migmatitic fabric that is observed near the contact with the metamorphic rocks. The rock is also characterized by rare inclusions of the metamorphic rocks, and by numerous leucogranodiorite dikes that include fine-grained aplites[14] and coarse-grained pegmatites.[15] The tonalite comes in contact with the metamorphic rock along the easterly side slopes of Gregory Canyon, although the contact is typically covered by colluvium or obscured by surficial soils. Based on its map position, as inferred from isolated outcrops of both rock types, the contact appears to dip to the east at angles of 20 to 25 degrees. The tonalite is moderately to intensely weathered—in most outcrops, although small cores of only slightly weathered tonalite do form boulder knobs on the western flank of Gregory Canyon. Moderately weathered tonalite still preserves its phaneritic texture, but is less cohesive than the pristine rock, with the constituent minerals slightly altered to oxides and clays, particularly along the edges. The intensely weathered tonalite is oxidized throughout and has a granular texture that only vaguely reflects the original phaneritic texture. The constituent minerals are partially altered to oxides and clays, and disaggregate easily under pressure. The depth of weathering, as determined in exploratory drilling by Geraghty & Miller (1990), ranges between 65 feet (GMP-3) and 95 feet (GMW-2) (Exhibit 4.2-2).

Geraghty and Miller (1990) reported the results of two seismic refraction traverses across the tonalite, and concluded that at depths shallower than 30 feet the seismic wave velocity in weathered tonalite was approximately 3,000 feet per second (ft/sec[16]). At depths greater than 30 feet, seismic wave velocity increased to between 11,000 and 17,000 ft/sec. In general, excavation of materials with seismic velocities greater than 7,000 to 11,000 ft/sec requires blasting.

[13] Tonalites and gabbros are types of intrusive rocks, distinguished by the fact that they contain the mineral calcium plagioclase. This type of rock generally has phaneritic textures, which means that the crystals that form the rock can be distinguished by the naked eye.

[14] An aplite is a type of intrusive rock characterized by light color, abundance of quartz and potassium feldspar, and a fine granular texture that resembles the texture of sugar. Aplite is generally found forming dikes.

[15] A pegmatite is a type of intrusive rock with exceptionally coarse texture, characterized by large interlocking crystals of quartz and potassium feldspar. Pegmatite is generally found forming dikes.

[16] The velocity at which a seismic or elastic wave moves through a rock is a function of its firmness, which in turn is a function of the degree of weathering.

Leucogranodiorite (Kglg)

The leucogranodiorite map unit is a light-colored, biotite-bearing granodiorite[17] that forms the prominent mountain flanking the eastern side of Gregory Canyon (Exhibit 4.2-1). Although this prominent mountain is referred to as Gregory Mountain, Larsen (1948) referred to it as Indian Mountain and to the granodiorite as the Indian Mountain Leucogranodiorite. In hand specimen, the rock has medium- to coarse-crystallinity, is light gray to buff, and has less than five percent dark minerals (biotite and iron-titanium oxides). Besides forming the core of Gregory Mountain, the leucogranodiorite also forms dikes that cut older units and vary in thickness from less than an inch up to five feet. The degree of weathering of the leucogranodiorite is generally slight, as can be inferred from the bold outcrops of Gregory Mountain. The hardness and coherence of these rocks generally makes them un-rippable, no grading is planned in the outcrop area of this unit. In contrast, leucogranodiorite dikes vary in degree of weathering from low to moderate, and should offer no significant resistance to ripping. Moderately weathered dikes are pervasively oxidized and have "cloudy" feldspars, but still preserve their phaneritic texture.

The main body of the leucogranodiorite is in intrusive contact with the metamorphic screen midway along the easterly slope of Gregory Canyon. The contact zone is generally buried under talus,[18] but is narrow and abrupt where it can be observed. Based on its map position, as inferred from the abrupt change in topography, the contact is nearly vertical.

4.2.1.4 Regional Structural Geology

The tectonic[19] regime of the region has changed significantly between the time of emplacement of the intrusions of the Bonsall Tonalite and the Indian Mountain Leucogranodiorite and the present. During the Mesozoic, a subduction zone[20] was active off the coast of California, which led to magma generation and intrusion to form these units.

Tectonic conditions changed during the Cenozoic, when subduction ceased, and transform faulting[21] began on what is now identified as the San Andreas fault system (i.e., the underthrust of the Pacific plate was replaced by lateral shear between the plates). Horizontal motion started between 25 and 20 million years ago in the San Diego region (Atwater, 1970), and since then the tectonic "grain" of the Peninsular Ranges province has been dominated by strike-slip faulting

[17] Granodiorites and leucogranodiorites are types of intrusive rocks, distinguished by the fact that they contain the minerals quartz, potassium feldspar, and sodium plagioclase. These types of rocks generally have phaneritic textures, which means that all the crystals that form the rock can be distinguished by the naked eye.

[18] Talus is the name given to the accumulation of debris at the foot of a cliff.

[19] Tectonics is the name given to the processes that result in mountain building, folding of rocks, or faulting of rocks.

[20] Subduction is the process through which a portion of the Earth’s crust, a plate, is forced under another plate. The Southern California batholith (of which the tonalite and leucogranodiorite are a part) formed when the Pacific plate was forced under the North American plate during the Mesozoic. This type of plate boundary is called a convergent boundary.

[21] Transform faulting occurs when one plate rubs against the other, rather than separating along an oceanic ridge or converging along a subduction zone. The San Andreas fault is the current-day transform margin between the Pacific plate and the North American plate.

along northwest-trending faults like the San Andreas, San Jacinto, Elsinore, and Rose Canyon faults.

The Elsinore fault zone runs approximately 6 miles northeast of Gregory Canyon, and is the closest of these large fault systems to the site. Like the rest of the mentioned faults, the Elsinore fault zone is the result of the right-lateral strike-slip motion between the North American and Pacific plates, although the individual fault strands within the Elsinore fault zone may have strike-slip, normal, or thrust fault motions[22] as a result of complex local geometries (Lamar and Rockwell, 1986). The northwest-trending fabric of the fault zone also results in distinctive structural features, including large-scale structural depressions like the Elsinore Trough, and structural highs such as the Agua Tibia Mountains.

Of more immediate interest to the structural setting of Gregory Canyon is the fact that the "block" between the Elsinore fault zone to the northeast and the Rose Canyon fault zone to the southwest is under a shear stress regime (Exhibit 4.2-3). In effect, the area between both fault zones is being "wrenched" clockwise by the relative motion along these faults. Under these conditions, north-oriented extensional fractures would form, as shown in the stress diagram of Exhibit 4.2-3. This is the most likely explanation for the predominance of north-striking fractures on the site, and for the dominant orientation of topographic lineaments in the region.

4.2.1.5 Local Structural Geology

<u>Lineaments</u>[23]

In regard to lineaments, GLA (1997) inspected aerial photographs of the area, to identify potential structural discontinuities at or near the proposed site, and concluded that there were no regional, through-going discontinuities. Likewise, geologic mapping of the site did not disclose the existence of major faults, although thin shear zones[24] of limited lateral extent were mapped. Some of these shear zones have been annealed[25] by granitic dikes, which demonstrates that they are Mesozoic in age. The complete lineament analysis is presented in Appendix F.

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[22] Faults are classified in three types according to the tectonic stress that formed them. Faults formed by the lateral shear of one rock mass against another are called strike-slip faults. Faults formed under an extensional tectonic regime, and showing surficial displacements, are called normal faults. Finally, those formed under a compressional regime, and showing surficial displacements, are called reverse faults.

[23] Lineament is the name given to a geographic linear feature of regional extent. Lineaments can be topographic features (e.g., linear valleys, linear scarps) or geologic features (e.g., the contact between two rock types that have different erosion patterns).

[24] A shear zone is a tabular zone of rock that has been crushed and cracked by many parallel fractures due to shear strain.

[25] Annealing refers to the filling of fractures by mineral matter.

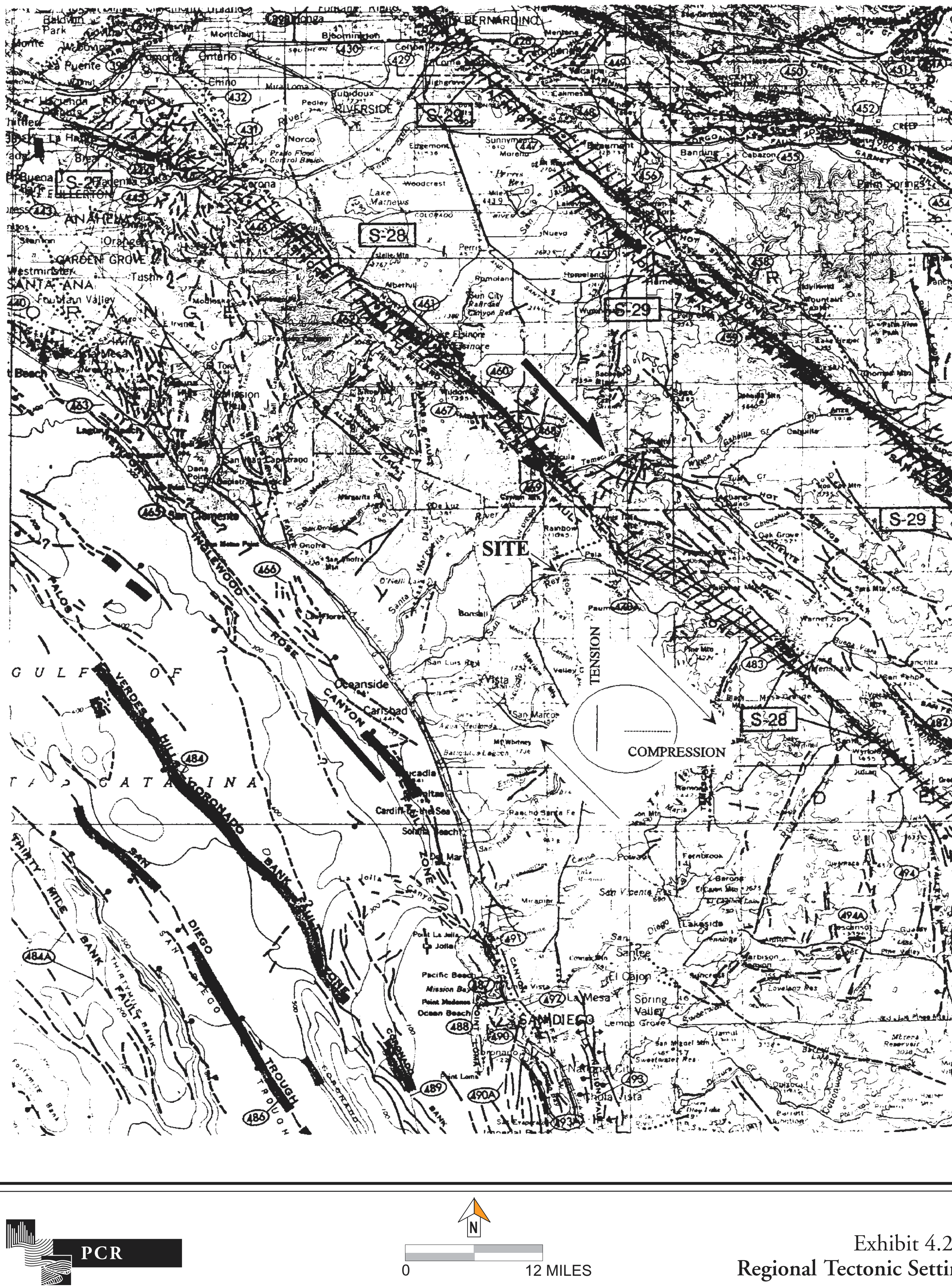

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Exhibit 4.2-3
Regional Tectonic Setting

Sources: Modified from: Jennings, 1994; GeoLogic Associates, 1998; David Evans and Associates, Inc., 1999; PCR Services Corporation, 1999

Discontinuities at Outcrop Level

Joints, dikes and other structural discontinuities are common in the rocks that form the substrate of the canyon. Table 4.2-1 indicates the main orientations of discontinuity, which are consistent with the overall tectonic stress regime of the area, as described in Section 4.2.1.

TABLE 4.2-1
ORIENTATIONS OF MAIN DISCONTINUITIES

	DIP DIRECTION [a]	DIP ANGLE [b]
Direction 1	270º	65º
Direction 2	90º	80º
Direction 3	255º	60º
Direction 4	330º	65º
Direction 5	360º	45º

º = degree
[a] Dip direction is the direction, with respect to magnetic north, of the line of maximum inclination of a tilted plane.
[b] Dip angle is the vertical angle between an imaginary horizontal plane and the tilted plane of interest.
Source: GeoLogic Associates, 1997

4.2.1.6 Soil Resources and Engineering Properties

Exhibit 4.2-4 shows the distribution and variability of surficial soil types in the site area (USDA, SCS 1963). The soils on the site are derived from the alluvium and colluvium, or from in-place weathering of the bedrock (residual soils). As shown, acid igneous rock (AcG) is generally exposed along the easterly slope of the site and soil, if present, generally consists of silty coarse sand. Since this soil type is generally composed of large boulders and rock outcrops, it is likely to have a high runoff potential. Erodibility will vary with soil development but is likely to be high to very high.

Two upland residual soils, Las Posas stony fine sandy loam (LrG) and Cieneba coarse sandy loam (ClG2), are exposed on the westerly slope of the canyon. Runoff in these areas can be rapid to very rapid, and the erosion potential can be high to very high. Sandy loam of the Fallbrook series (FaD2) has been mapped at the north end of the canyon. The runoff in this area is medium, and the erosion hazard moderate.

Cieneba very coarse sandy loam (CmrG) is mapped on the steeper slopes of the site, primarily north of SR 76 (Exhibit 4.2-4). Runoff in this area is rapid to very rapid and the erosion hazard is high to very high. The Tujunga sand (TuB) is exposed below these slopes and is characterized by very slow to slow runoff and only a slight erosion hazard. Riverwash (Rm) is exposed in the San Luis Rey River stream channel and it is typically composed of sandy, gravelly and cobbly materials. Thin slivers of Visalia sandy loam (VaA and VaD) are mapped on the southwest portion of the site. The VaA soil occurs on nearly level terrain, while the VaD soil occurs on moderate slopes. As a result, runoff for the nearly flat-lying VaA soils is very slow and the erosion hazard is slight. The steeper VaD runoff potential is medium with a moderate erosion hazard.

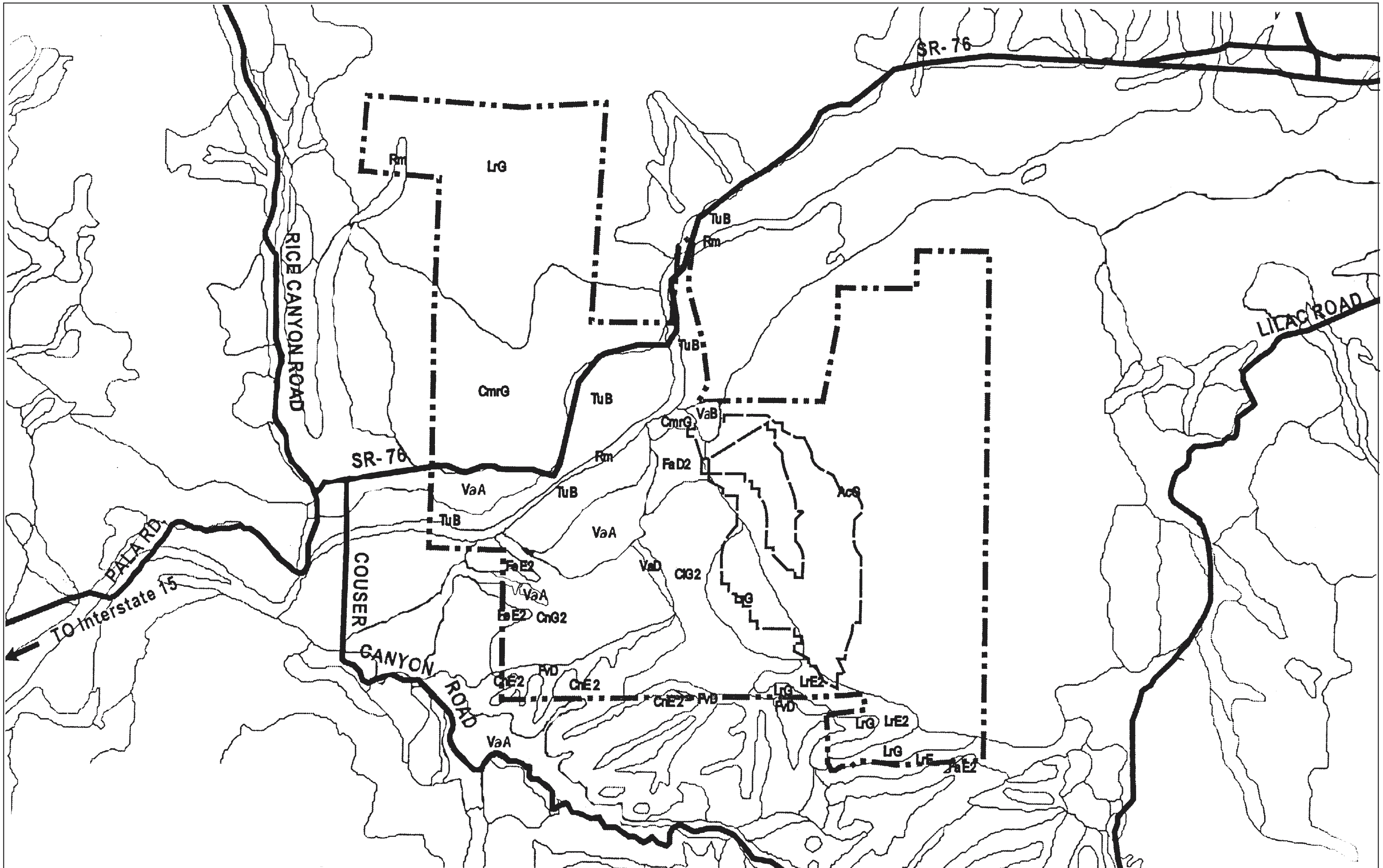

Exhibit 4.2-4
Surficial Soil Types Within the Project Site

Sources: GeoLogic Associates, 1998; Base Map: SANDAG 1998; David Evans and Associates, Inc., 1999; PCR Services Corporation,1999

Laboratory testing was performed by WCC (1995) on soil samples obtained from test pits excavated at the site. A summary of the classification tests performed on the samples shown on Exhibit 4.2-2 is presented in Table 4.2-2. Compaction tests, performed in accordance with ASTM D-1557 on material that was finer than the No. 4 sieve was also completed and a summary of these test results is presented in Table 4.2-3. Strength tests were performed on samples remolded to approximately 90 percent of their maximum dry density and a summary of these test results is presented in Table 4.2-4. The results of consolidation testing are presented in Table 4.2-5. Finally, laboratory permeability tests were also performed and these results are presented in Table 4.2-6.

4.2.1.7 Mineral Resources

San Diego County has a wide variety of mineral resources. Some of these, such as sand, gravel, and dimension stone, are essential to the construction industry and the region's economy. Sand and crushed rock are used as aggregate in Portland cement concrete and asphaltic concrete for construction. Blocks of granite rock (dimension stone) are quarried for decorative rock, monuments, and surface plaster. Of the rock products utilized in San Diego county, concrete-quality sand is in the shortest supply. The major river valleys are by far the most important source of sand in this area. Roughly two-thirds of available sand is in the San Luis Rey River. The San Luis Rey River through the project site is designated as a Mineral Resource Zone-2 (MRZ-2) by the California Department of Conservation, Division of Mines and Geology. The riverbed contains deposits of sand and gravel. The MRZ-2 zone is intended to preserve valuable mineral resources. However, it does not permit extraction of these resources without a major use permit. The MRZ-2 designation, which is an area containing potentially significant mineral resources, is confined to the riverbed.

The site is located 2.5 miles southwest of the Pala pegmatite district, which is a widely known source of gems (e.g., tourmaline, beryl and spodumene) and lithium minerals.

The district has an area of about 13 square miles and is underlain by granodiorite, tonalite, and gabbro of the Southern California batholith.[26] The pegmatites themselves are most abundant in the gabbroic rocks, which are known collectively as the San Marcos gabbro, but are also present in reduced amounts in the granodiorites and tonalites. Most of the pegmatite occurs as tabular masses that trend north to north-northwest, and dip gently to moderately westward. They may have strike lengths of as much as a mile, and range from thin stringers (e.g., a small crack filling) to large dikes with bulges nearly 100 feet thick.

The bedrock substrate of Gregory Canyon is formed by rocks similar to those of the Pala district, and pegmatite dikes, albeit rare, have been identified in the course of geologic mapping. There is, therefore, a small probability that lithium minerals might be found in the bedrock of the site. However, the probability is regarded as small, because 100 years of mineral exploration in and around the Pala district have not yielded mineral prospects in or near Gregory Canyon.

[26] Batholith is the name given to an intrusion or number of intrusions that has more than 40 square miles of exposure. The Southern California batholith extends from the Los Angeles area, through San Diego County, down the entire length of the Baja California Peninsula in Mexico. It was formed during the Mesozoic, in the period between 200 and 65 million years ago.

TABLE 4.2-2
SUMMARY OF SOIL CLASSIFICATION TESTS

SUMMARY OF SOIL CLASSIFICATION TESTS SAMPLE NUMBER	SAMPLE DEPTH (ft)	SOIL CLASSIFICATION	GEOLOGIC UNIT	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	NO. 200 SIEVE (%)
TP1-2	2-3	Fine sandy lean clay (CL)	Older colluvium	10	30	17	53
TP2-1	0-1	Clayey fine sand (SC)	Topsoil	3	28	15	38
TP2-4	3-5	Clayey fine sand (SC)	Highly weathered metamorphic rock		29	15	44
TP3-1	1-2	Fine sandy lean clay (CL)	Residual soil		32	18	56
TP4-2	3-4	Fine sandy lean clay (CL)	Residual soil	8	24	10	61
TP4-3	6-7	Silty medium to coarse sand (SM)	Highly weathered tonalite				
TP4-4	1-4	Fine sandy lean clay (CL)	Residual soil		23	10	58
TP4-5	4-7	Silty sand (SM)	Highly weathered tonalite			NP	18
TP5-1	2	Fine sandy lean clay (CL)	Residual soil	10	39	26	56
TP5-2	4-5	Silty sand with clay (SM)	Highly weathered metamorphic rock				
TP5-3	3-5	Silty fine sand (SM)	Highly weathered metamorphic rock			NP	42
TP6-1	0-1	Silty fine sand (SM)	Topsoil	2		NP	33
TP6-2	2.2-5	Silty fine sand (SM)	Colluvium	6		NP	31
TP7-2	2.2-5	Clayey fine sand (SC)	Older colluvium	8	28	12	31
TP8-1	1-2	Silty fine sand (SM)	Colluvium			NP	26
TP9-1	1-3	Silty fine sand (SM)	Colluvium			NP	43
TP9-2	3-6	Silty fine sand (SM)	Highly weathered metamorphic rock			NP	21
TP10-1	1-4	Silty fine sand (SM)	Colluvium			NP	34
TP10-2	4-7	Silty fine sand (SM)	Colluvium			NP	32
Source: Woodward-Clyde, 1995							

TABLE 4.2-3
SUMMARY OF LABORATORY COMPACTION TEST RESULTS

SAMPLE NUMBER	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
TP2-4	Clayey fine sand (SC)	129.5	10.5
TP3-1	Fine sandy lean clay (CL)	128.0	10.5
TP4-4	Fine sandy lean clay (CL)	131.0	10.0
TP4-5	Silty sand (SM)	131.0	8.5
TP5-3	Silty fine sand (SM)	121.0	12.5
TP8-1	Silty fine sand (SM)	129.5	8.5
TP9-1	Silty fine sand (SM)	132.0	9.5
TP9-2	Silty fine sand (SM)	127.0	10.0
TP10-1	Silty fine sand (SM)	133.5	8.5
TP10-2	Silty fine sand (SM)	133.0	9.0
Source: Woodward-Clyde, 1995			

4.2.1.8 Geologic Hazards Due to Surficial Processes

Landslides

The potential for landsliding was evaluated by WCC (1995) based on review of stereo aerial photographs and field reconnaissance study and geologic or geomorphic features characteristic of landslides were not observed.

Rockfalls

Rockfalls are abrupt movements of independent blocks of rock that become detached from steep slopes. Falling rocks can reach the base of a slope by free-falling, bouncing, rolling down the slope surface, or by some combination of the above. There is clear evidence that rockfalls have occurred at the site during mass wasting of Gregory Mountain.

Debris Flows

Earth, mud, and debris flows form when a mass of unconsolidated sediment is mobilized by sudden ground vibration (e.g., an earthquake) or by a sudden increase in weight and pore water pressure (e.g., after soaking of the soil by heavy rains). The initial movement of a flow is enhanced by steep topography and deforestation, but once mobilized, flows can spread over gently sloping terrain.

Debris flows cannot be forecasted, but the susceptibility for formation of debris flows on any given site can be estimated by looking for evidence of previous flow events. GLA (1998) reviewed aerial photographs of the site, and concluded that there is a deposit of poorly sorted colluvium that could have been formed as a quarternary debris flow deposit (Qd(?) (Exhibit 4.2-5). The deposit forms a landform with a rough lobate (e.g., rounded) shape and comparatively steep boundaries, but lacks levees or pressure ridges, and so could also have been formed by erosion of an older colluvial fan. It appears that the one formation within Basin 1 may

TABLE 4.2-4
SUMMARY OF STRENGTH TEST RESULTS

SAMPLE NUMBER	SAMPLE DEPTH (FT)	SOIL CLASSIFICATION	GEOLOGIC UNIT	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	NORMAL STRESS	COHESION (PSF)	FRICTION ANGLE (DEGREES)
TP2-4	3-5	Clayey fine sand (SC)	Highly weathered granite	11	116	Low	440	28
TP3-1	1-2	Fine sandy lean clay (CL)	Residual soil	11	114	Low	240	28
TP4-5	4-7	Silty sand (SM)	Highly weathered tonalite	9	119	Low	500	47
TP5-3	3-5	Silty fine sand (SM)	Highly weathered tonalite	13	108	Low	720	32
TP5-3	3-5	Silty fine sand (SM)	Highly weathered tonalite	13	109	High	1500	30
TP8-1	1-2	Silty fine sand (SM)	Colluvium	9	117	Low	650	39
TP9-1	1-3	Silty fine sand (SM)	Colluvium	10	121	Low	770	30
TP9-2	3-6	Silty fine sand (SM)	Highly weathered granite	10	114	Low	660	41
TP10-1	1-4	Silty fine sand (SM)	Colluvium	9	120	Low	680	33
TP10-1	1-4	Silty fine sand (SM)	Colluvium	9	120	High	1120	33
TP10-2	4-7	Silty fine sand (SM)	Older colluvium	9	120	Low	610	33
TP10-2	4-7	Silty fine sand (SM)	Older colluvium	9	120	High	150	35
Source: Woodward-Clyde, 1995								

TABLE 4.2-5
SUMMARY OF CONSOLIDATION TESTS

SAMPLE NUMBER	DEPTH (FT)	SOIL DESCRIPTION	LIQUID LIMIT	PLASTICITY LIMIT	VIRGIN COMPRESSION INDEX
TP2-4	3 to 5	Clayey sand (SC)	29	14	12.4
TP4-4	1 to 5	Fine sandy lean clay (CL)	23	13	16.8
TP9-1	1 to 3	Silty sand (SM)	--	NP	11.3

NP = Non-plastic
All samples inundated with water at 2 ksf.
Source: Woodward-Clyde, 1995

TABLE 4.2-6
SUMMARY OF LABORATORY PERMEABILITY TEST RESULTS

SAMPLE NUMBER	SOIL DESCRIPTION	DRY DENSITY (PCF)	NO. 200 SIEVE (%)	HYDRAULIC CONDUCTIVITY[a] (CM/SEC)
TP2-4	Clayey fine sand (SC)	116	44	3.7×10^{-6}
TP3-1	Fine sandy lean clay (CL)	115	56	3.8×10^{-7}
TP4-4	Fine sandy lean clay (CL)	118	58	7.3×10^{-7}
TP4-5	Silty sand (SM)	118	18	3.5×10^{-4}
TP5-3	Silty fine sand (SM)	109	42	1.1×10^{-6}
TP8-1	Silty fine sand (SM)	116	26	7.4×10^{-7}
TP9-1	Silty fine sand (SM)	119	43	7.3×10^{-7}
TP9-2	Silty fine sand (SM)	114	21	1.6×10^{-4}
TP10-1	Silty fine sand (SM)	120	34	7.8×10^{-6}
TP10-2	Silty fine sand (SM)	120	32	7.6×10^{-6}

[a] Samples remolded to 90 percent relative compaction (maximum density per ASTM D-1557) at optimum moisture content.
Source: Woodward-Clyde, 1995

have been created about 100 or 200 years ago. There is no evidence of recent debris flows and it should be noted that during the heavy storms of the 1990s no debris flows occurred.

4.2.1.9 Geologic Hazards Due to Deep-Seated Processes

<u>Faults</u>

Faulting was evaluated by WCC (1995) for the project site and surrounding area based on a review of geologic literature, large- and small-scale stereo aerial photographs, and field reconnaissance data. GLA (1997) augmented the lineament analysis. The closest mapped faults to the site are an east-northeast-trending fault first located by Jahns and Wright (1951), and a shear zone described by WCC (1995) (Exhibit 4.2-6). The Jahns and Wright (1951) fault is the only nearby fault depicted in the 1994 Fault Activity Map of California (Jennings, 1994), and it shows no evidence for Cenozoic displacement (e.g., inactive fault).

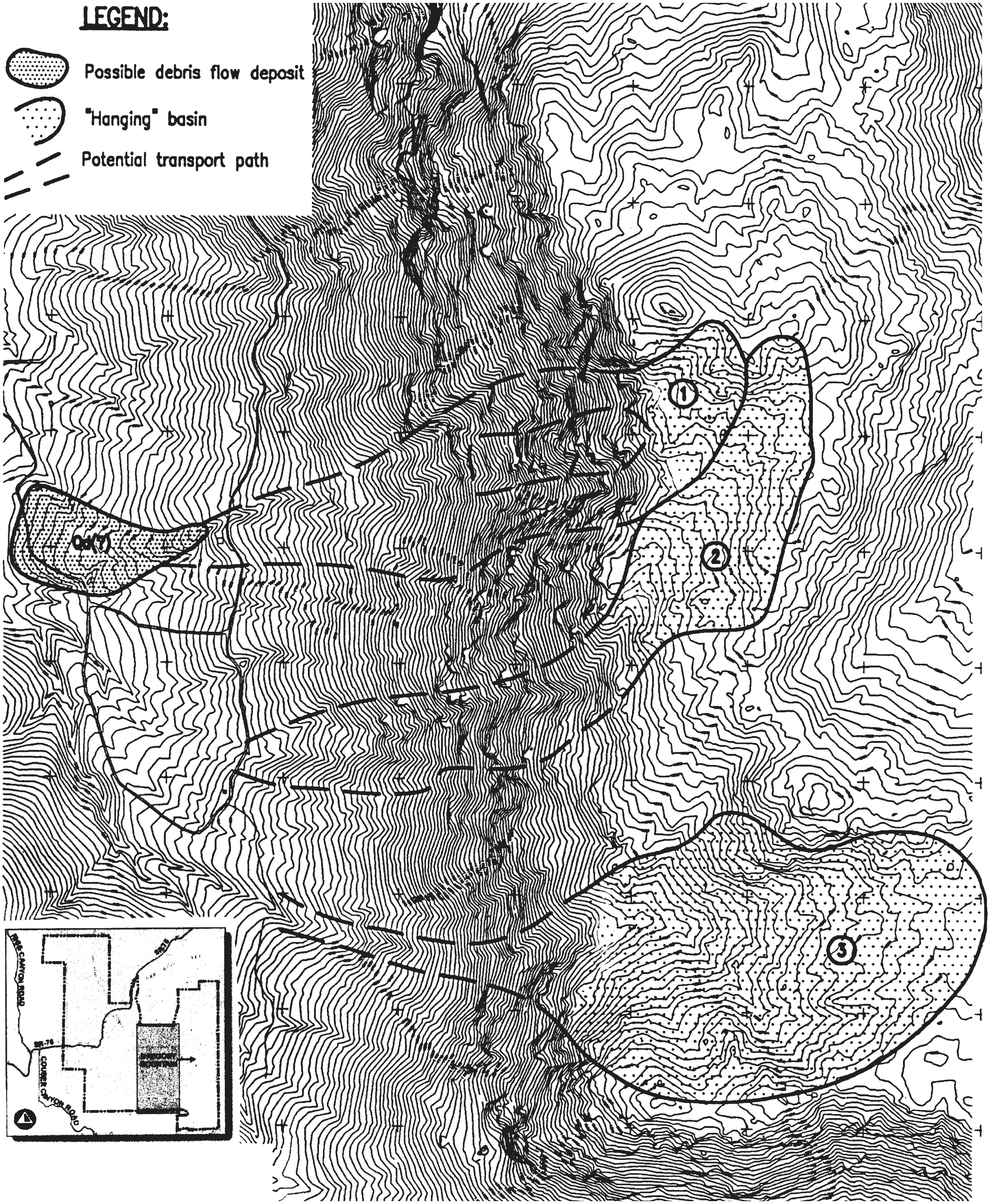

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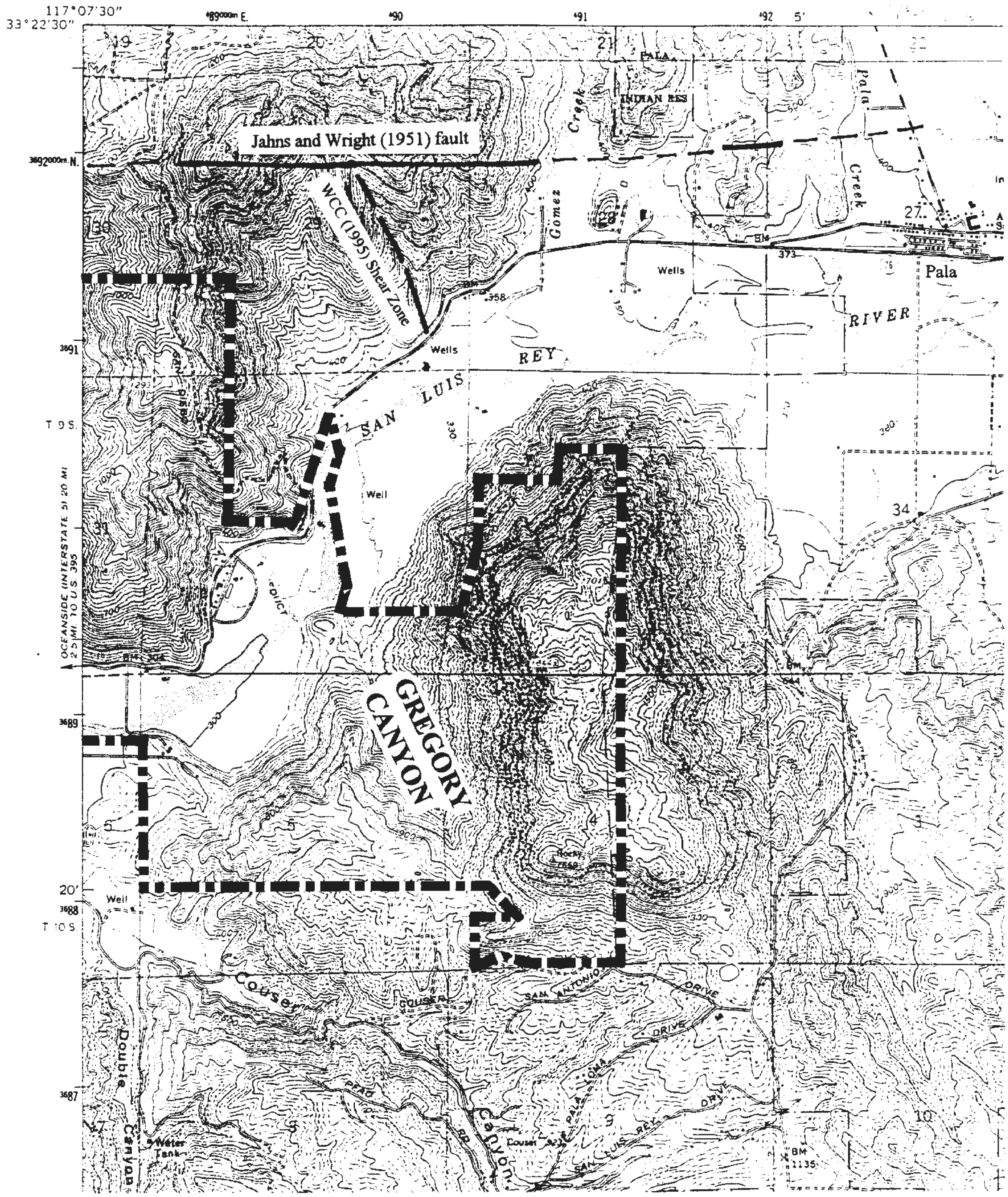
Exhibit 4.2-5
Potential Sources of Debris Flow Hazard

Sources: GeoLogic Associates, 1998; David Evans and Associates, Inc., 1999; PCR Services Corporation, 1999



0 4000 FEET

Sources: USGS, 1988, 7.5 Minute Pala Quadrangles; GeoLogic Associates, 1997; David Evans and Associates, Inc., 1999; PCR Services Corporation, 1999

Exhibit 4.2-6

Location of Nearby Faults

With respect to the potential shear zone located across SR 76, WCC (1995) noted that there is no evidence to support continuity of the high-angle shear feature (such as lineations or similar exposures) along its general strike to the north or south. From this they inferred it to be a localized feature. GLA inspected this outcrop, and concludes that the so-called shear zone is actually a steep planar contact between metamorphic rocks and hydrothermally altered[27] gabbro. The gabbro is brecciated (i.e., the rock is not homogeneous, but rather it is formed by an agglomeration of angular blocks), but the fragments do not show tectonic shearing, alignment, or fault gauge between them. A couple of hundred feet east of the contact the rock becomes progressively less brecciated and hydrothermally altered.

The 200-foot zone of brecciated gabbro does not have the characteristic features of a fault zone since such a thick "fault zone" would be indicative of a major fault, shearing should be pervasive. In fact, there are no prominent shear planes through this portion of the outcrop. In addition, careful inspection of the ravines to the north of the outcrop did not disclose continuation of the breccia, so GLA concludes that it has the shape of a vertical chimney, rather than a planar feature. The limited extent of the breccia zone in the strike direction is uncharacteristic of a major fault zone, as such structures normally extend for several miles. In contrast, intrusive breccia chimneys or pipes are a common feature in shallow plutons (e.g., Norton and Cathles, 1974), and characteristically show the effects of hydrothermal alteration.

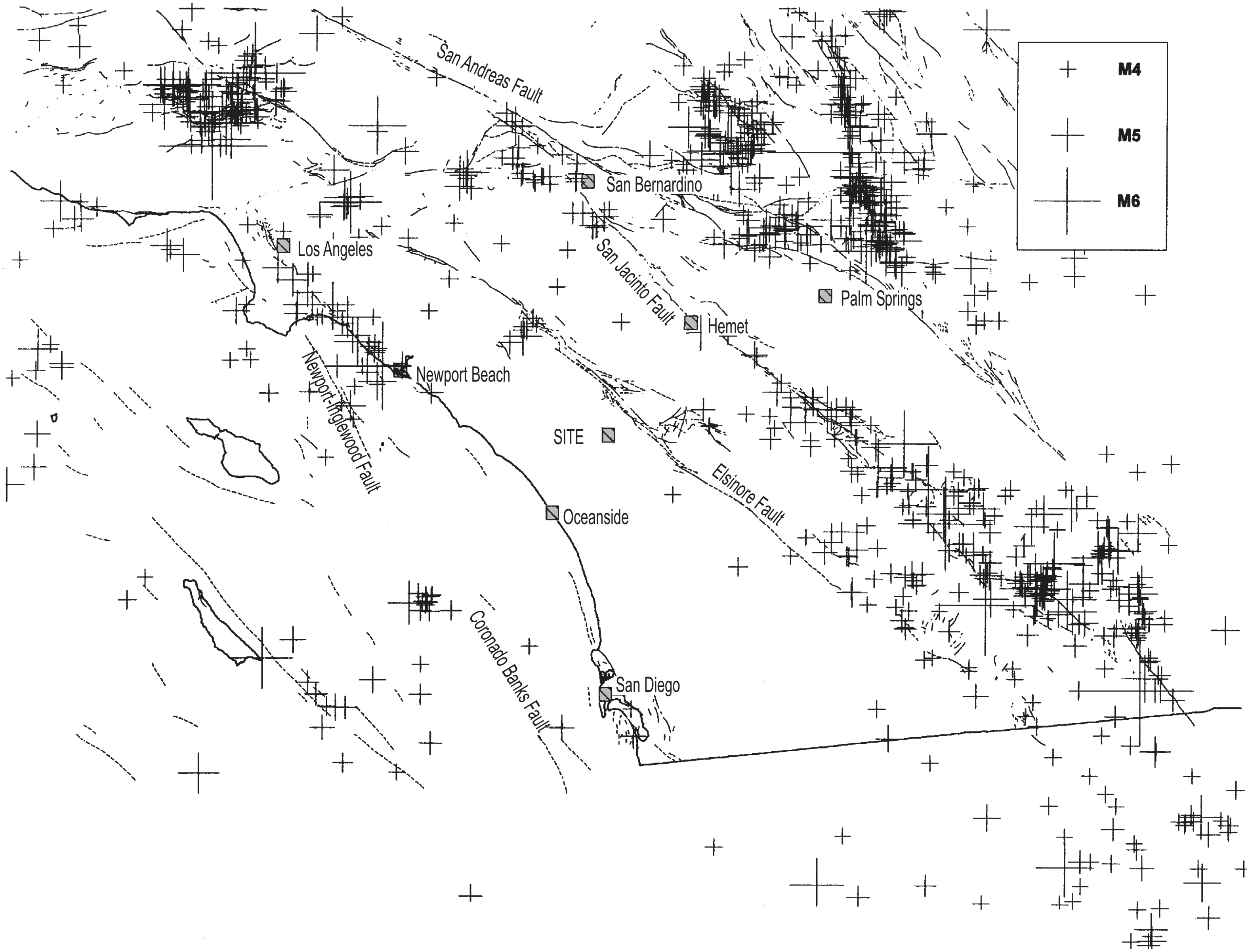
To confirm this interpretation GLA made a careful inspection of the north flank of Gregory Mountain, where the contact would be reasonably expected to project if it were an extensive planar feature. This inspection identified only non-brecciated tonalite/gabbro along the northern flank of Gregory Mountain, thus confirming that the -gabbroic breccia does not extend across the San Luis Rey River. Finally, GLA performed a careful inspection of the outcrops that the roadcuts of SR 76 have created. These outcrops are in direct alignment with Gregory Canyon itself, and expose a continuous non-brecciated and non-faulted section of weathered tonalite/gabbro. This continuous section is further evidence that a major fault zone does not extend along the axis of Gregory Canyon.

Several active faults[28] exist within 60 miles of the project. These include the San Andreas, San Jacinto, and Elsinore fault zones, as well as the offshore portions of the Rose Canyon fault zone (Exhibit 4.2-7). All these fault zones have an overall trend to the northwest, and right-lateral, strike-slip senses of movement. At its closest approach to the site, the San Andreas fault is located 54 miles to the northeast, the San Jacinto fault is 30 miles to the northeast, the Rose Canyon fault is 25 miles to the southwest, and the Elsinore fault is located 6 miles to the northeast.

<u>San Andreas fault</u>. Based on the large number of historic earthquakes generated along the San Andreas fault, Wallace (1970) estimated that in any given year there is a 20 percent probability

[27] Hydrothermal alteration occurs when hot water moves through a rock, reacting with the minerals present to form minerals such as clay.

[28] For convenience, geologists consider a fault as potentially active if it has had movement sometime during the last 11,000 years.



PCR

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NOT TO SCALE

Sources: California Institute of Technology; David Evans and Associates, Inc., 1999; PCR Services Corporation, 1999

Exhibit 4.2-7
Epicenter of Earthquakes from 1932-1997

that an earthquake of magnitude[29] 6.0 would occur somewhere along the 600 miles of the San Andreas fault, and that there is a one percent probability that a magnitude 8.0 earthquake would be generated in any given year. Trenching and geochronometric studies by Sieh (1978) in Pallett Creek (e.g., located approximately 50 kilometers northeast of Los Angeles) suggest annual probabilities of 0.5 to 0.3 percent probability for magnitude 8.0 earthquakes.

For the purpose of being conservative in evaluating seismic hazard, the California Division of Mines and Geology (CDMG) has estimated the Maximum Probable Earthquake[30] (MPE) along the San Andreas-Coachella and San Andreas-San Bernardino segments of the fault at magnitudes 7.1 and 7.3, respectively, using the regressions of Wells and Coppersmith (1994). GLA (1998) used a value of M8.0 for the seismic analysis of this fault.

<u>San Jacinto/Casa Loma fault</u>. The San Jacinto/Casa Loma fault extends more than 120 miles from northwest of El Centro to northwest of San Bernardino. This fault has been the source of numerous micro-seismic events in modern history including a M6.6 event that occurred in November 1987 at Superstition Hill, just north of the international border. Another recent rupture zone, extends across Borrego Mountain (San Diego County) and was the source of a M6.8 seismic event in 1968. The Anza section of the fault extends northwestward from Borrego Mountain section and generated a M6.8 event in 1918. Finally, there was an 1890 event along the San Bernardino Valley section of the fault in which surface lateral displacement was estimated at 1.4 ± 0.4m.

For the purpose of being conservative in evaluating seismic hazard, the CDMG has estimated the MPE magnitude along both the Elmore Ranch and Superstition Mountain segments of the San Jacinto/Casa Loma fault at M6.6, using the regressions of Wells and Coppersmith (1994). GLA (1998) used a value of M7.0 for the seismic analysis of this fault.

<u>Elsinore/Whittier fault</u>. The Elsinore/Whittier fault extends 150 miles from the Mexican border to west of the northern edge of the Santa Ana Mountains. The Whittier-North Elsinore fault zone is an oblique fault, with the north block uplifted (hence the existence of the Puente Hills) and displaced to the right. Five earthquakes of magnitude greater than M5 have been generated along this fault during the last 100 years, three of which had epicenters near Lake Elsinore.

For the purpose of being conservative in evaluating seismic hazard, the CDMG has estimated MPE magnitude along the Earthquake Valley and Elsinore-Temecula segments of the Elsinore/Whittier fault at magnitudes 6.5 and 6.8, respectively, using the regressions of Wells and Coppersmith (1994). GLA (1998) used a value of M7.0 for the seismic analysis of this fault.

<u>Rose Canyon/Newport-Inglewood fault</u>. The offshore Rose Canyon/Newport-Inglewood fault zone extends more than 150 miles from the international border to Newport Beach, as a 1,000- to 15,000-foot wide zone of strike-slip faults, folds, and related thrust faults. Of immediate interest

[29] Magnitude is a quantitative measure of the energy released by an earthquake. Moment magnitude is calculated from the seismic moment of the rupture that generates the earthquake by multiplying the rigidity of the rock times the area of faulting times the amount of fault slip.

[30] Maximum Probable Earthquake is the maximum earthquake that is likely to occur during a 100-year interval. It is to be regarded as a probable occurrence, not as an assured event that will occur at a specific time (CDMG, 1975).

for the proposed project is the Del Mar segment, which extends from the latitude of Carlsbad to the latitude of La Jolla.

With respect to historical seismic activity, rupture along the northernmost segment of the fault, the South Los Angeles segment, resulted in the 1933 M6.3 Long Beach earthquake. In addition, where on shore, the fault zone has significant microseismic activity, while offshore seismicity south of Newport Beach decreases by an order of magnitude.

For the purpose of being conservative in evaluating seismic hazard, the CDMG has estimated the MPE magnitude along the Del Mar segment of the Rose Canyon/Newport-Inglewood fault at 6.9, using the regressions of Wells and Coppersmith (1994). GLA (1998) used a value of M7.0 for the seismic analysis of this fault.

Seismic Hazards

The worst-case scenario that can be expected for the site is a near-field[31] MPE of M7.0 along the Elsinore fault, or a far-field[32] MPE of M8.0 along the San Andreas fault. Peak horizontal accelerations, as a fraction of the acceleration of gravity, can be estimated from the regression relations of Boore et al. (1997), Abrahamson and Silva (1997), and Campbell (1997), and are summarized in Table 4.2-7.

TABLE 4.2-7
PEAK HORIZONTAL GROUND ACCELERATIONS

SCENARIO	0.5 PROBABILITY	0.16 PROBABILITY
Elsinore fault M 7.0 earthquake 6 miles from the site	0.26g to 0.38g	0.43g to 0.58g
San Andreas fault M 8.0 earthquake 54 miles from the site	0.06g to 0.09g	0.1g to 0.15g
San Jacinto fault M 7.0 earthquake 30 miles from the site	0.06g to 0.08g	0.1g to 0.14g
Rose Canyon fault M 7.0 earthquake 25 miles from the site	0.08g to 0.1g	0.13g to 0.15g
Source: GeoLogic Associates, 1998		

From these estimates, it appears that the project site area is likely to experience short-period peak horizontal accelerations of up to 0.4g (with a probability of 0.5), and has a small probability of experiencing short-period peak horizontal accelerations as high as 0.6g (probability of 0.16). The quoted probability expresses the likelihood that, in the event of a magnitude 7.0 earthquake along the Elsinore fault, the site will experience an acceleration as high as the values given. This should not be confused with the probability that a magnitude 7 earthquake will actually happen. As stated above, Blake (1993) estimated the long-term recurrence interval for M7 events at 100 years, from which an annual probability of occurrence of 0.01 can be calculated. Following the rules of conditional probability, on an annual basis, the probability that a M7 event will

[31] Near field earthquakes are generally defined as those generated at distances similar to those of the dimensions of the source (typically less than 10 miles).

[32] Far field earthquakes would be those generated at distances that are significantly larger than the dimensions of the source (typically more than 10 miles).

happen and peak ground acceleration at the site will be as large as 0.4g has a probability of 0.005, which is equivalent to a recurrence interval of 200 years. Likewise, the probability that a M7 event will happen and peak ground acceleration at the site will be as large as 0.6g has a probability of 0.0016, which is equivalent to a recurrence interval of 625 years.

In a completely independent way, the National Seismic Hazard Mapping Project of the U.S. Geological Survey (USGS) estimated an annual probability of exceedance of 0.002 for a peak ground acceleration of 0.6g at the town of Pala three miles from the proposed landfill (http://geohazards.cr.usgs.gov/eq). This probabilistic hazard analysis considers contributions from all faults within a 100-mile radius, though naturally the largest contribution is from earthquakes generated along the Elsinore fault.

Probabilistic hazard analysis is not as intuitive as deterministic analysis, but it has the advantage that it takes into consideration many possible scenarios of fault rupture. As shown by the previous comparison between the GLA deterministic analysis and the USGS probabilistic analysis, the results of both methods of analysis are comparable. Engineering analysis for the project has been based on a peak horizontal acceleration for 0.6g. This value was used in the stability and wave propagation analysis, and was incorporated in the liquefaction analysis, as discussed by GLA (1998).

4.2.2 Impact Significance Criteria

Appendix G of the State CEQA Guidelines provides environmental checklist questions. Using these questions, the project may be deemed to have a significant impact on geology and soils and mineral resources if the project would:

- Expose people or structures to potential substantial adverse effects, including the risk of loss, injury, or death involving:
  - Rupture of a known earthquake fault as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area or based on other substantial evidence of a known fault (refer to Division of Mines and Geology Special Publication 42)
  - Strong seismic ground shaking
  - Seismic-related ground failure, including liquefaction
  - Landslides
  - Mudflow/debris flows
- Be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction or collapse
- Be located on expansive soil, as defined in Table 18-1-B of the Uniform Building Code (1994), creating substantial risks to life or property
- Result in the loss of availability of a known mineral resource that would be of value to the region and the residents of the state
- Result in the loss of availability of a locally important mineral resource recovery site delineated on a local general plan, specific plan or other land use plan

4.2.3 POTENTIAL IMPACTS

4.2.3.1 Short-Term (Construction) Impacts

Construction impacts are analyzed for SR 76 road improvements, construction of the access road and bridge, and initial construction of the landfill and associated facilities. The impacts of the construction of the future phases of landfill development are included within the environmental impact analysis for long-term operations.

Soil Resources

Landfill Excavation and Development: The initial construction of the project includes the construction of an access road and bridge; construction of the ancillary facilities, including the scalehouse, maintenance building, and water tank; excavation of a portion of the defined landfill footprint; and installation of the waste containment system (liner system) for Phase I. The leachate collection and removal system, leachate storage tank, and drainage system will also be constructed during the initial liner construction phase. The initial construction period will be approximately nine to twelve months in duration. SR 76 improvements at the access road will occur at this time. The construction period for the bridge crossing the San Luis Rey River will be approximately six months, occurring during the initial construction period. Construction equipment will be brought into the site over the existing river crossing and will remain on the site.

Initial excavation activities within the refuse footprint will involve approximately 1.5 million cubic yards (mcy). During the initial excavation, a portion of the excavated material will be used for engineered fill necessary to construct the ancillary facilities area and a toe buttress, with the remainder of the material being stockpiled in the landfill footprint or Borrow/Stockpile Area A. The borrow/stockpile haul road, connecting Borrow/Stockpile Area A with the landfill footprint will be 20 feet wide and most of the alignment will follow an existing dirt road on the site.

For excavation of the landfill footprint, the colluvial units along the base of the easterly natural slope contain clusters of leucogranodiorite boulders emplaced as talus debris from the cliff-forming outcrops at higher elevations. In addition, dikes of more resistant intrusive rocks and/or unweathered core rock within more weathered zones may also be encountered during excavations for the base cut. The oversized rock will have to be removed either individually by excavation or by blasting during landfill development.

Liner Materials: The results of laboratory permeability testing conducted by WCC indicate that the on-site soils do not meet the permeability requirement for use as liner material (Table 4.2-6). Therefore, clay liner materials or geosynthetic clay liners will be obtained from off-site commercial sources. The most likely source of low permeability liner material is in the Lake Elsinore area. As an alternative for the low permeability liner material, on-site material can be used if it is blended with additional imported bentonite to achieve the desired low-permeability performance criteria. Potential traffic impacts from importation of materials for liner construction has been included in Section 4.5 Traffic and Circulation, within this document.

Erosion or Siltation

Construction activities will loosen soil and rock particles, which could in turn accelerate erosion of these loose materials, and increase siltation at the mouth of Gregory Canyon. Similar impacts could occur in drainages downgradient of the borrow/stockpile areas. See Section 4.4, Surface Hydrology for a complete discussion and analysis of this environmental issue.

Exposure of People or Structures to Natural Geologic Hazards

Landsliding

As described in Section 4.2.1, the site slopes have not experienced significant landsliding in the recent geologic past. Because the natural slopes will be modified by the project the stability of the man-made cut slopes must be analyzed.

The three most common types of cut-slope failures are block-slip, wedge-slip, and circular failures. Block-slip failures are most common in slopes that are underlain by bedrock with distinctive partings (e.g., fractures) that dip in the same direction but at a shallower angle than the cut. Wedge-slip failures occur when the bedrock has two or more partings (e.g., a weathered dike and a joint) with orientations such that their line of intersection dips at a shallow angle in the direction of the cut. Finally, circular failures develop where the substrate is loosely consolidated and comparatively homogeneous.

A stability assessment was performed using a kinematic analysis (Norrish and Wyllie, 1996), to see if movement along one or more of the main discontinuity planes is possible. The kinematic analysis shows that large-scale block-slip movement and wedge-failure are not feasible given the geometry of the dominant directions of discontinuity in Gregory Canyon. All the rocks exposed at Gregory Canyon are compact and cohesive, even when weathered, so a circular failure of the cut slopes is similarly unlikely. As a result, the proposed cut slopes are anticipated to be stable and no significant impacts would occur.

Rockfalls

The east flank of Gregory Canyon is dotted with numerous boulders that may be unstable under seismic or blasting vibration. Erosion near the base of the boulders during rapid runoff may also undermine support of the boulders. This creates the potential for rockfalls that could create a safety risk to workers and equipment and could cause damage to the landfill development.

Falling rocks can reach the base of a slope by free-falling, bouncing, rolling down the slope surface, or by some combination of these modes of transport. A first scenario was calculated by GLA (1998) for elastic bouncing trajectories, which yield the maximum encroachment of a bouncing rock fragment into the footprint of the landfill. The encroachment distance from the edge of refuse was estimated at 300 feet, and the travel time from the top of the profile to its final resting point was estimated at 22 seconds. GLA (1998) calculated a second scenario, incorporating the more realistic condition that some of the kinetic energy of the falling rock fragment would be dampened by impact. The bouncing rock would stop within a few feet after reaching the limit of refuse with an estimated travel time of 23 seconds. The analysis of this scenario indicated that the bouncing trajectories become smaller in length and traveling height as the bouncing rock fragment moves from the medial to the lower reaches of the slope.

A third scenario addressed rolling particles, and suggested that rolling rock fragments could travel as much as 360 feet onto the landfill if unchecked (Exhibit 4.2-8).

All boulders 24 inches in size and greater will be inspected by designated site personnel prior to development of any area of the landfill. In addition, prior to any on-site blasting, a qualified geologist will identify areas of potential rockfall concern. All boulders upslope of the landfill

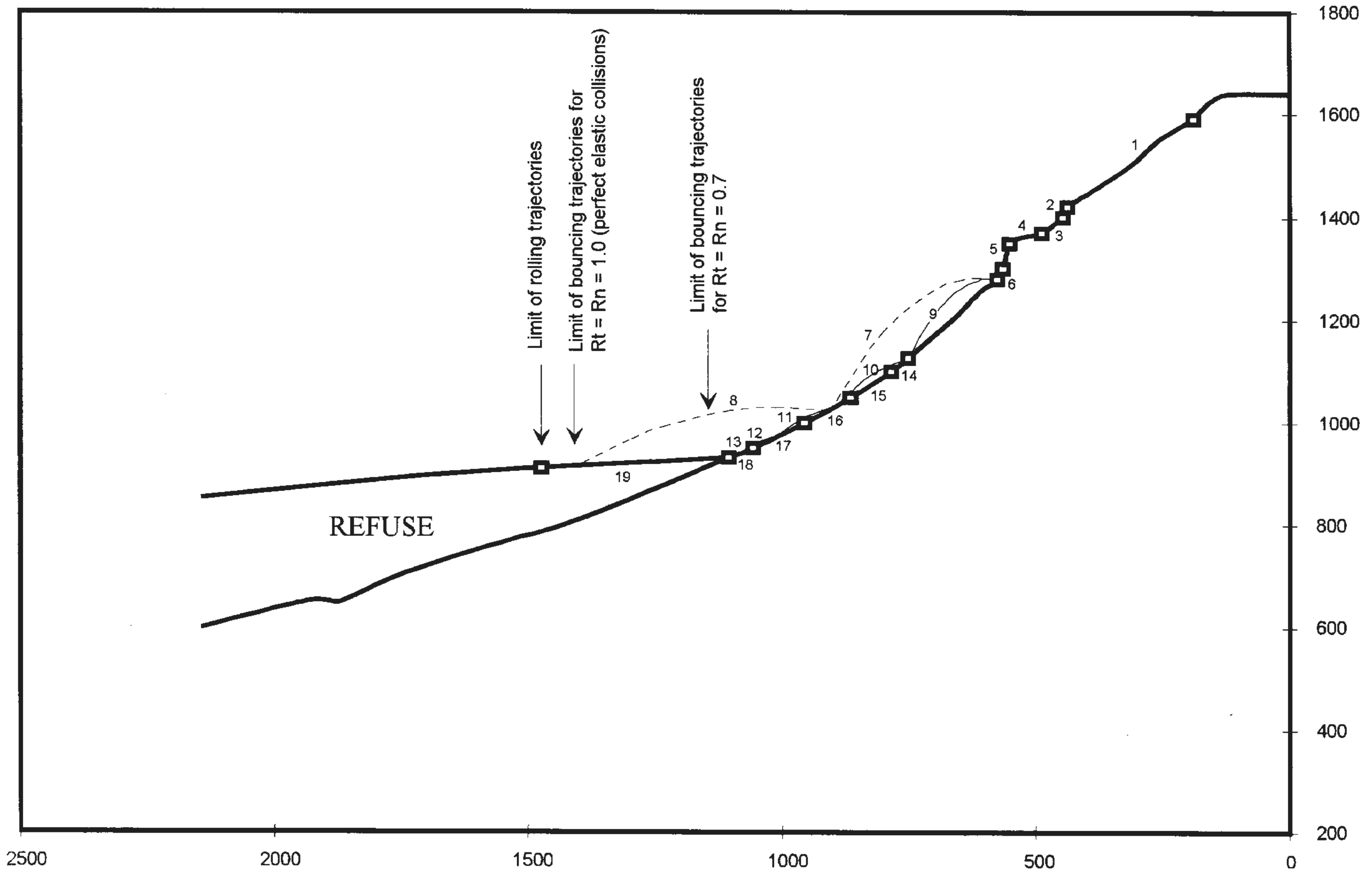


Exhibit 4.2-8
End-Member Trajectories Used in the Rockfall Analysis

Sources: GeoLogic Associates, 1998; David Evans and Associates, Inc., 1999; PCR Services Corporation, 1999

footprint will be left in place unless a boulder is identified that may be insecure. Any removal of boulders will meet the requirements under 8 CCR, Division 1, Chapter 3, Section 1541. Physical force will be applied to the boulders using pry bars or cables and heavy equipment. Any loose boulders that are identified will be wrapped in a netting material and moved with landfill heavy equipment to flatter ground within the refuse footprint area for processing. Larger, heavier boulders will be wrapped in steel netting to reduce the potential for tensile failure of the net. Cables will be attached to the net to apply a constant tension to balance the load and to allow the controlled movement of the boulder down slope in a safe manner. In addition, as the phases of development move up canyon a spotter will be stationed up-gradient of the construction area to observe for falling rocks/boulders. The spotter will provide an early warning of rockslide. In addition, if necessary, rockfall restraining nets could also be used. This approach will ensure that rockfalls would not create a safety risk to workers and equipment, or to the landfill development. No significant rockfall impacts would occur.

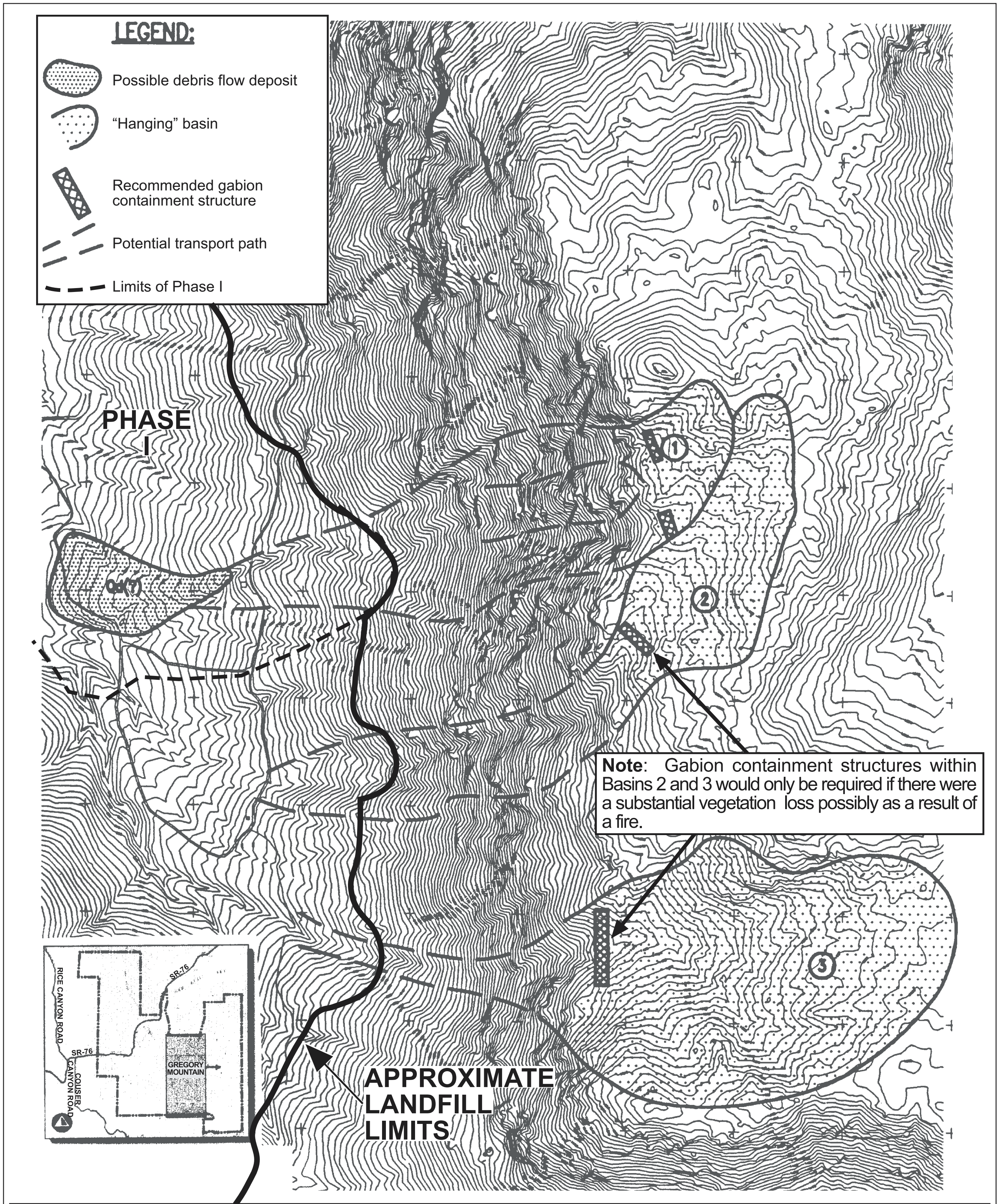
Debris Flows

Potential debris-flow sources are found in the three “hanging” basins[33] that drain the western summit of Gregory Mountain (Exhibit 4.2-9). These basins have gentle slope gradients in their head regions, but drain into the steep western flank of Gregory Mountain. The most effective measure to minimize the impact from a debris flow is the natural development of vegetation within the drainage basins. Basin 1 has a large amount of bare rock exposure, with modest development of vegetation along the centerline of its tributaries. Basins 2 and 3 also have some areas where bare rocks are exposed, but overall appear to be more densely and evenly vegetated than Basin 1. Therefore, based on further analysis for this Final EIR, no diversion structure is needed in Basins 2 and 3 at this time because of the existence of sufficient vegetation. Basin 1 has the potential for a debris flow and, therefore, a diversion structure, such as a gabion dam, may need to be installed to prevent a debris flow. A gabion is a large steel wire mesh basket filled with rock (see Section 3.3.2). The rock-filled baskets can be wired together to create an embankment or retaining walls. A gabion structure obtains most of its stability and resistance to load forces, such as ramming debris flow, from: (a) the mass of the gabion’s rock fill; and (b) the friction and interlocking that develops within a basket’s rockfill and between the rockfill surfaces of the individual gabion units. Structurally, the main purpose of the gabion basket is to keep the rock materials assembled in place. A gabion embankment has the advantage that it is a flexible structure that can tolerate foundation settlement and lateral movements, while still retaining stability, and the open structure of the rockfill permits water to move through the wall, so high pressures do not build up behind the embankment.

Ground Rupture

Natural hazards that pose a risk to people or structures at the Gregory Canyon Landfill would include ground rupture if the site were cut by an active fault. Because of the possibility of a fault contact extending through the canyon, GLA (1999) conducted an investigation to describe the formational contacts present on the site and evaluate the likelihood of a north-south trending fault projecting into Gregory Canyon. Careful inspection of the outcrops along SR 76 and the north

[33] In the sense used here, the term basin refers to the area that contributes overland flow to a given stream segment. The term "hanging" basin refers to a basin that drains from a cliff or very steep slope.



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Sources: GeoLogic Associates, 1998; David Evans and Associates, Inc., 1999; PCR Services Corporation, 1999

Exhibit 4.2-9
Potential Sources of Debris Flow Hazard and Gabion Containment Structures

flank of Gregory Mountain, where the contact would be reasonably expected to project if it were an extensive planar feature, did not indicate evidence for bedrock faulting in Gregory Canyon. The complete fault investigation is presented in Appendix F. No significant impacts from ground rupture are expected. No active, through-going faults have been identified within the project area, and, therefore, no significant impacts from ground rupture would occur.

Seismic shaking

Shaking induced by a nearby earthquake could trigger ground failures, which in turn could endanger lives, disrupt the landfill development, or disrupt nearby man-made structures. Failure of the excavation slopes was found to be unlikely given the dominant fracture orientations (Section 4.2.3.2) and therefore is not considered significant.

Soil Liquefaction

Seismic shaking can induce soil liquefaction of cohesionless soils that are saturated with water. Since grading operations at the project site will remove all loose soils from the footprint of the landfill, liquefaction will not occur within the landfill footprint. The ancillary facilities at the mouth of the canyon will be constructed over the alluvial wedge, however, and therefore soil liquefaction potential would exist in the event of strong seismic shaking.

Liquefaction typically occurs at depths less than 50 feet below ground surface (bgs), with the most susceptible conditions occurring in sandy soils with less than 15 percent silt and clay at depths shallower than 30 ft bgs. Deposits in which the zone of saturation is deeper than 50 ft bgs are generally stable regardless of their grain-size distribution.

GLA (1998) drilled and sampled the alluvial wedge at four different locations to a depth of 50 feet, and concluded that the soils are mostly silty sands and clayey sands with 14 to 45 percent silt and clay content. Depth to the zone of saturation varied between 19 and 26 feet bgs. The liquefaction susceptibility of these alluvial soils was then evaluated using the analytical procedure described by Seed and Idriss (1982). The procedure relies on the calculation of two parameters (cyclic-stress ratios, C_d and C_l) that determine liquefaction susceptibility. The first one represents the theoretical susceptibility to liquefaction of sediments with the same characteristics as those encountered, while the second represents actual resistance to liquefaction. Dividing the second ratio by the first one, the factor of safety against liquefaction is calculated. Table 4.2-8 summarizes the cyclic-stress ratios calculated for each location, and the calculated factors of safety. Factors of safety above 1.3 are considered conservatively adequate for engineering design.

As shown, the lowest calculated factor of safety is 1.39, and most other values are considerably higher. As a result, GLA (1998) concluded that the liquefaction susceptibility of the alluvial wedge at Gregory Canyon is low, and therefore, no significant impacts related to soil liquefaction are anticipated.

Slope stability of liner system

When first constructed, landfill liners can be regarded as a thinly layered section, in which many layers have low interface strengths (e.g., the interface between the plastic geomembrane and the overlying geotextile fabric on the slopes is generally of a low strength), and before they are buttressed by refuse they are susceptible to environmental stresses and potential sliding failures. To mitigate this potential, the geosynthetic materials (i.e., plastic geomembranes and geotextile fabrics) will be anchored at the top of the slope, and protected in place and weighted throughout

TABLE 4.2-8
RESULTS OF LIQUEFACTION SUSCEPTIBILITY ANALYSIS

DEPTH IN FEET	MATERIAL	PERCENT FINES	DEPTH TO WATER TABLE (FT)	C_D FIRST CYCLIC-STRESS RATIO	C_L SECOND CYCLIC-STRESS RATIO	FACTOR OF SAFETY
Boring 1						
20	SM	17%	19	0.200	0.320	1.60
30	SM		19	0.234	0.700	2.99
40	SM		19	0.240	0.686	2.86
Boring 2						
30	SP/SM	14%	25	0.207	0.288	1.39
40	SM		25	0.218	0.679	3.12
Boring 3						
25	SM-SC	45%	24	0.195	0.500	2.56
35	SM-SC		24	0.219	0.700	3.20
45	SM		24	0.218	0.443	2.03
55	SM		24	0.203	0.660	3.26
Boring 4						
35	SM-SC	20%	26	0.211	0.302	1.43
45	SM-SC		26	0.212	0.672	3.17
Source: GeoLogic Associates, 1998						

their extent with 20-pound sand bags placed on 5-foot vertical spacing. This construction protocol is standard in the industry, and has a successful performance record for 2:1 slopes, which is proposed in the project. If for any reason the liner system is damaged before it is weighted down by refuse the damaged portion of the liner would be repaired and reconstructed to maintain slope stability and retain containment capability. A mitigation measure has been included in Section 4.2.4 (Mitigation Measure 4.2-1) to mitigate this potentially significant environmental impact. After the incorporation of this mitigation measure, no adverse effects would occur.

Aqueduct Stability

During construction, movement of scrapers and other heavy equipment over the aqueduct pipes may be a source of distress to the existing pipelines. Construction of reinforced slabs at the access road and internal haul roads will transfer heavy equipment loading across each of the pipelines without placing loads on these pipes. The loading will be transferred to the bridge slab grade beams at each end of the bridge slabs. Polystyrene will be placed below each of these slabs to absorb the slab deflection and avoid incidental loading to the pipes. These measures will reduce the potential impacts of distress to the pipelines to below a level of significance. Vibration induced by blasting during landfill construction would not cause damage to the aqueduct. No significant impacts to the pipelines are anticipated.

Mineral Resources

Construction would not result in the loss of a known mineral resource that would be of value to the region and the residents of the state. The tonalite of Gregory Canyon could be considered a low-value mineral resource in its own right, as a source of dimension stone or crushed gravel. Landfill development would limit access to this resource, but this rock type is abundant in the Southern California batholith and project development would not affect its availability. As part of the proposed project, excavated rock materials from the landfill footprint would be processed on-site (i.e., crushed) and shipped off-site, thereby making this resource commercially available. Therefore, no significant impact to mineral resources would occur.

The proposed project will not impact sand and gravel materials contained in the San Luis Rey River. Mineral resources located in the San Luis Rey River will continue to be preserved under the MRZ-2 designation which preserves these resources. The MRZ-2 designation encompasses large segments of the San Luis Rey River which are not located on the project site. The proposed project will not impact the operation of existing sand and gravel mining operations located near the project site or affect the development of future sand and gravel mining operations in the area. No significant impacts to mineral resources on the site will occur as the result of construction activities for the project.

4.2.3.2 Long-Term (Operational) Impacts

Operational impacts include impacts associated with the operation of the bridge, landfill and associated uses.

The proposed Gregory Canyon Landfill has been designed as a canyon fill to be developed in consecutive phases over the site's life. The conceptual engineering design proposes four excavation and three fill phases or sequences. The development sequence is based on the master excavation plan, phasing plans, final grading plan, and established design criteria. Each excavation/fill phase will be broken down into smaller stages depending on site conditions and capital expenditure scheduling. The stages or sequences will be developed to create an environmentally sound operation which provides the most efficient operation and use of the facility. The smaller sequences will limit the amount of earth and refuse exposure.

The depth of excavation ranges from near zero to about 160 feet deep. The elevations of the bottom subgrade or floor area for the overall excavation range between approximately 370 feet amsl at the lowest elevation to 440 feet amsl along the southern portion of the bottom area.

Assuming a 4:1 cover ratio, approximately 12.4 mcy will be needed for project operations. An additional 1.83 mcy of soil will be needed for the containment system (0.63 mcy) and the final cover (1.2 mcy), for a total of 14.3 mcy of soil for operations. The proposed landfill development will include the excavation of approximately 9.8 million cubic yards of topsoils, alluvium/colluvium or weathered bedrock from within the landfill footprint.[34] Excavated colluvium and weathered bedrock material will be stockpiled for use during the operation of the landfill and at the time of landfill closure. Excess rock will be transported off-site for sale.[35] Based on drilling conducted on the site, approximately 40 percent of the material could be used directly as cover material. Therefore, about 3.9 mcy of material would be available for cover

[34] Total excavation within the landfill footprint will actually include 11.5 mcy, however 1.7 mcy will be used during initial construction.

[35] As indicated in Chapter 3 and discussed in Section 4.1, exportation and sale of material would require a Major Use Permit (MUP), which would be obtained if such activity were to occur. However, the borrow/stockpile areas have been designed to accommodate all of the material on-site. The analysis contained in this EIR presents a worst-case analysis for each topical area. For example, Traffic (Section 4.5) considers the exportation of material since the number of trips would increase if exportation were to occur while Aesthetics (Section 4.13) considers storage on-site as that would result in the maximum use of the borrow/stockpile areas.

needs from the refuse footprint excavation. The borrow/stockpile areas A and B have a combined acreage of approximately 87 acres. All of the material excavated from the borrow/stockpile areas will be colluvium and weathered bedrock which will be available for cover needs. The approximate volumes of soil material to be excavated from Borrow/Stockpile Areas B and A are 3.2 mcy and 1.3 mcy, respectively. In addition, the borrow/stockpile areas will each have a haul road. The maximum slope of the borrow/stockpile haul roads will be a 15 percent grade.

There would be 8.4 mcy of material readily available on-site for cover, or a shortfall over the life of the landfill of 5.9 mcy of soil materials. However, this shortfall will be offset by obtaining additional material for cover from on-site processing of the material by mechanical means (i.e., rock crushing) or weathering and the use of alternative daily cover (ADC) materials to reduce on-site cover demands and to maximize refuse capacity. The use of ADC has been shown to reduce refuse-to-daily/intermediate cover ratios from 4:1 to 7:1. This equates to a 37.5 percent reduction in on-site soil cover needs. The use of ADC can, therefore, reduce the daily/intermediate soil cover demand from 12.4 to 7.8 mcy, for a total project demand of 9.63 mcy (i.e., including containment system and final cover). Since 8.4 mcy of material is readily available, the potential shortfall would be 1.2 mcy of material. The 1.2 mcy of material will be gained by crushing approximately 20 percent of the harder rocky material from the landfill footprint excavation area. Processing activities will consist of rock crushing to render a finished product which meets a 6 inch minus or smaller size quantity. The rock crushing operations (both primary and secondary) will produce a material with uniform size distribution. The material will be screened as necessary to remove larger material. The finished product may also be blended with finer earthen material obtained from the alluvial deposits within the refuse footprint and the borrow/stockpile areas. The material blending will occur as the cover is placed and then compacted with heavy equipment. Using these methods, the daily and intermediate cover material will meet 27 CCR requirements for minimization of surface water infiltration and provide odor, litter and fire control as well as prevent scavenging. Therefore, with the use of ADC and the processing of materials on site, no importation of materials would be needed to meet the daily or intermediate cover requirements.

The JTD reflects the use of soil, as well as an ADC consisting of synthetic tarps during initial refuse disposal operations at the landfill. The selection of a synthetic tarp is based on previous demonstration projects performed throughout the State of California. These project studies found that woven synthetic tarps are the most durable. Synthetic ADC panels will be placed using landfill equipment and/or employees, and will be anchored along the edges and across the center of the landfill working face. The tarp will be removed before placing additional waste. The number of panels used each day will depend on the size and location of the refuse active face and the size of the panels. Tears in the tarp will be repaired as recommended by the manufacturer. Proposed design and operations at the Gregory Canyon Landfill will provide for the diversion of surface water from areas of active filling, thus minimizing potential contact between the proposed ADC and surface water run-on.

Other ADC materials, such as shredding green and used material, biosolids, and shredded tires, will be evaluated later with appropriate regulatory procedures under 27 CCR Sections 20690 and 20705 followed to demonstrate the ADC suitability at the Gregory Canyon Landfill.

<u>Slope stability of refuse fill</u>

Static and pseudo-static analyses

The stability of refuse fills is regulated by Section 21750(f)5 of Title 27 CCR, which states that the critical slope of the final refuse prism must have a factor of safety (i.e., the ratio of resisting forces over driving forces) of at least 1.5 under *dynamic* conditions (Section 21750(f)5(C)). The standard of practice also uses the concept of *static* factor of safety and has converged on static factors of safety of at least 1.5 for slopes where failure would impact the integrity of the waste containment system. Static conditions are those in which no external forces are imposed on the refuse prism. Dynamic conditions are those in which the lateral force of earthquake shaking is imposed on the refuse prism. The final fill plan with locations of slope stability sections is shown on Exhibit 4.2-10.

Although a large number of hypothetical "failure" scenarios can be envisioned, the main stability concerns for refuse fills are: (1) sliding of the whole prism, or of a frontal wedge, along a basal interface often occurring below the lining; and (2) sliding of a thin veneer of material (interim or final cover) under low confining pressures and sometimes high pore pressures.

GLA (1998) performed a 3-dimensional slope stability analysis of the refuse fill, using the program CLARA (Hungr et al., 1989), and the results are summarized in Table 4.2-9. The base case assumed the small soil buttress, currently designed (Exhibit 4.2-11), and the sensitivity case assumed a large buttress (Exhibit 4.2-12).

Within the low strength liner, the interface between the geotextile and the high-density polyethylene (HDPE) typically exhibits the lowest strength in both slope and floor liner areas. The internal strength of these liner components are much lower than those of the bedrock substrate or the refuse. As a result, the most critical area for refuse stability is along the liner system. While liner instability would be a significant project impact, in the two cases analyzed, the static factor of safety was larger than 1.8 (Table 4.2-9), and is acceptable under the current standards of practice. As a result, no significant slope stability impacts related to static refuse stability are anticipated.

Dynamic stability analysis of the refuse prism

Sections 21750(f)5(C) and 21750(f)5(D) of Title 27 of the California Code of Regulations state that the critical slope of the final refuse prism must have a factor of safety of at least 1.5 under *dynamic* conditions, and that if this is not the case a more rigorous analysis must be performed to estimate the magnitude of movement under seismic loading conditions. For the proposed landfill, the pseudo-static factor of safety was below 1.5, and therefore a more rigorous deformation analysis was performed, and is presented below.

Calculation of displacement for a rigid prism

Once the yield acceleration[36] of a potential slide block has been calculated, permanent displacement can be estimated by double numerical integration of those parts of a strong-motion

[36] Yield acceleration is the acceleration required to overcome frictional resistance and initiate sliding on an inclined plane.

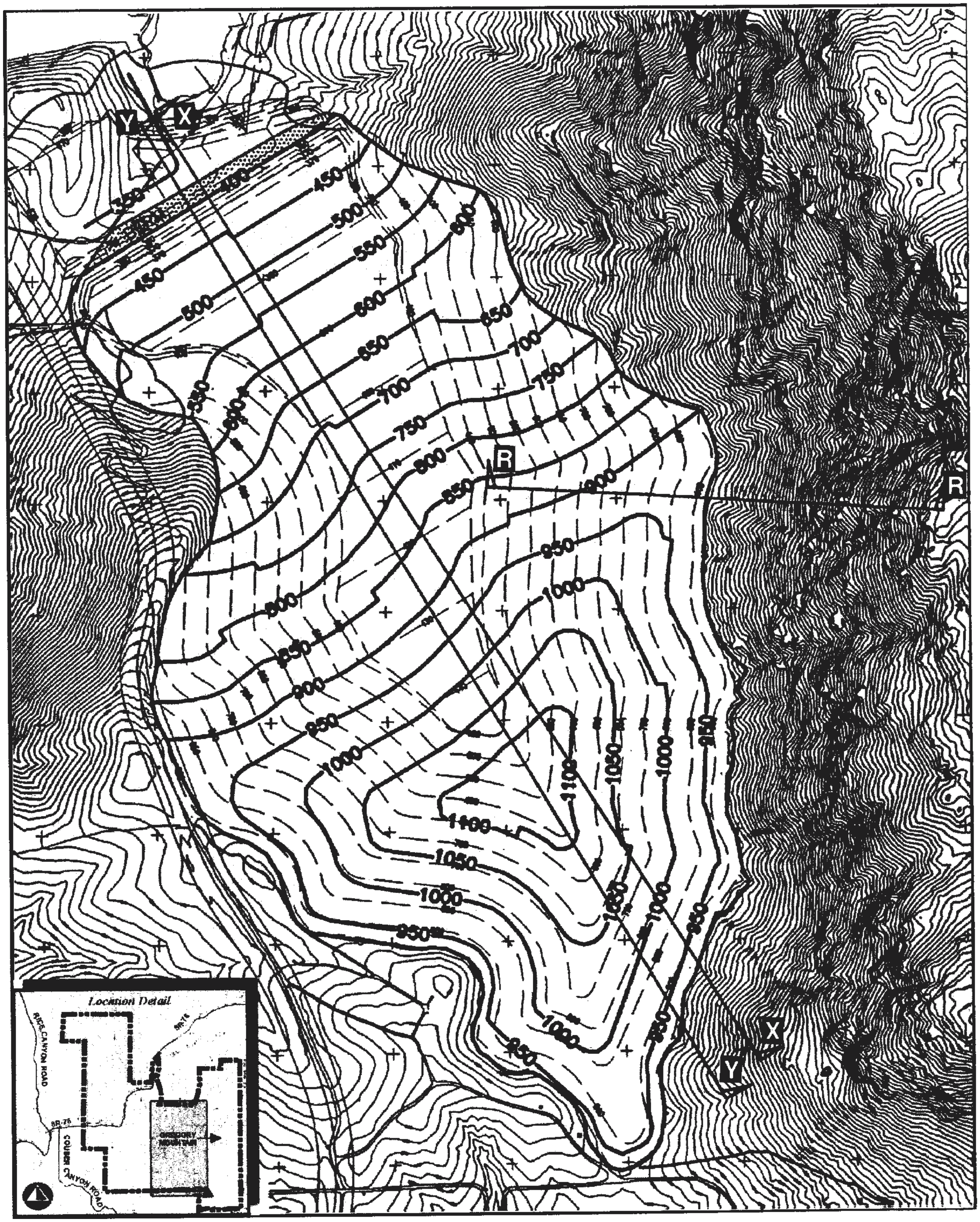

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Exhibit 4.2-10
Final Fill Plan with Locations of Slope Stability Sections

Sources: GeoLogic Associates, 1997; David Evans and Associates, Inc., 1999; PCR Services Corporation, 1999

TABLE 4.2-9
SUMMARY OF REFUSE FILL STABILITY ANALYSIS

RUN	TYPE OF RUN	STATIC FACTOR OF SAFETY	F.S. FOR *1* = 0.15G	YIELD ACCELERATION	COMMENTS
GREG3D5	Basal block failure	1.81	0.90	0.11g	Base case
GREG3D6	Basal block failure	1.83	0.89	0.11g	Base case
Source: GeoLogic Associates, 1998					

acceleration record[37] that lie above the yield acceleration. The calculation is based in the procedure initially proposed by Newmark (1965), and reviewed by Jibson (1993). Newmark's method models a failing slope as a rigid-plastic friction block[38] that has a discrete yield acceleration. The analysis consists of calculating the cumulative downhill displacement of the block as it is subjected to the effects of an earthquake acceleration-time history.[39]

Calculation of equivalent acceleration for a deformable refuse prism

Repetto et al. (1993) recognized that the propagation of seismic waves through a deformable refuse prism is a dynamic process, in which the intensity and direction of the acceleration varies with time from one point to the other. Throughout the vibrating refuse mass, peak horizontal accelerations are different from point to point and occur at different times. At any given moment during shaking, values at different points not only differ in peak values, but may also vary in direction. There is thus considerable opportunity for attenuation and/or amplification of the forces acting throughout the prism.

Attenuation and amplification through a refuse prism can be evaluated using a relatively simple, 1-dimensional wave propagation analysis such as that performed by the program SHAKE91 (Schnabel et al., 1972; Idriss and Sun, 1992). This program computes the response of a semi-infinite horizontally layered deposit overlying a uniform half-space[40] (bedrock in the case of Gregory Canyon) subjected to vertically propagating shear waves.

The initial acceleration time histories used for the bedrock at Gregory Canyon included two synthetic near-field records, one recorded near-field record (the 1940 El Centro earthquake recorded at a bedrock site six miles from the epicenter), and one synthetic far-field record. The

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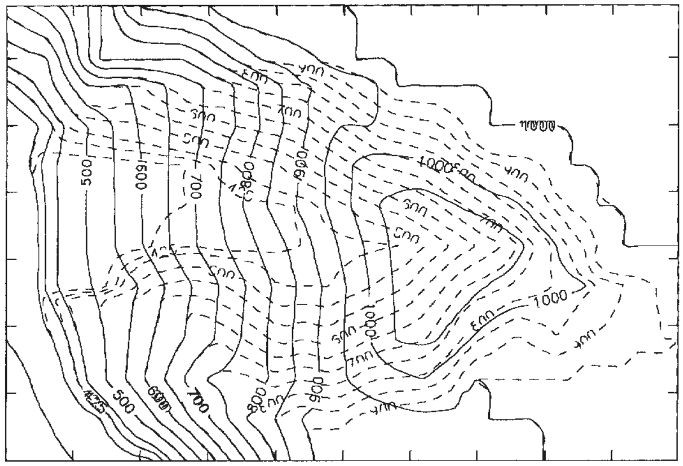
[37] An acceleration record is an actual record of the lateral acceleration forces experienced by a specific site during an earthquake measured at 1-second intervals. These values are measured by an instrument called an accelerometer.

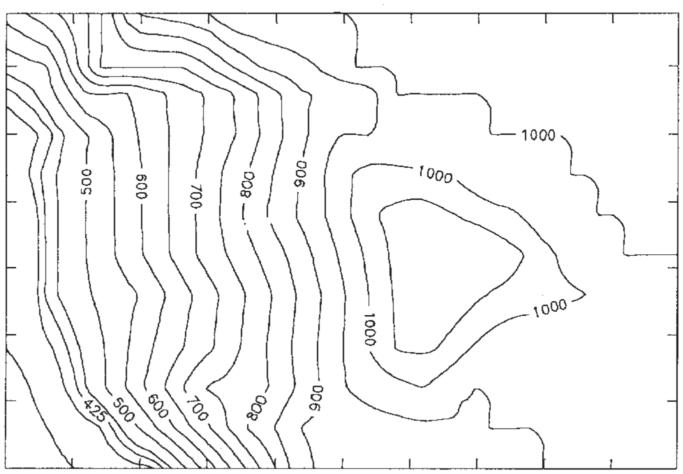
[38] In displacement analysis, a rigid-plastic friction block is a block that does not deform internally (or if it deforms is not by transient elastic deformation but by permanent plastic deformation), such that displacement of the block only occurs if the frictional resistance at its base is overcome by the earthquake-induced forces.

[39] For definition of earthquake acceleration-time history see acceleration record above.

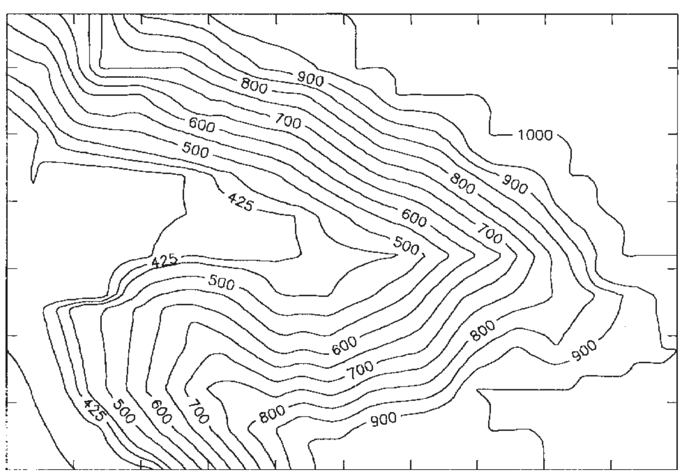
[40] Within a certain medium, like a refuse prism, seismic waves propagate in one direction at a constant velocity. When a seismic wave meets a surface of discontinuity, such as the boundary between a refuse pile and the atmosphere, or between bedrock and the surface of the ground, the waves are refracted or reflected. The reflected waves interfere with the incoming waves to generate complex patterns of lateral acceleration. For the analysis of seismic wave propagation through a landfill, the uniform half space refers to a refuse prism with lateral dimensions much larger than the vertical dimension, whose upper boundary is delimited by the atmosphere (the atmosphere is, then, the other half of the space).



CONFIGURATION OF
REFUSE FILL AND
FAILURE SURFACE



CONFIGURATION OF
REFUSE FILL



CONFIGURATION OF
FAILURE SURFACE

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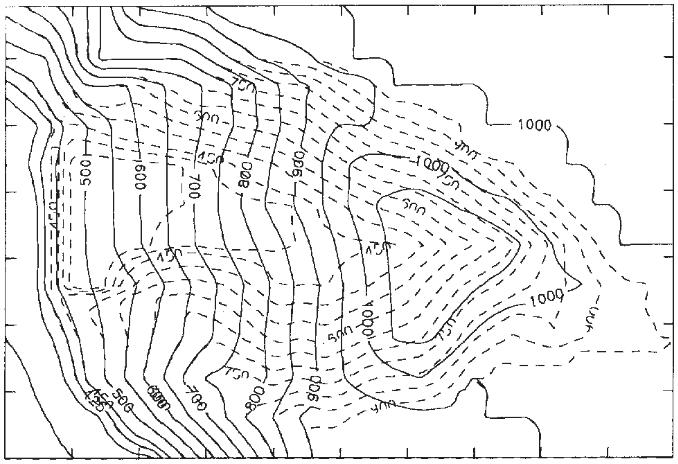
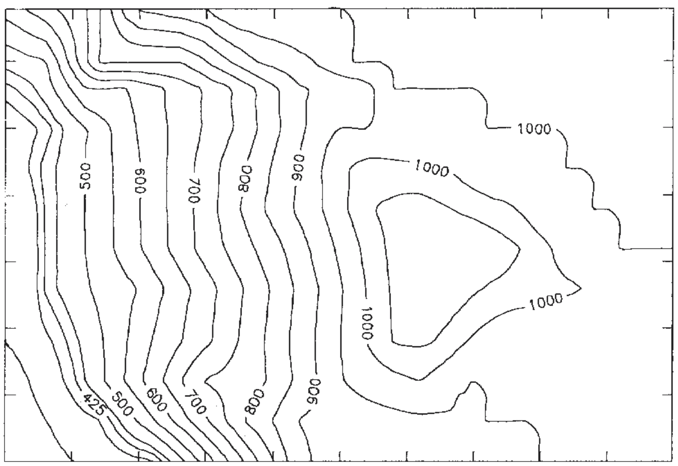
0 5000 FEET

Sources: GeoLogic Associates, 1998; David Evans and Associates, Inc., 1999;
PCR Services Corporation, 1999

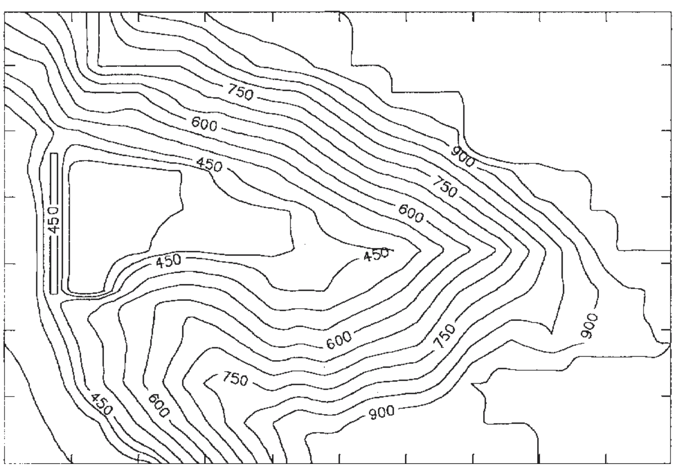
Exhibit 4.2-11
Small Buttress Scenario for
Slope Stability Analysis



CONFIGURATION OF REFUSE FILL AND FAILURE SURFACE



CONFIGURATION OF REFUSE FILL



CONFIGURATION OF FAILURE SURFACE

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0 5000 FEET

Sources: GeoLogic Associates, 1998; David Evans and Associates, Inc., 1999; PCR Services Corporation, 1999

Exhibit 4.2-12
Large Buttress Scenario for Slope Stability Analysis

one-dimensional wave propagation analysis was performed by GLA (1998) for four profiles across the proposed refuse fill. For each profile, the liner system and the overlying refuse were idealized as a system of homogeneous, visco-elastic[41] sublayers of infinite horizontal extent. The response of this idealized profile was then calculated considering vertically propagating shear waves.

In most cases the peak equivalent acceleration was below the yield acceleration, so the refuse prism should experience no displacement. Under the base case scenario (peak bedrock acceleration of 0.4g for near-field events, 0.1g for far-field events, and 0.11g yield acceleration) displacement was only predicted when the thickness of the refuse prism is 280 feet, and even then the total displacement was anticipated to be less than 2 cm (Table 4.2-10). Under "worst-case" conditions (peak bedrock acceleration of 0.6g for near-field events) there is one instance in which predicted deformation could be as high as 18.5 cm, when the thickness of the refuse prism is 280 feet. This does not imply that the landfill liner will tear during worst-case peak bedrock acceleration, and common practice in the industry assumes that displacements of up to 30 cm are tolerable for well-designed waste fills (Seed and Bonaparte, 1992). Therefore, the landfill liner could sustain much greater deformation without resulting in adverse impacts. However, recognizing the fact that significant seismic events have the potential to impact environmental control systems, a mitigation measure has been included in Section 4.2.4 (Mitigation Measure 4.2-2) to mitigate this potentially significant environmental impact. After the incorporation of this mitigation measure, no adverse effects would occur.

<u>Stability of the Final Cover</u>

Taking into account the high cohesion and friction angle of refuse (e.g., c = 900 psf and $\phi = 31°$ [Kavazanjian, 1997; Kavazanjian, et al., 1995]), relative to typical strengths of fine grained and/or granular fills or geosynthetics, it is reasonable to assume that a cover failure would follow the weakest interface in the cover itself, (e.g., a geosynthetic/soil interface).

For example, assuming a prescriptive final cover consisting of 24 inches of foundation soil, a textured plastic geomembrane, and 12 inches of topsoil, then the critical slip planes would be either the textured HDPE/vegetative soil interface, or failure of the topsoil itself. The stability of such an interface can be analyzed using the *infinite slope* method described by Duncan (1996). Using this method the factor of safety for a textured HDPE/vegetative soil interface on a 3:1 slope is 4.90, and for failure through topsoil on a 3:1 slope is 5.65. For the case where the topsoil is saturated, the respective factors of safety are 2.89 and 3.27 (GLA, 1998).

These final calculations indicate that final covers over slopes shallower than 3:1 are inherently stable. Still, given unusual circumstances, such as transient loading by seismic vibration or heavy equipment, portions of a final cover may experience cracking and minor displacements. The impact of such an event can be easily repaired through a regular maintenance and repair program, and therefore, this potential impact is considered less than significant.

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[41] A homogeneous visco-elastic material is a material in which the elastic properties are modified by "viscous creep," such as happens in soils and refuse. The application of a slowly varying stress leads to perfect elastic deformation (just like the slow deformation of the springs in a truck as it is slowly loaded), whereas for rapidly varying stresses the elastic deformation is dampened by viscous resistance to deformation (similar to the dampening of vibration by the shock absorbers in a truck moving quickly over rough terrain).

TABLE 4.2-10
RESULTS OF THE SHAKE AND NEWMARK DEFORMATION ANALYSES

	SHAKE RUN	PROFILE	THICKNESS OF REFUSE (FT)	INITIAL INPUT MOTION	PEAK EQUIVALENT ACCELERATION	DISPLACEMENT FOR YIELD ACCEL = 0.11G (IN CM)
BASE CASE	GREG1	1	280	A-1 at 0.1g	0.058	0.0
	GREG2	1	280	HBS at 0.4g	0.125	1.9
	GREG3	1	280	IMI at 0.4g	0.092	0.0
	GREG4	1	280	El Centro at 0.4g	0.114	0.1
	GREG5	2	475	A-1 at 0.1g	0.039	0.0
	GREG6	2	475	HBS at 0.4g	0.073	0.0
	GREG7	2	475	IMI at 0.4g	0.045	0.0
	GREG8	2	475	El Centro at 0.4g	0.069	0.0
	GREG9	3	630	A-1 at 0.1g	0.033	0.0
	GREG10	3	630	HBS at 0.4g	0.048	0.0
	GREG11	3	630	IMI at 0.4g	0.035	0.0
	GREG12	3	630	El Centro at 0.4g	0.052	0.0
	GREG13	4	350	A-1 at 0.1g	0.041	0.0
	GREG14	4	350	HBS at 0.4g	0.102	0.0
	GREG15	4	350	IMI at 0.4g	0.059	0.0
	GREG16	4	350	El Centro at 0.4g	0.092	0.0
WORST CASE	GREG17	1	280	A-1 at 0.15g	0.071	0.0
	GREG18	1	280	HBS at 0.6g	0.160	18.5
	GREG19	1	280	IMI at 0.6g	0.092	0.0
	GREG20	1	280	El Centro at 0.6g	0.126	2.2
	GREG21	2	475	A-1 at 0.15g	0.050	0.0
	GREG22	2	475	HBS at 0.6g	0.086	0.0
	GREG23	2	475	IMI at 0.6g	0.058	0.0
	GREG24	2	475	El Centro at 0.6g	0.086	0.0
	GERG25	3	630	A-1 at 0.15g	0.047	0.0
	GREG26	3	630	HBS at 0.6g	0.056	0.0
	GREG27	3	630	IMI at 0.6g	0.042	0.0
	GREG28	3	630	El Centro at 0.6g	0.075	0.0
	GREG29	4	350	A-1 at 0.15g	0.062	0.0
	GREG30	4	350	HBS at 0.6g	0.134	4.7
	GREG31	4	350	IMI at 0.6g	0.078	0.0
	GREG32	4	350	El Centro at 0.6g	0.105	0.0

Source: GeoLogic Associates, 1998

Stability of Borrow/Stockpile Areas

GLA reviewed the stability of the cut slopes in the borrow/stockpile areas, and calculated static factors of safety ranging between 2.10 and 4.04 for six critical sections. The pseudo-static factors of safety for these sections ranged from 1.50 to 2.89 for a seismic coefficient of 0.15. The latter values are equal or larger than the threshold of 1.5 required by the applicable regulations, and therefore, potential impacts related to borrow/stockpile area design and slope stability concerns are considered less than significant.

Landfill Settlement

Since most of the soils will be removed during the base excavation, compression of the rock remaining below the refuse fill is anticipated to be negligible. Settlement of the landfill itself will occur due to compression and decomposition of the refuse fill and movement of soils into the voids within the refuse. It is estimated that this settlement could equal 20 to 40 percent of the original refuse thickness and could potentially affect final grades and runoff control structures, landfill gas control systems, and the integrity of the final cover. The impact of such landfill settlement would be mitigated through a regular maintenance and repair program (Mitigation Measure 4.2-3). No adverse effects would occur after the incorporation of mitigation for landfill settlement.

Aqueduct Stability

Studies conducted by Woodward-Clyde Consultants (1995) concluded that 2:1 slopes are appropriate with a factor of safety of 1.5 under permanent static conditions and, therefore, the impacts on the aqueduct are not significant. To assess whether the landfill would impact the stability of the First San Diego Aqueduct during an earthquake event, GLA (2000) performed a pseudo-static analysis of the proposed landfill cut slopes adjacent to the aqueduct. Static analysis of modeled wedges indicates a factor of safety of 5.9. This means that the forces resisting movement are approximately six times greater than the forces causing movement. When subjected to ground acceleration associated with the Maximum Probable Earthquake (0.6 g), the factor of safety is calculated to be 1.7. This value exceeds the prescriptive 1.5 dynamic factor safety for landfill foundation and final fill slopes required by CCR Title 27.

Movement of scrapers and other heavy equipment over the aqueduct pipes could be a source of potential distress. In areas where equipment must travel over the aqueduct, proposed construction of reinforced slabs included as part of the project will reduce these potential impacts to below a level of significance.

Access Road and Bridge

A foundation investigation was conducted for construction of the access road and bridge over the San Luis Rey River (GLA, 1999). The earth materials encountered in borings drilled in this area consist of predominantly granular alluvium and fine-grained overbank deposits overlying the granitic bedrock. Colluvium overlies the bedrock in areas outside of the San Luis Rey alluvial channel. Review of a seismic refraction survey across the river at the bridge crossing identified unweathered bedrock at depths of between 66 feet (on the southeast) and 97 feet (on the northwest).

The foundation investigation provided conclusions and recommendations for pier design and construction including foundations, seismic design criteria, settlement, lateral loads, and corrosion protection. The investigation also evaluated lateral earth pressures for abutment walls and wing walls below the bridge, slope stability of proposed embankment fill slopes along the access road, provided a preliminary pavement design for the anticipated heavy traffic loads and earthwork guidelines. The complete foundation investigation report (GLA, 1999) is provided in Appendix F.

Because of the dense to very dense nature of the alluvium, potential seismic hazards from liquefaction and dynamic compaction from soil densification during earthquakes are not considered significant at the site. Ground shaking, in response to nearby and distant earthquakes, is anticipated at the proposed bridge site. This has been accounted for in the design of the bridge and is therefore not significant.

<u>Rockfalls and Transmission Towers</u>

Although a specific geotechnical investigation has not been performed at the relocation sites of the transmission towers, reconnaissance geologic mapping indicates that the new locations will be founded into the leucogranodiorite bedrock of Gregory Mountain, so foundation stability is not likely to be a concern.

Visual examination of the hillside east of the proposed landfill indicates several large boulders that may pose a rockfall hazard to the towers. Some of these boulders could become unstable if shaken by an earthquake or proposed blasting operations and, as a result, a pre-blast survey of these rocks will be performed by a qualified geologist to determine whether or not they are a threat to the new tower locations. If it appears that a boulder may be insecure and has the potential for being dislodged, landfill personnel will attempt to dislodge the boulder using pry bars. If it is not possible to dislodge the boulder by using pry bars, a rope, cable, or chain will be attached around the boulder and to a bulldozer located within the footprint of the landfill, but away from the potential path of descent of the boulder, should it be successfully dislodged. The boulder will then be pulled downward. If the boulder cannot be dislodged using the methods described above, the boulder will be left in place. A mitigation measure has been included in Section 4.2.4 (e.g., Mitigation Measure 4.2-4) to mitigate this potentially significant environmental impact. With the implementation of this mitigation measure, no adverse effects would occur.

4.2.3.3 Site Closure Impacts

Phased closure of the landfill may be implemented throughout development. When the landfill or a designated area for phased closure is brought to final grade, the final cover will be applied. The foundation layer will be a minimum of two feet in thickness and consist of soil material. A comprehensive Quality Assurance/Quality Control (QA/QC) Program will be developed and included in the Final Closure Plan for placement of the final cover. The primary purpose of the QA/QC program is to provide evidence that suitable materials and good practices are used to place the final cover and to document that the cover is placed in a manner consistent with the closure plan design specifications.

State and Federal regulations dictate that the final cover design have a permeability less than or equal to any bottom liner or natural underlying soil. Therefore, because the Gregory Canyon Landfill will be a lined refuse disposal facility, the final cover system design will include a barrier layer consisting of a synthetic cover (i.e., 60-mil liner low-density polyethylene geomembrane). The geomembrane will be overlain in deck areas by a geocomposite layer consisting of two geotextile layers with a high density polyethylene (HDPE) geonet placed between the two layers. This will facilitate drainage for the barrier layer.

The depth of the vegetative layer will be designed to allow for an adequate root depth to sustain natural vegetation while giving protection to the barrier layer from potential root penetration and the drying effects of evapotranspiration. To enhance slope protection and erosion control, final site faces will be planted with native vegetation. The vegetative cover will be a mixture of native grasses and plants which are compatible with the site end-use of nonirrigated, open space. Plants will be selected for their suitability to the local climate, drought resistance, percentage of surface coverage, root zone depths less than one foot, hardiness and low maintenance qualities.

The final deck area will have a minimum grade of three percent to promote drainage and allow for future settlement. Slight modifications to the proposed final contours may be necessary in the future to achieve optimum drainage control and prevent ponding and/or excessive erosion of completed fill areas or to reduce impacts associated with anticipated settlement throughout the post-closure maintenance period.

As noted previously, the landfill will be designed to withstand seismic impacts and potential geologic impacts associated with ground failure, slope stability, and ground rupture. The landfill will be closed in accordance with a Final Closure Plan, prepared and submitted to the appropriate regulatory agencies two years prior to the anticipated closure date. The Final Closure Plan, as well as Preliminary Closure and Post-Closure Maintenance Plan element, will include a proposed final cover design in compliance with State and federal regulatory requirements in effect at the time of closure. No significant geology and soils impacts related to closure are anticipated.

4.2.3.4 First San Diego Aqueduct Relocation Option

To ensure that the aqueduct relocation complies with all applicable standards, a geotechnical investigation will be completed to evaluate the types, distribution and engineering properties of the earth materials; address material excavatability and temporary trench stability; provide design parameters for trench wall support; provide recommendations for trench backfill materials; provide active and passive earth pressures for thrust block and/or other permanent underground structures; and estimate seismic effects on the pipeline and liquefaction potential. Recommendations of the geotechnical investigation will be implemented to ensure that no significant geotechnical impacts result from relocation of the aqueduct.

4.2.4 MITIGATION MEASURES AND PROJECT DESIGN FEATURES

Proposition C

Section 5H of Proposition C contains the following mitigation measure relative to potential impacts due to earthquakes:

MM 4.2.C5H: *All structures located at the Gregory Canyon site shall be designed by a qualified engineer to withstand the maximum probable earthquake, to avoid potential impacts associated with earthquakes and ground shaking.*

Project Design Features

- The engineered drainage system for the project includes desilting basins to control soil erosion and siltation.
- Reinforced slabs will be placed over the aqueduct easement so that earth-moving equipment places no weight on the pipelines while crossing the easement.
- A pre-blast survey will be conducted by a qualified geologist to identify areas of potential rockfall concern. Identified isolated rock masses will be removed as necessary if deemed insecure.

- Natural vegetation will be maintained to the maximum extent possible. Diversion structure(s) will be constructed within Basin 1 prior to the start of grading activities where debris flow risk is anticipated.

Impacts and Mitigation Measures

In addition to the mitigation measure contained in Proposition C, more specific mitigation measures have been developed to reduce potential geological impacts identified in the environmental analysis from project implementation.

Impact 4.2-1: *During construction, the liner system of the landfill could be susceptible to sliding failures.*

MM 4.2-1: Before the liner is buttressed with refuse, the geosynthetic materials (i.e., plastic geomembranes and geotextile fabrics) shall be anchored at the head of the slope, and weighted throughout their extent with 20 pound sand bags on five-foot vertical spacing. If the liner system were to be damaged before it is weighted down by refuse, the applicant shall repair, and if necessary reconstruct, the liner. Repairs to the geosynthetic materials will be completed and tested in accordance with regulations and project specifications. The RWQCB will perform field observations to ensure compliance.

Impact 4.2-2: *Although uncommon, significant seismic events could impact the landfill's environmental control systems, and the landfill liner could tear.*

MM 4.2-2: Following significant seismic events, inspection of all facilities and structures, as well as surrounding natural features, shall be performed, and necessary repairs shall be made. If a tear in the liner is identified, repairs to the geosynthetic materials shall be completed immediately by placing a patch over the torn sections and fusing the materials by patch-welding. The operator shall perform vacuum testing on the patch welds to ensure compliance with the standards established for the original liner construction. Patching shall be performed under strict construction quality assurance protocols used during original construction, and the RWQCB may be present to perform field observations at any time during the repair to ensure compliance with applicable regulations.

Impact 4.2-3: *Settlement of the landfill will occur due to compression and decomposition of the refuse fill and movement of soils into the voids within the refuse. Landfill settlement can affect final grades and runoff control structures, landfill gas control systems, and the integrity of the final cover.*

MM 4.2-3: A monitoring and maintenance program that includes annual topographic surveys to measure settlement, quarterly visual inspections to identify damage to the final cover or gas systems, and repair of these systems as required shall be implemented. The frequency of monitoring may be reduced after closure of the landfill. The gas collection system shall be flexible to accommodate settlement and allow for repair. The County of San Diego Department of Environmental Health will perform inspections to ensure compliance.

Impact 4.2-4: *Rockfalls could result in damage to landfill facilities or personnel.*

MM 4.2-4: Additional inspection of the rock masses surrounding the landfill will be completed every 5 years and/or after a significant earthquake event in order to identify new areas of potential rockfall concerns. The applicant's geotechnical consultant shall submit a letter to the County of San Diego Department of Environmental Health after any such inspection summarizing findings and necessary actions.

4.2.5 LEVEL OF SIGNIFICANCE AFTER MITIGATION

With the project design features and implementation of the proposed mitigation measures, potential impacts to soils and geology will be less than significant