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GEOTECHNICAL CONSULTANTS

UPDATE GEOTECHNICAL INVESTIGATION

MEADOWOOD (PANKEY RANCH) SAN DIEGO COUNTY, CALIFORNIA

PREPARED FOR

PARDEE HOMES
SAN DIEGO, CALIFORNIA

NOVEMBER 20, 2006
PROJECT NO. 06931-42-01

Project No. 06931-42-01
November 20, 2006

Pardee Homes
12626 High Bluff Drive, Suite 100
San Diego, California 92130

Attention: Ms. Karen Kosup

Subject: MEADOWOOD
(PANKEY RANCH)
SAN DIEGO COUNTY, CALIFORNIA
UPDATE GEOTECHNICAL INVESTIGATION

References:

1. *Geotechnical Feasibility Study [for] Pardee – Fallbrook (Pankey Ranch), San Diego County, California,* prepared by Geocon Incorporated, dated November 20, 2006 (Project No. 06931-42-01).

2. *Rock Rippability Study [for] Water Storage Tanks, Meadowood (Pankey Ranch), San Diego County, California,* prepared by Geocon Incorporated, dated June 5, 2006 (Project No. 06931-42-01).

3. *San Diego County Review Comments [for] Meadowood,* updated November 15, 2006.

4. *Tentative Map [for] Meadowood (Pankey Property),* prepared by Latitude 33, undated.

Gentlemen:

In accordance with your request, we have prepared this update geotechnical investigation for the Meadowood (Pankey Ranch) project located in Fallbrook, San Diego County, California. The accompanying report combines and summarizes results of previous geotechnical studies, responds to specific comments made by the County of San Diego Geology and Mineral Resources Review and updates our previous studies to address the current tentative map under review. Conclusions and recommendations pertaining to geotechnical aspects of proposed development are also included in this report. The findings of this update geotechnical investigation indicate no soil or geologic conditions exist that would preclude development of the site provided the recommendations contained in this report are followed.

If you have questions regarding this update geotechnical investigation, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

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TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

UPDATE GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report summarizes findings of previous geotechnical studies for the Meadowood Project (formerly Pankey Ranch) in the Fallbrook area of the County of San Diego, California (Reference No. 1), Geocon's rock rippability study dated June 5, 2006 (Reference No. 2) and recent small diameter borings performed in February 2006. The purpose of this report was to combine our previous studies into one comprehensive report including Geologic Maps (Figures 2 through 7), respond to review comments made by the County of San Diego and provide geotechnical recommendations for developing the property as shown on the current tentative map.

The scope of this update report included a review of previous geotechnical reports (References Nos. 1 and 2), review of the current tentative map and transposing geologic information including geologic contacts, subsurface exploratory excavations and seismic lines from previous studies onto the current tentative map.

Additional field studies were performed in 2006 that consisted of drilling small diameter hollow-stem and "air-track" borings. The results of our recent field studies are presented in this update. The logs of all exploratory excavation performed by Geocon Incorporated and other geotechnical firms are contained in Appendices A and C.

Laboratory tests were previously performed on selected representative samples obtained during the field investigation (2002 and 2006) to evaluate pertinent physical soil properties for engineering analysis to assist in providing recommendations for site grading and development. Details of laboratory testing and a summary of test results are presented in Appendix B.

The base map used for our geologic maps consisted of a reproducible copy (Autocad file) of the Tentative Map for Meadowood, San Diego County, California, undated. The map depicts the configuration of the property, existing topography, proposed grading of slopes, streets and building pads, geology as previously mapped in 2002 (updated in 2006) as well as approximate locations of exploratory trenches, small diameter borings, "air-track" borings and seismic traverses. Previous borings, trenches, and seismic traverses performed by others have also been included on the Geologic Maps (see Figures 2 through 7).

The conclusions and recommendations presented herein are based on an analysis of data obtained from exploratory field investigations, laboratory tests, review of previous reports for the property and our experience with similar soil and geologic conditions.

2. SITE AND PROJECT DESCRIPTION

The irregularly shaped property is situated in a major tributary drainage valley to the San Luis Rey River valley on the west-facing slopes of Monserate Mountain, in the County of San Diego, California. The site comprises approximately 389 acres of an actively operated ranch consisting of citrus and avocado trees. Several areas of undocumented fill and rubble-fill as well as water-storage reservoir embankments were observed within the major drainages on the west slope of Monserate Mountain. In addition, numerous buried waterlines fed by water wells and pumped river water presently irrigate fruit groves that occupy over half of the property acreage. Electric power is supplied by overhead lines while individual propane and septic systems are provided for a fruit-packing and maintenance facility, as well as several residences within the property.

Topographically, the site consists of the north-trending promontory-ridge of Monserate Mountain that slopes east toward Rice Canyon and west toward the northern branch of the San Luis Rey valley. The majority of developable land is situated on the west-facing slopes. Elevations on the site vary from approximately 280 to 800 feet above Mean Sea Level (MSL).

Current plans propose construction of building pads for residential, recreational, commercial/retail and large sheet graded pads for future development that include school sites, park sites or other suitable uses. Streets and other infrastructure, as well as underground utilities will be constructed. Grading has proposed maximum cuts and fills of approximately 40 feet. Some cut slopes are proposed at maximum inclinations of 1.5:1 (horizontal:vertical), however, the majority of cut slopes are proposed at maximum inclinations of 2:1 (horizontal:vertical). Cut slopes are shown to have maximum heights of approximately 90 feet. Fill slopes are proposed at maximum inclinations of 2:1 (horizontal:vertical) with maximum heights on the order of 90 feet.

The above locations and descriptions are based on a site reconnaissance and review of the referenced tentative map (see Figures 2 through 7). If development plans differ significantly from those described herein, Geocon Incorporated should be contacted for review and possible revisions to this report.

3. SOIL AND GEOLOGIC CONDITIONS

Three surficial soil types and three geologic formations were encountered during the field investigation. Surficial soil deposits include undocumented fill, topsoil, and alluvium. Formational units include Quaternary-aged Terrace Deposits, Cretaceous-aged Bonsall Tonalite, and San Marcos Gabbro (Larsen, 1948, and Tan, Siang S., 2000, see List of References). Surficial soil types and geologic units encountered are described below in order of increasing age. The approximate lateral extent of each soil and geologic unit is depicted on Figures 2 through 7 (Geologic Maps).

3.1 Undocumented Fill and/or Rubble Fill (Qudf, Qudfr)

Two westward-draining arroyos in the central portion of the site are filled with loose, dry or variable moisture, porous, undocumented fill containing oversize concrete rubble. Some areas of fill consist almost entirely of rubble. Attempts to penetrate portions of the fill resulted in refusal at a depth of approximately 10 feet (see logs of Trenches T-10 and T-11, Appendix A). The rubble fill is anticipated to be in excess of 20 feet thick in the deeper arroyos and canyons. The undocumented fill is characterized as loose, light gray to dark brown, silty to clayey sand and coarse gravel. It is also possible that isolated pockets of undocumented fills exist within the project site that are masked by agricultural activities. Undocumented fills are potentially compressible and subject to collapse with an increase in moisture content. Removal and recompaction including special placement procedures for the rubble fills will be necessary in areas planned to receive structural fill and/or settlement-sensitive structures.

3.2 Topsoil (Unmapped)

Topsoil on the order of 2 to 3 feet thick consisting of loose, dry to moist, dark brown, porous clayey to silty sand with occasional scattered gravel was encountered in exploratory trenches. The topsoil is anticipated to blanket a wide area of the site. The topsoil is considered compressible and unsuitable for support of additional structural fill and structural improvements and will require removal and recompaction.

3.3 Alluvium (Qal)

Alluvial deposits exist within main drainages on the site. Alluvial deposits are most extensive in the southern and western parts of the property that parallel the north branch of the San Luis Rey River valley. As encountered in exploratory excavations, alluvium within the main drainage along the western property margin extends to depths of approximately 20 to 40 feet (see Borings B-2 through B-4, SB-7 and SB-8, Appendix A). At the south end of the property, the alluvium extended to depths in excess of 50 feet (see SB-6). Within western trending tributary drainages that feed the main drainage alluvium depths on the order of 8 feet were encountered. The alluvium is typically comprised of an upper layer of loose silty fine sand or silt that is underlain in the main drainage by saturated fine to coarse sand resting on the underlying bedrock formation. The alluvium is compressible and will require remedial grading. Portions of the alluvium below the groundwater are also potentially liquefiable. A discussion of potential impacts from liquefaction is discussed in section 6.4.

3.4 Terrace Deposits (Qt)

Quaternary-age Terrace Deposits are present beneath alluvium in the southern valley and cropping out at higher elevations in the central and northwestern portions of the site. This unit consists of over

60 feet of medium-dense to dense reddish-brown silty to clayey fine to coarse sand. The Terrace Deposits are considered suitable for foundation and/or fill support in its present condition.

3.5 Bonsall Tonalite and Bonsall Tonalite Gneiss (Kb and Kbg)

Cretaceous-age units comprised of very strong siliceous granitic rocks crop out in the west-central and southeast margins of the property. Bonsall Tonalite underlies the southeast margins of the site, extending eastward offsite up the steep west-facing slopes of Rosemary's Mountain. This unit, especially on the steep mountain slopes, is manifested by light gray outcrops of large rounded boulders. Bonsall Tonalite Gneiss, is a related variety of tonalite, but contains parallel bands of weaker micaceous foliated rock, and is present as a northwest -trending low ridge in the west-central portion of the property. Seismic traverses in the Bonsall Tonalite units indicate marginal to nonrippable conditions at shallow depths using conventional heavy- duty grading equipment. The Bonsall Tonalite and Bonsall Tonalite Geneiss are considered suitable for support of structural improvements. Undercutting may be required on lots and in streets where this unit is exposed at grade to facilitate excavation of foundations and utilities.

Because of the frequent occurrence of large rounded boulders on the steep west-facing slopes of Rosemary's Mountain, there is a potential of rockfall and /or isolated rolling boulders. Measures to reduce rockfall potential are discussed herein.

3.6 San Marcos Gabbro (Ksm)

Early Cretaceous-age San Marcos gabbro is present over a majority of the central and northern portions of the site, underlying the lower, middle and upper slopes of Monserate Mountain (see Geologic Maps, Figures 2 through 7). Gabbro has a lower percentage of siliceous minerals (quartz) and as a result this bedrock unit is typically deeply fractured and weathered, causing relatively smooth rounded slopes with dark grayish-brown isolated outcrops and round “floater boulders.” According to personal communications with Mr. W. Pankey, several existing road cuts and borrow pits on the site were excavated to depths exceeding 20 feet with small-to-medium earth-moving equipment. Our seismic traverse data also suggests the presence of relatively deep weathered gabbro, with seismic velocities that indicate marginally rippable material to depths on the order of 20 feet. A previous report by Pacific Soils Engineering, Inc., dated August 15, 2000, indicated marginally rippable conditions in some areas of the gabbro of over 30 feet. The San Marcos Gabbro formation is considered suitable for support of structural loads and/or fills in its present condition; it can also provide a significant source of granular material suitable for pad capping.

Excavation of the gabbro may generate oversize “floaters” and “knobs” possibly requiring localized blasting and special handling and placement techniques. In addition, hardrock zones exposed at grade in cut lots will require undercutting and replacement with compacted low-expansive soils to facilitate excavation of foundations and shallow utilities.

4. ROCK RIPPABILITY

4.1 Geophysical Studies – Seismic Refraction

To evaluate rippability characteristics of the on-site materials, geophysical studies comprised of seismic traverses were conducted. One hundred feet long, seismic refraction traverses were performed using an EG&G Geometrics 1225-model, 12-channel seismograph unit. Figures 2 through 7 show the approximate location of the seismic traverses. Typically, depths that can be evaluated with seismic traverses are on the order of one-third the traverse length. This correlates to an approximate explored depth on the order of 30 feet for a 100-foot traverse length. Table 4.1 summarizes the seismic refraction data.

TABLE 4.1
SEISMIC TRAVERSES

Seismic Traverse No.	Average Velocity (ft./sec.)			Average Depth (ft.)			Length of Traverse (ft.)	Approx. Maximum Depth Explored (ft.)
	V_1	V_2	V_3	D_1	D_2	D_3		
S-1	1,100	2,400	3,500	2	15	--	100	30
S-2	1,000	2,500	2,900	2	7	--	100	30
S-3	1,500	3,400	6,000	5	21	>30	100	30
S-4	2,600	6,000	--	25	>30	--	100	30
S-5	1,000	3,100	5,000	3	24	>30	100	30
S-6	2,400	3,600	--	9	--	--	100	30
S-7	3,600	4,400	--	11	<30	--	100	30
S-8	2,000	6,800	--	8	<30	--	100	30
S-9	3,500	5,500	--	13	--	--	100	30
S-10	1,800	2,800	5,700	3	14	>30	100	30
S-11	1,600	3,800	--	4	--	--	100	30
S-12	4,600	6,000	--	6	<30	--	100	30
S-13	1,700	3,200	6,000	3	20	>30	100	30

V_1 = Velocity in feet per second of first layer of materials
V_2 = Second layer velocities
V_3 = Third layer velocities
D_1 = Depth in feet to base of first layer
D_2 = Depth in feet to base of second layer
D_3 = Depth in feet to base of third layer

NOTE:

For mass grading, materials with velocities of less than 4500 fps are generally rippable with a D9 Caterpillar Tractor equipped with a single shank hydraulic ripper. Velocities of 4500 to 5500 fps indicate marginally rippable and probable blasting. Velocities greater than 5500 fps generally require pre-blasting. For trenching, materials with velocities less than 3800 fps are generally rippable depending upon the degree of fracturing and the presence or absence of boulders. Velocities between 3800 and 4300 fps generally indicate marginally rippable and probable line blasting, and velocities greater than 4300 fps generally indicate non-rippable conditions and line blasting. The above velocities are based on a Kohring 505.

The reported velocities represent average velocities over the length of each traverse, and should not generally be used for subsurface interpretation greater than 100 feet from a traverse.

Seismic refraction data can be used to evaluate rock rippability and estimate depths at which excavation difficulty will occur. It should be recognized that rock rippability is a function of natural weathering processes, which can be variable and change vertically and horizontally over short distances depending on jointing, fracturing, and/or mineralogic discontinuities within the bedrock. With this in mind, a frequently used guideline to equate rock rippability to seismic refraction data has been included in Table 4.1. Velocities obtained from the seismic refraction traverses indicate soil/bedrock velocities vary between 1,000 feet per second (fps) and 6,800 fps. As indicated in Table 4.1, bedrock with velocities less than 4,500 fps are generally rippable with a D9 Caterpillar Tractor equipped with a single shank hydraulic ripper. Velocities greater than 4,500 generally indicate marginally to non-rippable material. The velocities we used to estimate boundaries between rippable and non-rippable material do not account for any productivity levels that grading contractors may apply to material when determining rippable versus non-rippable material. Perspective contractors should use their own judgment to identify the seismic velocity boundary between productive and non-productive ripping, and rippable versus non-rippable rock.

4.2 Geophysical Studies – "Air-Track" Borings

"Air-track" borings were performed on April 27, 2006 to evaluate rock rippability characteristics at the proposed water tank reservoir site located at the top of the ridge east of future Ridge Valley Drive. The study consisted of advancing 6 air-track borings using an ECM-370 track mounted air percussion drill rig equipped with a 4-inch-diameter bit. Boring depths varied from approximately 33 feet to 62 feet below existing grade. The air-track borings, in general, were advanced to approximately 10 feet below proposed finish tank pad grade. During drilling, drill rates were recorded using a stop watch. Drilling rates (penetration rates) are reported as the time required to drill one vertical foot. Drill penetration rates were used to evaluate rock rippability and to estimate depths at which difficult excavation will occur. Penetration rates (recorded in seconds per foot) for each air track boring are presented in Appendix A. The approximate locations of air track borings are depicted on Figures 2 through 7 (Geologic Maps).

A frequently used guideline to equate rock rippability to drill penetration rate is presented on Table 4.2. These general guidelines are typically based on drill rates using a rotary percussion drill rig similar to an Ingersoll Rand ECM 360 with a 3½-inch drill bit.

TABLE 4.2
ESTIMATED ROCK RIPPABILITY BASED ON AIR-TRACK PENETRATION RATES

Penetration Rate (seconds per foot)	Rippability Designation
0 to 20 seconds per foot	Rippable
Greater than 20 seconds per foot	Marginal to Non-rippable

The rippability designations listed above are based on the use of a D-9 or equivalent bulldozer equipped with a single shank ripper. Rippable materials can be excavated with heavy effort. Marginally rippable includes very heavy ripping and isolated zones of probable blasting. Non-rippable materials will require blasting to excavate the rock.

Similar to seismic refraction data, perspective contractors should use their own judgment and make their own interpretations to identify the penetration rate boundary between productive and non-productive ripping, and rippable versus non-rippable rock. Typically for productive rock excavation, marginally and nonrippable zones are blasted.

Rock rippability is a function of natural weathering processes which can vary vertically and horizontally over short distances depending on jointing, fracturing and/or mineralogic discontinuities within the bedrock. This is demonstrated by the varying penetration rates recorded between Borings AT-4 and AT-5 that were both located in the same tank footprint area. Based on field data collected, it seems likely that the majority of the cut at the tank pads will require blasting to efficiently excavate.

Estimates of anticipated volume of rock materials requiring blasting should be evaluated based on the information from each boring and drill penetration rate criteria acceptable to the contractor. Tank pad undercutting to facilitate foundation construction should be considered when calculating the volume of hard rock. Proposed cuts in hard rock areas can be expected to generate oversized fragments (rocks greater than 12 inches in dimension) which will necessitate typical hard rock handling and placement procedures during grading operations. Typical grading specifications including rock placement requirements are presented in Appendix D.

5. GROUNDWATER

Groundwater encountered in alluvium along the north branch of the San Luis Rey valley (Borings B-1 through B-5 and SB-6 through SB-10) occurs at depths ranging between 12 and 34.5 feet. Subsurface seepage and wet zones of perched groundwater also occur in tributary drainages, but is likely the result of citrus and avocado grove irrigation. It is anticipated the basal portion of surficial deposits may become wet or saturated during rainy seasons contributing to a perched groundwater condition. No springs or seeps were observed elsewhere in the project.

6. GEOLOGIC HAZARDS

6.1 Faulting

Based on our reconnaissance, evidence obtained in exploratory excavations, and a review of published geologic maps and reports, the site is not located on any known active or potentially active fault trace. One unnamed inactive fault (California Geological Survey County Report 3, 1963) was mapped approximately 3 miles northeast of the site. Projection of the strike of this fault shows that it does not come closer than 3 miles from the site and is not considered to pose a significant seismic risk to the site.

The nearest known *active* faults are the Temecula and Julian segments of the Elsinore Fault located approximately 7 and 8 miles northeast of the site, respectively. Major earthquakes occurring on the Elsinore Fault, or other regional active faults located in the southern California area, could subject the site to moderate-to-severe ground shaking within the life span of proposed structures.

6.2 Seismicity-Deterministic Analysis

Earthquakes that might occur on the Elsinore Fault or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. In order to determine the distance of known faults to the site, the computer program *EQFAULT*, (Blake, 2000), was utilized. In addition to fault location, *EQFAULT* was used to estimate ground accelerations at the site for the maximum anticipated seismic event.

Within a search radius of 62 miles (100 kilometers) from the site, 29 known active faults were identified. The results of the deterministic analyses indicate that the Elsinore-Temecula Fault is the dominant source of potential ground motion at the site. Earthquakes having a maximum earthquake Magnitude of 6.8 are considered to be representative of the potential for seismic ground shaking within the site (from this fault). The "maximum credible earthquake" is defined as the maximum earthquake that seems possible of occurring under the presently known tectonic framework (California Division of Mines and Geology Notes, Number 43). The estimated maximum peak ground acceleration from the Elsinore-Julian and Elsinore-Temecula Faults is approximately 0.33g.

Presented on Table 6.2 are the earthquake events and site accelerations based on attenuation relationships of Sadigh, *et al.* (1997) for the faults considered most likely to subject the site to ground shaking.

TABLE 6.2
MAXIMUM EARTHQUAKE MAGNITUDE AND PEAK SITE ACCELERATIONS

Fault Name	Distance From Site (miles)	Maximum Earthquake Magnitude	Peak Site Accelerations
Elsinore – Temecula	7	6.8	0.33
Elsinore – Julian	8	7.1	0.33
Newport – Inglewood (Offshore)	21	7.1	0.13
Rose Canyon	22	7.2	0.13
Point Loma*	22	7.2	0.13
Elsinore – Glen Ivy	23	6.8	0.09
San Jacinto – Anza	29	7.2	0.09
San Jacinto – San Jacinto Valley	30	6.9	0.07
Earthquake Valley	35	6.5	0.04
Coronado Bank	38	7.6	0.08

*Faults shown on Alquist-Priolo maps are southern extensions of the Rose Canyon Fault system.

While listing of peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including frequency and duration of motion and soil conditions underlying the site.

The site could be subjected to moderate to severe ground shaking in the event of a major earthquake on any of the faults referenced above or other faults in Southern California. With respect to seismic shaking, the site is considered comparable to the surrounding developed area.

6.3 Probabilistic Seismic Hazard Analysis

The computer program FRISKSP (Blake, 1995, updated 1998) was used to perform a site-specific probabilistic seismic hazard analysis. The program is a modified version of FRISK (McGuire, 1978) that models faults as lines to evaluate site-specific probabilities of exceedance of given horizontal accelerations for each line source. Geologic parameters not included in the deterministic analysis are included in this analysis. The program operates assuming the occurrence rate of earthquakes on each mappable Quaternary fault is proportional to the fault's slip rate. Fault rupture length as a function of earthquake magnitude is accounted for, and site acceleration estimates are made using the earthquake magnitude and closest distance from the site to the rupture zone. Uncertainty in each of following are accounted for: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from all earthquake sources, the program calculates the total average annual expected number of occurrences of a site acceleration greater than a specified value. Attenuation relationships suggested by Sadigh et al. (1997) were utilized in the analysis. The results of the analysis indicate that, for a 100-year exposure period and a 10 percent probability of occurrence (Upper bound Earthquake as defined in 1998 California Building Code Chapter 16), a mean site acceleration of 0.49g may be generated. This value corresponds to a return period of approximately 1,000 years (correct statistical return period of 949 years). For a return period of approximately 475 years (10 percent probability in 50 years), a mean site acceleration of 0.41g may be generated.

6.4 Liquefaction

Liquefaction is a phenomenon where loose, saturated, and relatively cohesionless soil deposits lose strength during strong ground motions. Primary factors controlling the development of liquefaction include intensity and duration of ground accelerations, characteristics of the subsurface soil, in situ stress conditions, and depth to groundwater. The "Simplified Method" of evaluating liquefaction potential, originally developed by Seed and Idriss (1971), with modifications and updates from Technical Report NCEER-97-0022 (1997) was used in conjunction with the computer program *LIQUEFY2* (Blake & Blake, 1998) to evaluate liquefaction potential. An acceleration of 0.33g (10 percent probability of exceedance in 50 years weighted to magnitude 7.5 earthquake) was used for the analysis (limit suggested by CDMG Special Publication 117).

Results of our liquefaction analysis indicate that for the level of ground shaking assumed, layers of alluvium below the groundwater could liquefy. Based on our analysis, a zone of approximately 10 to 14 feet of potentially liquefiable material exists in the main drainage area at the southwestern corner of the property (area near Borings B-2, SB-6, SB-7, and SB-8). Table 6.4 shows the depth of potentially liquefiable layers based on our analysis for the assumed ground shaking identified above.

Distress to foundation elements from liquefaction is usually the result of differential settlement within the liquefying strata, ground-surface disruption, or lateral spread. Because the drainage area is confined within the valley and no free face or slopes are present near the drainage, the potential for lateral spreading is considered low.

The potentially liquefiable zones were further evaluated to quantify the magnitude of settlement that could be expected within the liquefying strata. Methods suggested by Tokimatsu and Seed (1987) were used to evaluate settlement based on volumetric strain in the event of liquefaction. Based on this analysis, liquefaction-induced settlement on the order of 1 to 3 inches could occur in the main drainage area at the southwest portion of the property. Differential settlement is generally estimated to be one-half of the total settlement. It is our opinion that differential settlements within the liquefying strata would be less than 1.5 inches.

Damage due to liquefaction is greater when ground-surface disruption occurs. Research by Ishihara (1985) indicates the presence of a non-liquefiable surface layer may prevent the effects of at-depth liquefaction from reaching the ground surface. This can occur when the non-liquefiable layer is thick enough to resist upward pressures of the liquefying stratum. Based on charts prepared by Ishihara, and modified by Youd (1995), a non-liquefiable surface layer varying from 18 to 22 feet would be required to provide sufficient overburden pressure to resist upward pressures should the entire potentially liquefiable zones identified in each boring liquefy. Table 6.4 shows the required non liquefiable zone required to resist ground-surface disruption at each boring location. The non-liquefiable surface layer includes all material above the groundwater table (alluvium and compacted fill). Based on information obtained from exploratory borings, a 10 to 22-foot non-liquefiable surface layer currently exists above potentially liquefiable alluvium.

TABLE 6.4
SUMMARY OF POTENTIALLY LIQUEFIABLE LAYERS, ESTIMATED LIQUEFACTION SETTLEMENT, AND THICKNESS OF NON-LIQUEFIABLE SURFACE LAYER REQUIRED TO RESIST GROUND-SURFACE DISRUPTION

Location	Depth of Potentially Liquefiable Layers	Estimated Liquefaction Settlement (inches)	Required Non-Liquefiable Layer	Current Non-Liquefiable Layer Provided based on Existing Grades (feet)
B-2	19 to 30	3.2	19	19
SB-6	18 to 25 and 45 to 50	1.1	20	18
SB-7	29 to 39	2.3	18	22
SB-8	12 to 26	2.6	22	12

To provide a sufficient overburden layer to reduce the effects of at-depth liquefaction, consideration should be given to raising pad grades within potential liquefiable areas. It is estimated that approximately 2 to 10 feet of fill would be required in the main drainage at the southwestern corner of the property to provide the required non-liquefiable surface layer (assuming removal and recompaction of alluvium to within 2 to 3 feet of the groundwater table as recommended hereinafter).

6.5 Rockfall

A potential rock fall hazard exists on the east side of proposed Meadowood Boulevard from Pala Road to approximately 2,000 feet into the project area. During our field studies boulders on the order of 20 feet in diameter were observed on the natural slopes above the road. The natural hillside has slope inclinations ranging between 1.3:1 (horizontal:vertical) to 3:1 (horizontal:vertical) with slightly steeper slopes at 1:1 inclinations occurring over very short distances of approximately 20 feet. Based on field observations the highest potential source area for rock fall is located approximately 1,200 feet north of Pala Road at an elevation of approximately 500 feet MSL. Several large boulders were observed at the base of this slope in the flat area west of future Meadowood Boulevard. The area considered to have the highest potential for rock fall, based on our observations of fallen boulders, has been mapped as Qrf on Figures 2 through 7, Geologic Map. The potential for rock fall hazard can be reduced by providing an open space buffer zone beyond the toe of the native hillside, construction of embankments and debris fences along the hillside toe, and rock anchoring of suspect boulders.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 No soil or geologic conditions were encountered which would preclude development of the property as proposed, provided the recommendations contained in this report are followed.

7.1.2 The site is underlain by compressible surficial soil deposits consisting of undocumented fill, topsoil and alluvium. Surficial soils will require remedial grading in the form of removal and recompaction, with the exception of alluvial soils that are located below the groundwater table. Cretaceous-aged igneous and metamorphic rock, and Quaternary-aged terrace deposits underlie surficial soil deposits and are considered suitable for support of additional loads from proposed compacted fills and/or settlement sensitive structures and improvements.

7.1.3 Development of the site as proposed will generate a relatively large volume of oversize rock as well as shot rock. Earthwork considerations which need to be addressed within the development schedule are summarized as follows:

- Grading and blasting operations should be designed to generate a sufficient quantity of granular material for use as low expansive capping material. Where this is not possible, rock crushing may be required.
- Consideration should be given to stockpiling select materials to be utilized for capping.
- Oversize rock should be placed in deeper fill areas in accordance with the *Recommended Grading Specifications* presented in Appendix D.

7.2 Soil and Excavation Characteristics

7.2.1 On-site soils consist predominately of fine to coarse grained, silty sands, clayey sands and sandy silts. These materials generally have a *very low* to *medium* expansion potential and should provide good capping material for streets and pads. Building pads that expose highly expansive material should be undercut a minimum of 3 feet and replaced with low expansive compacted fill soil. During grading sufficient expansion index testing should be performed to evaluate the expansion potential of materials within the upper 3 feet of finish grade.

7.2.2 Water soluble sulfate testing performed on representative samples of the soil and geologic units encountered indicate that onsite materials have a very low sulfate content and a corresponding *negligible* sulfate rating based upon Table 19-A-4 of the 1997 Uniform Building Code (UBC). It should be noted the presence of water-soluble sulfates is not a visually discernible characteristic. Therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e. addition of fertilizers and other soil nutrients) or chemicals within the local water supply may affect the sulfate concentration. During grading sufficient sulfate testing of finish grade soils should be performed to evaluate the sulfate content of subgrade soils.

7.2.3 Surficial deposits (undocumented fill, topsoil, and alluvium) can be excavated with light to moderate effort with conventional heavy duty grading equipment. Areas of undocumented fill composed of rubble and concrete (Qudfr) may require a heavy effort to excavate. Heavy effort may also be required to excavate cemented terrace deposit or terrace deposits containing oversize material.

7.2.4 Excavations within Bonsall Tonalite and San Marcos Gabbro will require a very heavy effort to excavate. Less weathered zones will require rock breaking or blasting. Large rock concretions and floaters will likely be generated during excavation. Placement of oversize (defined as material with nominal dimension of 12 inches of greater) material should be performed in accordance with Appendix D (Grading Specifications).

7.3 Subdrains

7.3.1 Subdrains are recommended to reduce the potential for adverse impacts associated with observed and potential seepage conditions and to collect perched water that migrates along the contact between natural ground and fill surfaces. Figure 13 presents a typical canyon subdrain detail. Recommended subdrain locations are depicted on our Geologic Maps (Figures 2 through 7, map pocket). The approximate locations for subdrain outlet points or connection points are also shown on the Geologic Maps.

7.3.2 The lower approximately 20 feet of subdrains should consist of non-perforated PVC pipe. The perforated/non-perforated joint should have a concrete cutoff wall built in accordance with Figure 14. The subdrains shown on the Geologic Maps should outlet at the toe of fill slopes or connected to the storm drain system. Subdrains that outlet at the toe of slopes or onto surface grades should be provided with a concrete outlet headwall at the outlet point in accordance with Figure 15.

7.3.3 Final grading plans should show locations of all proposed subdrains. Upon completion of remedial excavations and subdrain installation, the project civil engineer should survey drain locations and prepare an “as-built” map depicting surveyed locations and elevations of the drainpipes. Geocon Incorporated should be notified when subdrain outlet headwall installation is complete to verify their location.

7.4 Grading

7.4.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix D. Where the recommendations of Appendix D conflict with this section of the report, **the recommendations of this section take precedence.**

7.4.2 Prior to commencing grading, a pre-construction conference should be held at the site with the owner or developer, grading contractor, civil engineer, geotechnical engineer, and county officials in attendance. Special soil handling requirements can be discussed at that time.

7.4.3 Proposed building pad and structural improvement areas, and areas to receive fill should be cleared of any deleterious material, vegetation and debris prior to commencing grading. Organic or unsuitable material generated during stripping should be exported from the site.

7.4.4 All compressible deposits (undocumented fill, topsoil, and alluvium above groundwater) should be removed and recompacted within structural improvement areas. Based on our field investigation, undocumented fills may be in excess of 20 feet thick in the deeper arroyos and canyons. Topsoil generally blankets a wide area of site with thicknesses of 2 to 3 feet. Trenches performed within tributary drainages indicates maximum alluvial thicknesses ranging from 8 to 10 feet. In the main drainages along the western property margin and in the southwestern portion of the site, alluvial thicknesses of approximately 10 to greater than 50 feet were encountered. It is anticipated that removal of alluvium in this area should be possible to within 3 feet of the groundwater elevation.

7.4.5. Removals within areas of canyon cleanouts and/or toes of proposed fill slopes should extend horizontally beyond the edge of improvements a distance equal to the depth of removal. Because compressible alluvium exists along the western property margin, structural setbacks may be required if remedial removals cannot extend laterally beyond the property limits. A typical detail of remedial grading beyond proposed grading and potential structural setback is presented in Figure 12.

7.4.6 Where groundwater is present, alluvium removals can be terminated 3 feet above the groundwater elevation. Sufficient potholing should be performed to establish the groundwater elevation in cleanout areas. Settlement monuments should be installed at the completion of grading in areas where alluvium was left in place. Actual locations of settlement monuments can be determined during grading based on observations of field conditions. A typical settlement monument detail is presented on Figure 16.

7.4.7 Portions of alluvium left in-place below the groundwater table are potentially liquefiable. Surface manifestation due to liquefaction may consist of surface rupture and/or sand boils, and surface settlement. Sand boils occur where liquefiable soil is extruded upward through the soil deposit to the ground surface. Providing an increase in overburden pressure and a compacted fill mat can mitigate surface manifestation. The southwest portion of the site will require additional fill on the order of 2 to 10 feet above existing grades to provide sufficient overburden pressure to resist upward pressures of the liquefying stratum. The potential for liquefaction will be mitigated in areas where complete removal and recompaction of alluvium is performed.

7.4.8 After removal of compressible deposits as recommended above, the exposed ground surface should be scarified, moisture conditioned and compacted. Fill soils may then be placed and compacted in layers to the design finish grade elevations. All fill, including backfill and scarified ground surfaces should be compacted to at least 90 percent of laboratory maximum dry density as determined by ASTM Test Procedure D-1557-02; at or slightly above optimum moisture content. Fill areas with in-place density test results indicating moisture contents less than optimum will require additional moisture conditioning prior to placing additional fill.

7.4.9 Oversize materials should be kept at least 10 feet below proposed finish grade elevations beneath building pads and 2 feet below the deepest utility in streets. This recommendation is intended to facilitate excavation for future improvements, such as swimming pools and underground utilities.

7.4.10 Residential lots should be graded such that the material within 3 feet of finish grade has a *low* expansion potential (Expansion Index less than 50).

7.4.11 A potential exists for rockfall from the west-facing slope of Rosemary's Mountain located immediately south of Monserate Mountain. Large boulders are present within the valley indicating previous rock fall. Future boulder instability as a result of seismic activity or erosional events could potentially dislodge boulders. Typical remediation measures for rockfall include providing an open space buffer zone beyond the toe of the native hillside, and/or construction of embankments and debris fences along the hillside toe. Individual rock anchoring can also be performed for suspect boulders.

7.4.12 The Rosemary Mountain Quarry Project limits may be located near the property limits of the Meadowood Project (Pankey Ranch), however we understand that proposed mining activities are located several thousand feet from the Meadowood project and on the opposite site of Monserate Mountain. From a geotechnical standpoint, impacts may include seismic shaking from blasting activities, which may result in triggering rock falls. Mitigating the potential for rock falls, regardless of the event that triggers the rock fall, is discussed above.

7.4.13 Further analyses of impacts from this project to future mining operations in the local area are beyond the scope of this update geotechnical investigation. Other impacts may include items such as traffic, noise and dust, however these are not geotechnical issues and should be addressed by others. If specific geotechnical issues arise associated with mining activities, specific issues can be addressed at that time.

7.5 Slopes

7.5.1 It is recommended that fill slopes be constructed at inclinations no steeper than 2:1 (horizontal:vertical). The tentative map indicates the majority of cut and fill slopes are proposed at inclinations of 2:1 (horizontal:vertical), however one cut slope located at the north end of the site has a proposed inclination of 1.5:1 (horizontal:vertical). Project cut and fill slopes comprised of granular soils or intact rock should be stable up to heights of approximately 100 feet. Slope stability analyses were based on assumed direct shear strength parameters utilizing our experience. The shear strengths used in the analysis are considered conservative. The results of our analyses indicate factors of safety in excess of 1.5 for both deep seated and surficial stability. Results of the analyses are presented on Figures 8 through 11.

7.5.2 Cut slopes should be observed during grading by an engineering geologist to observe the exposed slope face to assess if adverse geologic conditions exist. Remedial grading procedures may be recommended at that time if adverse conditions are observed.

7.5.3 The outer 15 feet of fill slopes, measure horizontal to the slope face, should be composed of properly compacted granular “soil” fill to reduce the potential for surface sloughing.

7.5.4 All fill slopes should be overbuilt at least 3 feet horizontally, and cut to design finish grade. As an alternative, fill slopes may be compacted by backrolling at vertical intervals not to exceed 4 feet and then trackwalking with a D-8 bulldozer, or equivalent, such that the soils are uniformly compacted to at least 90 percent relative compaction to the face of the finish slope.

7.5.5 All slopes should be planted, drained and properly maintained to reduce erosion.

7.6 Preliminary Foundation and Concrete Slabs-On-Grade Recommendations

7.6.1 Foundations for commercial/industrial buildings can be provided in update geotechnical reports for individual buildings as they are developed. In general, conventional continuous and/or isolated spread footings can be used for residential one- or two-story homes and commercial concrete tilt-up and/or masonry block buildings.

7.6.2 The following foundation recommendations are for proposed one or two-story residential structures. The foundation recommendations have been separated into three categories based on either the maximum and differential fill thickness or Expansion Index. The foundation category criteria are presented in Table 7.6.1.

TABLE 7.6.1
FOUNDATION CATEGORY CRITERIA

Foundation Category	Maximum Fill Thickness, T (Feet)	Differential Fill Thickness, D (Feet)	Expansion Index (EI)
I	$T<20$	--	$EI \leq 50$
II	$20 \leq T<50$	$10 \leq D<20$	$50<EI \leq 90$
III	$T \geq 50$	$D \geq 20$	$90<EI \leq 130$

7.6.3 Final foundation categories for each building or lot will be provided after finish pad grades have been achieved and laboratory testing of the subgrade soil has been completed.

7.6.4 Table 7.6.2 provides minimum foundation and interior concrete slab design criteria for conventional foundation systems based on soil conditions.

TABLE 7.6.2
CONVENTIONAL FOUNDATION RECOMMENDATIONS BY CATEGORY

Foundation Category	Minimum Footing Embedment Depth (inches)	Continuous Footing Reinforcement	Interior Slab Reinforcement
I	12	Two No. 4 bars, one top and one bottom	6 x 6 - 10/10 welded wire mesh at slab mid-point
II	18	Four No. 4 bars, two top and two bottom	No. 3 bars at 24 inches on center, both directions
III	24	Four No. 5 bars, two top and two bottom	No. 3 bars at 18 inches on center, both directions

7.6.5 Embedment depths presented in Table 7.6.2 should be measured from the lowest adjacent pad grade for both interior and exterior footings. Conventional foundations should have a minimum width of 12 inches.

7.6.6 Concrete slab-on-grade should be a minimum of 4 inches thick for Foundation Categories I and II and 5 inches thick for Foundation Category III. The concrete slabs-on-grade should be underlain by 4 inches and 3 inches of clean sand for 4-inch thick and 5-inch-thick slabs, respectively.

7.6.7 As an alternative to conventional foundation recommendations, consideration should be given to the use of post-tensioned concrete slab and foundation systems for the support of proposed structures. The post-tensioned systems should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (UBC Section 1816). Although this procedure was developed for expansive soil conditions, it is understood that it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented on Table 7.6.3 for the particular Foundation Category designated.

TABLE 7.1.3
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI) Design Parameters	Foundation Category		
	I	**II**	**III**
1. Thornthwaite Index	-20	-20	-20
2. Clay Type – Montmorillonite	Yes	Yes	Yes
3. Clay Portion (Maximum)	30%	50%	70%
4. Depth to Constant Soil Suction	7.0 ft.	7.0 ft.	7.0 ft.
5. Soil Suction	3.6 ft.	3.6 ft.	3.6 ft.
6. Moisture Velocity	0.7 in./mo.	0.7 in./mo.	0.7 in./mo.
7. Edge Lift Moisture Variation Distance	2.6 ft.	2.6 ft.	2.6 ft.
8. Edge Lift	0.41 in.	0.78 in.	1.15 in.
9. Center Lift Moisture Variation Distance	5.3 ft.	5.3 ft.	5.3 ft.
10. Center Lift	2.12 in.	3.21 in.	4.74 in.

7.6.8 UBC Chapter 18, Div. III, **§**1816 uses interior stiffener beams in its structural design procedures. If the structural engineer proposes a post-tensioned foundation design method **other than UBC Chapter 18, Div. III, §1816**, the following recommendations apply:

- The deflection criteria presented in Table 7.6.3 are still applicable.

- Interior stiffener beams be used for Foundation Categories II and III.

- The embedment depth and width of the perimeter foundations should be at least 12 inches.

- The perimeter footing depth should be at least 18 inches and 24 inches for foundation categories II and III, respectively.

7.6.9 Our experience indicates post-tensioned slabs are susceptible to excessive edge lift, regardless of the underlying soil conditions, unless reinforcing steel is placed at the bottom of the perimeter footings and the interior stiffener beams. Current PTI design procedures primarily address the potential center lift of slabs but, because of the placement of reinforcing tendons in the top of the slab, the resulting eccentricity after tensioning can reduce the ability of the system to mitigate edge lift. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for proposed structures.

7.6.10 During construction of post-tension foundations, the concrete should be placed monolithically. Under no circumstances should cold joints form between the footings/ grade beams and the slab during the construction of the post-tension foundation system.

7.6.11 Slabs expected to receive moisture sensitive floor coverings or used to store moisture sensitive materials should be underlain by a vapor inhibitor covered with at least 2 inches of clean sand or crushed rock as recommended herein. If crushed rock will be used, the thickness of the vapor inhibitor should be at least 10 mil to prevent possible puncturing.

7.6.12 Category I, II, or III foundations may be designed for an allowable soil bearing pressure of 2,000 pounds per square foot (psf) (dead plus live load). This bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

7.6.13 The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended for Category III. Where this condition cannot be avoided, isolated footings should be connected to the building foundation system with grade beams.

7.6.14 For Foundation Category III, consideration should be given to using interior stiffening beams and connecting isolated footings and/or increasing the slab thickness. In addition, consideration should be given to connecting patio slabs, which exceed 5 feet in width, to the building foundation to reduce the potential for future separation to occur.

7.6.15 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisture conditioned, as necessary, to maintain a moist condition as would be expected in any such concrete placement.

7.6.16 Where buildings or other improvements are planned near the top of a slope steeper than 3:1 (horizontal:vertical), special foundations and/or design considerations are recommended due to the tendency for lateral soil movement to occur.

- For fill slopes less than 20 feet high, building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

- When located next to a descending 3:1 (horizontal:vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. An acceptable alternative to deepening the footings would be the use of a post-tensioned slab and foundation system or increased footing and slab reinforcement. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.

- If swimming pools are planned, Geocon Incorporated should be contacted for a review of specific site conditions.

- Swimming pools located within 7 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming pool wall within 7 feet of the slope face be designed assuming that the adjacent soil provides no lateral support. This recommendation applies to fill slopes up to 30 feet in height, and cut slopes regardless of height. For swimming pools located near the top of fill slopes greater than 30 feet in height, additional recommendations may be required and Geocon Incorporated should be contacted for a review of specific site conditions.

- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures which would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

7.6.17 The recommendations of this report are intended to reduce the potential for cracking of slabs due to expansive soil (if present), differential settlement of existing soil or soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

7.6.18 Geocon Incorporated should be consulted to provide additional design parameters as required by the structural engineer.

7.7 Retaining Walls and Lateral Loads

7.7.1 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 35 pounds per cubic foot (pcf). Where the backfill will be inclined at no steeper than 2.0 to 1.0, an active soil pressure of 50 pcf is recommended. These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an Expansion Index of less than 50. For those lots with finish grade soils having an Expansion Index greater than 50 and/or where backfill materials do not conform to the above criteria, Geocon Incorporated should be consulted for additional recommendations.

7.7.2 Unrestrained walls are those that are allowed to rotate more than 0.001H at the top of the wall. Where walls are restrained from movement at the top, an additional uniform pressure of 7H psf (where H equals the height of the retaining wall portion of the wall in feet) should be added to the above active soil pressure.

7.7.3 All retaining walls should be provided with a drainage system adequate to prevent buildup of hydrostatic forces and should be waterproofed as required by the project architect. The use of drainage openings through the base of the wall (weep holes, etc.) is not recommended where seepage could be a nuisance or otherwise adversely impact property adjacent to the base of the wall. The above recommendations assume a properly compacted granular (Expansion Index less than 50) backfill material with no hydrostatic forces or imposed surcharge load. If conditions different than those described are anticipated, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

7.7.4 In general, wall foundations having a minimum depth and width of one foot may be designed for an allowable soil bearing pressure of 2,000 psf, provided the soil within 3 feet below the base of the wall has an Expansion Index of less than 90. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, Geocon Incorporated should be consulted where such a condition is anticipated.

7.7.5 For resistance to lateral loads, an allowable passive earth pressure equivalent to a fluid density of 300 pcf is recommended for footings or shear keys poured neat against properly compacted granular fill soils or undisturbed natural soils. The allowable passive pressure assumes a horizontal surface extending at least 5 feet or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material not protected by floor slabs or pavement should not be included in the design for lateral resistance. An allowable friction coefficient of 0.4 may be used for resistance to sliding between soil and concrete. This friction coefficient may be combined with the allowable passive earth pressure when determining resistance to lateral loads.

7.7.6 The recommendations presented above are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 8 feet. In the event that walls higher than 8 feet or other types of walls are planned, such as crib-type walls, Geocon Incorporated should be consulted for additional recommendations.

7.8 Slope Maintenance

7.8.1 Slopes that are steeper than 3:1 (horizontal:vertical) may, under conditions that are both difficult to prevent and predict, be susceptible to near surface (surficial) slope instability. The instability is typically limited to the outer three feet of a portion of the slope and usually does not directly impact the improvements on the pad areas above or below the slope. The occurrence of surficial instability is more prevalent on fill slopes and is generally preceded by a period of heavy rainfall, excessive irrigation, or the migration of subsurface seepage. The disturbance and/or loosening of the surficial soils, as might result from root growth, soil expansion, or excavation for irrigation lines and slope planting, may also be a significant contributing factor to surficial instability. It is, therefore, recommended that, to the maximum extent practical: (a) disturbed/loosened surficial soils be either removed or properly recompacted, (b) irrigation systems be periodically inspected and maintained to eliminate leaks and excessive irrigation, and (c) surface drains on and adjacent to slopes be periodically maintained to preclude ponding or erosion. Although the incorporation of the above recommendations should reduce the potential for surficial slope instability, it will not eliminate the possibility, and, therefore, it may be necessary to rebuild or repair a portion of the project's slopes in the future.

7.9 Drainage

7.9.1 Adequate drainage provisions are imperative. Under no circumstances should water be allowed to pond adjacent to footings. The building pads should be properly finish graded after the buildings and other improvements are in place so that drainage water is directed away from foundations, pavements, concrete slabs, and slope tops to controlled drainage devices.

7.10 Grading Plan Review

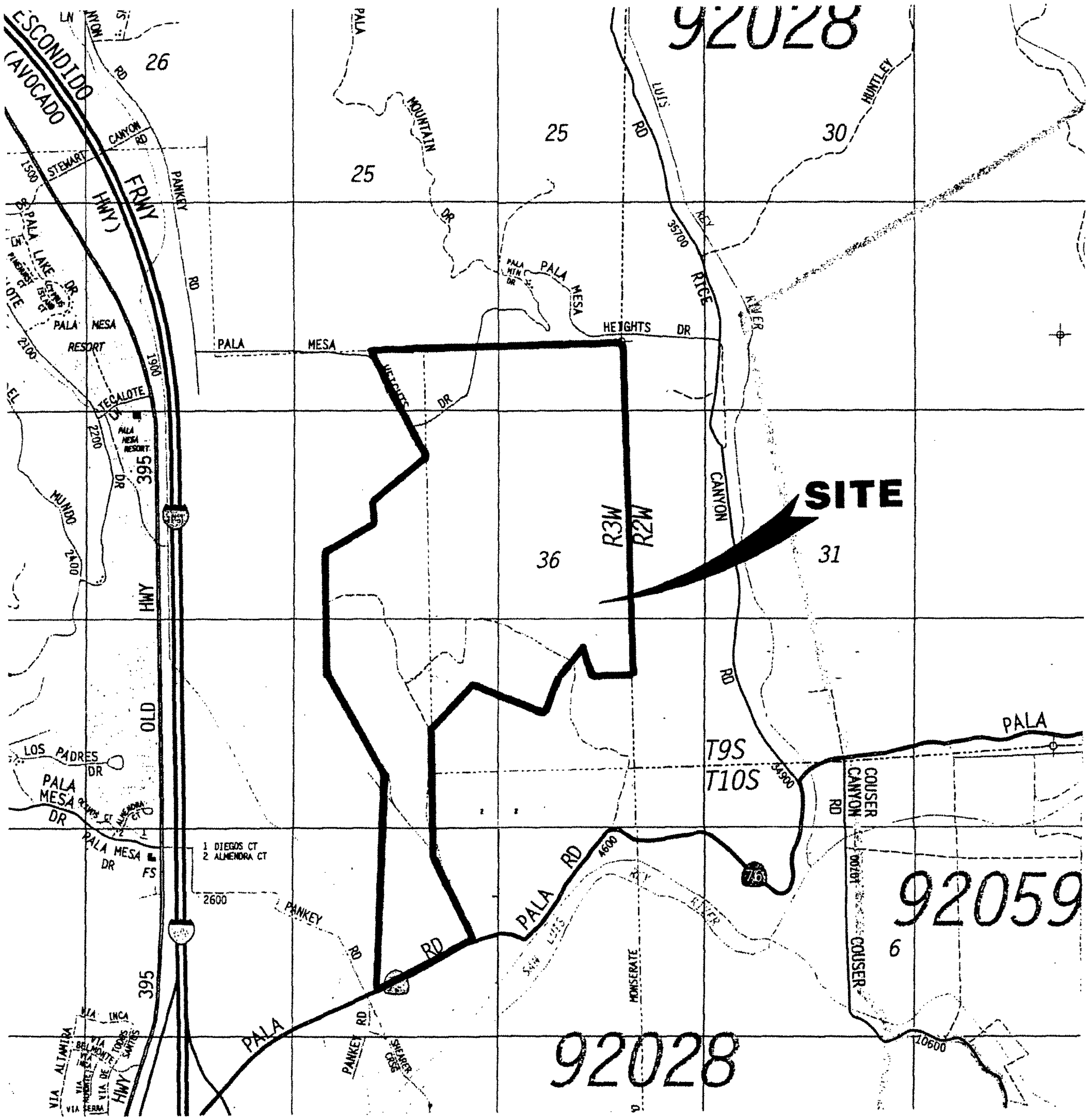
7.10.1. Geocon Incorporated should review the grading plans prior to finalization to verify their compliance with the recommendations of this report and determine the necessity for additional analyses and/or recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.

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

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.



SOURCE: 2002 THOMAS BROTHERS MAP
SAN DIEGO COUNTY, CALIFORNIA

N
NO SCALE

GEOCON
INCORPORATED
GEOTECHNICAL CONSULTANTS
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VICINITY MAP

MEADOWOOD
SAN DIEGO COUNTY, CALIFORNIA

JB / TA		DSK/GTYPD	DATE 11 - 20 - 2006	PROJECT NO. 06931 - 42 - 01	FIG. 1

Vicinity Map

LEGEND

- Qudf ........UNDOCUMENTED FILL
- Qudfr ........UNDOCUMENTED FILL RUBBLE (Mostly oversize concrete chunks)
- Qrf ....SUSPECTED ROCK FALL AREA
- Qal ....ALLUVIUM
- Qt ....TERRACE DEPOSIT
- Qtd ....LOCALIZED "HARDPAN" OR OVERSIZE
- Kb ....BONSALL TONALITE
- Kbg ....BONSALL TONALITE GNEISS
- Ksm ....SAN MARCOS GABBRO
- ....APPROX. LOCATION OF GEOLOGIC CONTACT
- ....APPROX. LOCATION OF SMALL-DIAMETER BORING (2006)
- ....APPROX. LOCATION OF "AIR-TRACK" BORING (2006)
- B-5 (9) ....APPROX. LOCATION OF SMALL-DIAMATER AUGER BORING (2002) (Remedial grading - feet)
- BA-3 ....APPROX. LOCATION OF PREVIOUS LARGE-DIAMETER BORING (2000) BY PACIFIC SOILS (Remedial grading - feet)
- T-19 (8) ....APPROX. LOCATION OF EXPLORATORY TRENCH (2002) (Remedial grading - feet)
- TP-5 ....APPROX. LOCATION OF PREVIOUS EXPLORATORY TRENCH (2000) BY PACIFIC SOILS (Remedial grading - feet)
- [>15] S-1 ....APPROX. LOCATION OF SEISMIC TRAVERSE (Depth to Non-Rippable - feet)
- [40-50] S-3 ....APPROX. LOCATION OF PREVIOUS SEISMIC TRAVERSE BY PACIFIC SOILS (Depth to Non-Rippable - feet)
- ....APPROX. LOCATION OF PROPOSED SUBDRAIN
- ....APPROX. LOCATION OF POTENTIALLY LIQUEFIABLE SOILS

GEOLOGIC MAP

MEADOWOOD

SAN DIEGO COUNTY, CALIFORNIA

GEOCON INCORPORATED
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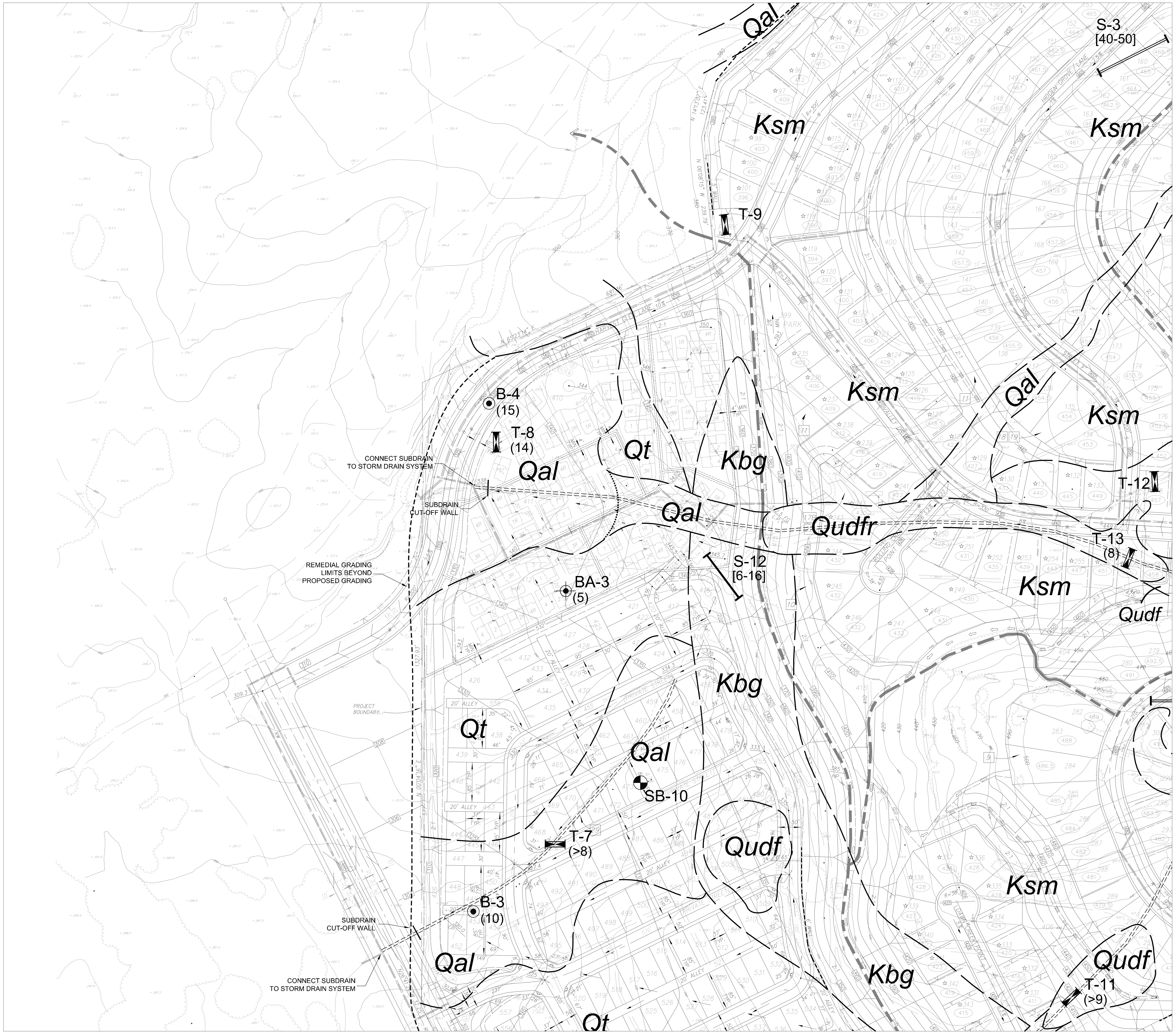
SCALE 1" = 100'	DATE 11 - 20 - 2006
PROJECT NO. 06931 - 42 - 01	FIGURE 2
SHEET 1 OF 5	

24X36.DWG/LADRILLONO



GRAPHIC SCALE

0' 50' 100' 150' 200' 300' 400'

SCALE 1" = 100'

GEOLOGIC MAP

MEADOWOOD

SAN DIEGO COUNTY, CALIFORNIA

GEOCON INCORPORATED

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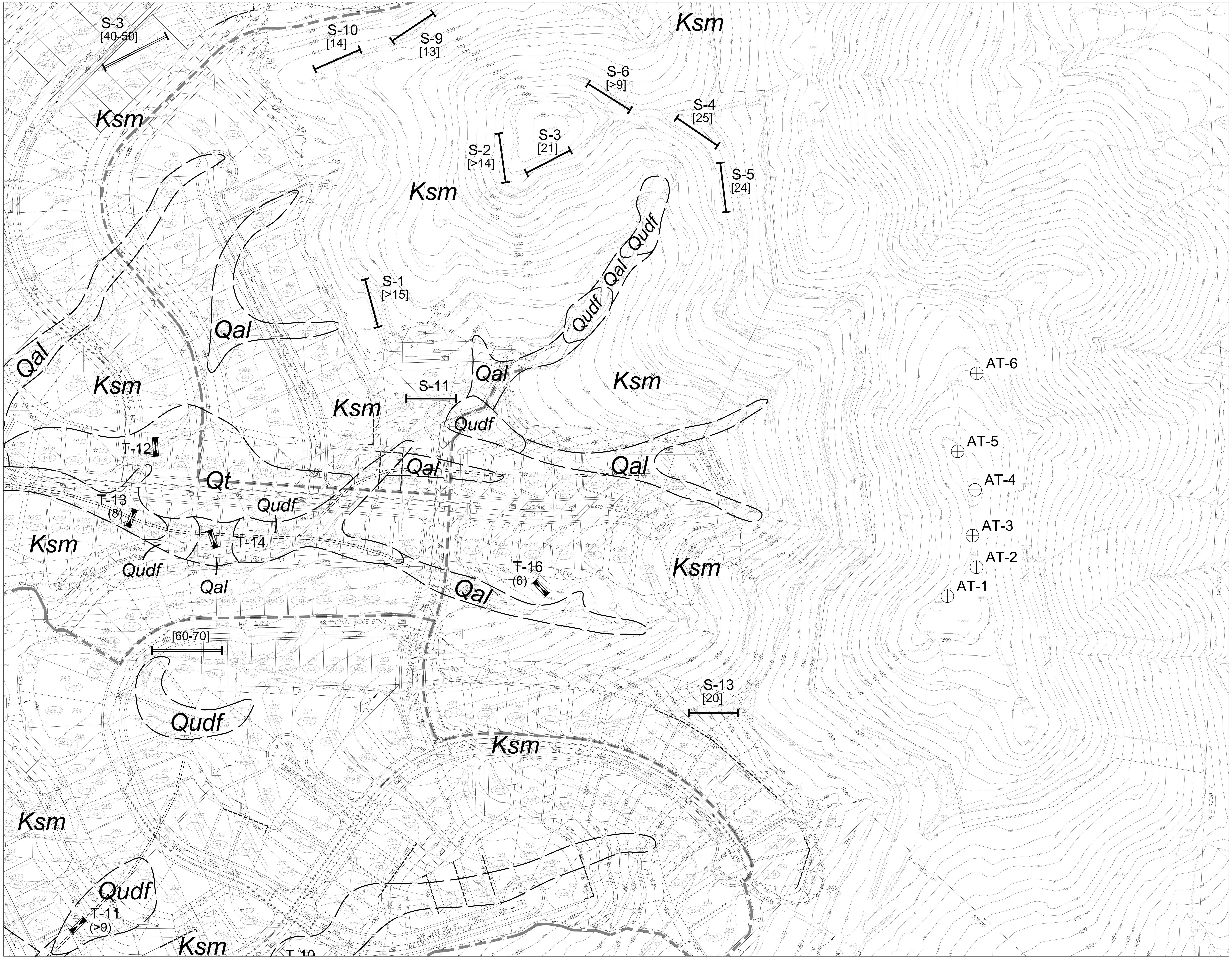
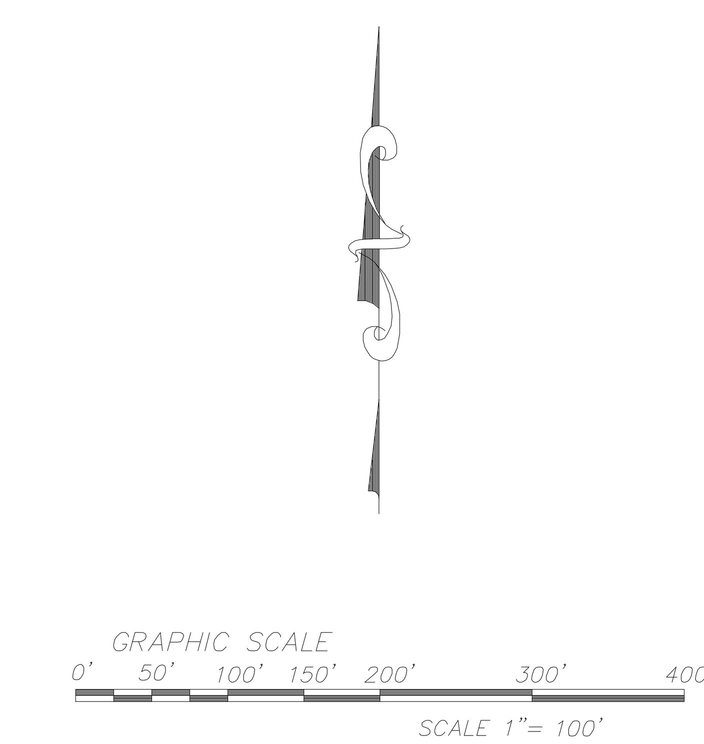
SCALE 1" = 100'	DATE 11 - 20 - 2006
PROJECT NO. 06931 - 42 - 01	FIGURE 3
SHEET 2 OF 5	

24X36.DWG/LADRILLONO





GEOLOGIC MAP

MEADOWOOD

SAN DIEGO COUNTY, CALIFORNIA

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SCALE 1" = 100'	DATE 11 - 20 - 2006
PROJECT NO. 06931 - 42 - 01	FIGURE 4
SHEET 3 OF 5	

GEOLOGIC MAP

MEADOWOOD

SAN DIEGO COUNTY, CALIFORNIA

GEOCON INCORPORATED

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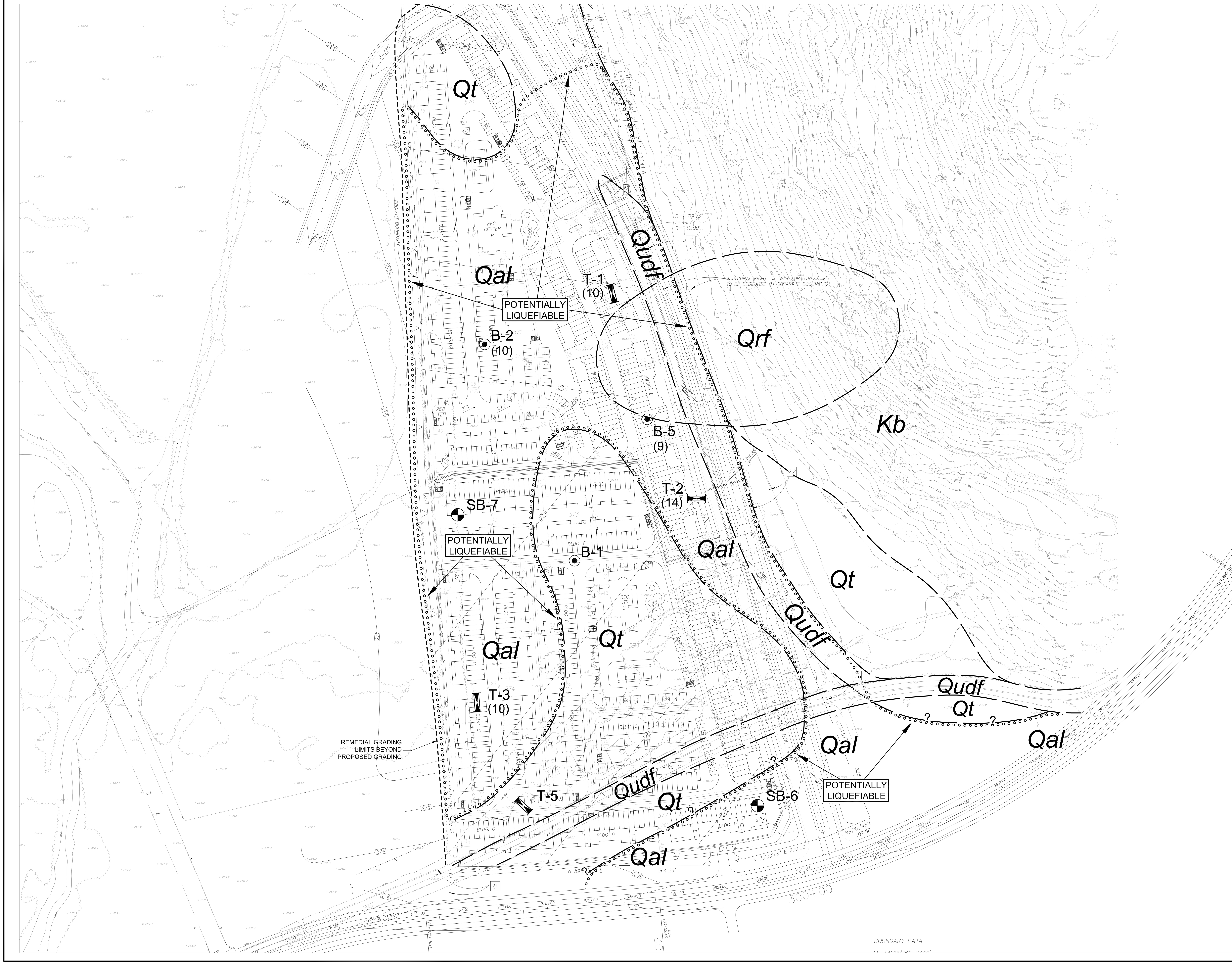
SCALE	DATE	
1" = 100'	11 - 20 - 2006	
PROJECT NO. 06931 - 42 - 01		FIGURE 5
SHEET 4 OF 5		

24X36.DWG/LADRILLONO



GEOLOGIC MAP

MEADOWOOD

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SCALE 1" = 100'	DATE 11 - 20 - 2006	FIGURE
PROJECT NO. 06931 - 42 - 01		6
SHEET 5 OF 5		

24X36.DWG/LADRILLONO

ASSUMED CONDITIONS :

SLOPE HEIGHT	H = 100 feet
SLOPE INCLINATION	2 : 1 (Horizontal : Vertical)
TOTAL UNIT WEIGHT OF SOIL	γ_t = 150 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ = 40 degrees
APPARENT COHESION	C = 500 pounds per square foot
NO SEEPAGE FORCES	

ANALYSIS :

$\gamma_{c\phi}$	$= \dfrac{\gamma H \tan\phi}{C}$	EQUATION (3-3), REFERENCE 1
FS	$= \dfrac{N_{cf} C}{\gamma H}$	EQUATION (3-2), REFERENCE 1
$\gamma_{c\phi}$	= 25	CALCULATED USING EQ. (3-3)
Ncf	= 62	DETERMINED USING FIGURE 10, REFERENCE 2
FS	= 2.0	FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES :

1......Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics, Series No. 46, 1954

2......Janbu, N., Discussion of J.M. Bell, Dimensionless Parameters for Homogeneous Earth Slopes, Journal of Soil Mechanics and Foundation Design, No. SM6, November 1967.

SLOPE STABILITY ANALYSIS - CUT SLOPES (2:1)

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ASSUMED CONDITIONS :

SLOPE HEIGHT	H = 100 feet
SLOPE INCLINATION	2 : 1 (Horizontal : Vertical)
TOTAL UNIT WEIGHT OF SOIL	γ_t = 130 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ = 30 degrees
APPARENT COHESION	C = 500 pounds per square foot
NO SEEPAGE FORCES	

ANALYSIS :

$\gamma_{c\phi} = \dfrac{\gamma H \tan\phi}{C}$	EQUATION (3-3), REFERENCE 1
$FS = \dfrac{N_{cf}C}{\gamma H}$	EQUATION (3-2), REFERENCE 1
$\gamma_{c\phi} = 15$	CALCULATED USING EQ. (3-3)
$N_{cf} = 45$	DETERMINED USING FIGURE 10, REFERENCE 2
$FS = 1.7$	FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES :

1......Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics, Series No. 46, 1954

2......Janbu, N., Discussion of J.M. Bell, Dimensionless Parameters for Homogeneous Earth Slopes, Journal of Soil Mechanics and Foundation Design, No. SM6, November 1967.

SLOPE STABILITY ANALYSIS - FILL SLOPES

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Y:/I_DETAIL/SSAF/DWG.

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= 100 feet
SLOPE INCLINATION	1.5 : 1	(Horizontal : Vertical)
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 150 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 40 degrees
APPARENT COHESION	C	= 500 pounds per square foot
NO SEEPAGE FORCES		

ANALYSIS :

$\gamma_{c\phi}$	=	$\dfrac{\gamma H \tan\phi}{C}$	EQUATION (3-3), REFERENCE 1
FS	=	$\dfrac{NcfC}{\gamma H}$	EQUATION (3-2), REFERENCE 1
$\gamma_{c\phi}$	=	25	CALCULATED USING EQ. (3-3)
Ncf	=	50	DETERMINED USING FIGURE 10, REFERENCE 2
FS	=	1.6	FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES :

1......Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics, Series No. 46, 1954

2......Janbu, N., Discussion of J.M. Bell, Dimensionless Parameters for Homogeneous Earth Slopes, Journal of Soil Mechanics and Foundation Design, No. SM6, November 1967.

SLOPE STABILITY ANALYSIS - CUT SLOPES (1.5:1)

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ASSUMED CONDITIONS :

SLOPE HEIGHT	H	=	Infinite
DEPTH OF SATURATION	Z	=	3 feet
SLOPE INCLINATION	2 : 1 (Horizontal : Vertical)		
SLOPE ANGLE	i	=	26.5 degrees
UNIT WEIGHT OF WATER	γ_w	=	62.4 pounds per cubic foot
TOTAL UNIT WEIGHT OF SOIL	γ_t	=	130 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	=	30 degrees
APPARENT COHESION	C	=	500 pounds per square foot

SLOPE SATURATED TO VERTICAL DEPTH Z BELOW SLOPE FACE

SEEPAGE FORCES PARALLEL TO SLOPE FACE

ANALYSIS :

$$\mathrm{FS} = \frac{C + (\gamma_t - \gamma_w)\ Z \cos^2 i \tan \phi}{\gamma_t\ Z \sin i \cos i} = 3.8$$

REFERENCES :

1......Haefeli, R. *The Stability of Slopes Acted Upon by Parallel Seepage* , Proc. Second International Conference, SMFE, Rotterdam, 1948, 1, 57-62

2......Skempton, A. W., and F.A. Delory, *Stability of Natural Slopes in London Clay* , Proc. Fourth International Conference, SMFE, London, 1957, 2, 378-81

NO SCALE

SURFICIAL SLOPE STABILITY ANALYSIS

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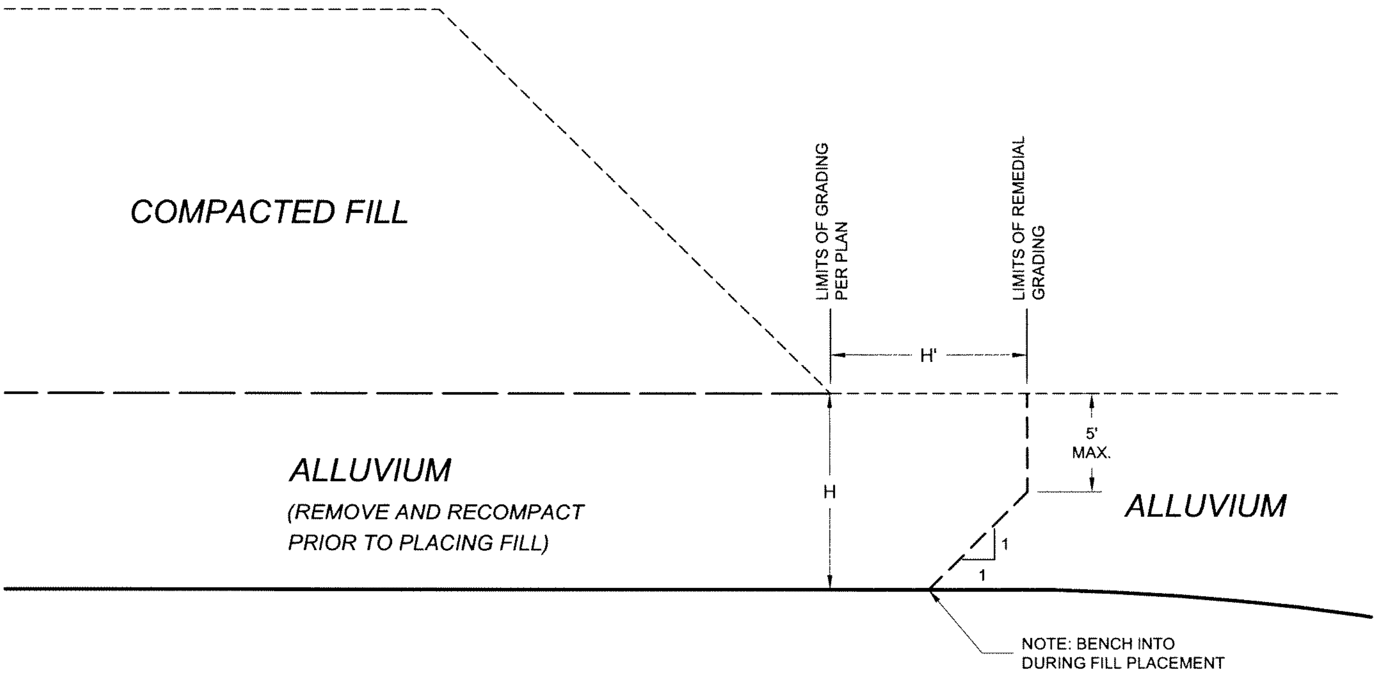

H = THICHNESS OF REMOVAL

H' = HORIZONTAL DISTANCE BEYOND
GRADING PER PLAN FOR REMEDIAL GRADING
H = H'

NOTE: WHERE H' IS LIMITED BY PROPERTY BOUNDARIES OR OTHER RESTRICTIONS A SETBACK EQUAL TO THE DISTANCE OF THE RESTRICTIONS TO H' MAY BE REQUIRED.

TYPICAL DETAIL FOR LIMITS OF REMEDIAL GRADING

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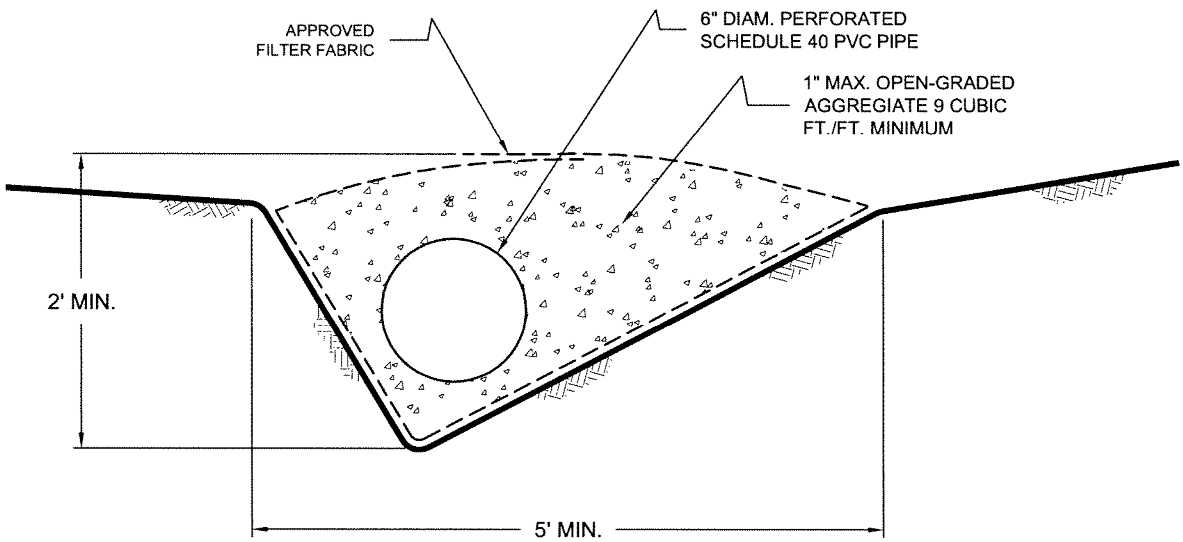
DG / RA		DSK/E0000	DATE 11 - 20 - 2006	PROJECT NO. 06931 - 42 - 01	FIG. 12

E:/GEOTECH/TYP/TDLRG.DWG



NOTES:

1.....SUBDRAIN PIPE SHOULD BE 6-INCH MINIMUM DIAMETER, PERFORATED, THICK WALLED SCHEDULE 40 PVC, SLOPED TO DRAIN AT 1 PERCENT MINIMUM AND CONNECTED TO STORM DRAIN SYSTEM OR APPROVED OUTLET.

2.....WHERE DRAIN EXCEEDS 1,000 FEET IN LENGTH, THE DOWNSTREAM SECTION GREATER THAN 1,000 FEET SHOULD BE INCREASED TO 8 INCHES.

3.....FILTER FABRIC TO BE MIRAFI 140N OR EQUIVALENT.

TYPICAL CANYON SUBDRAIN DETAIL

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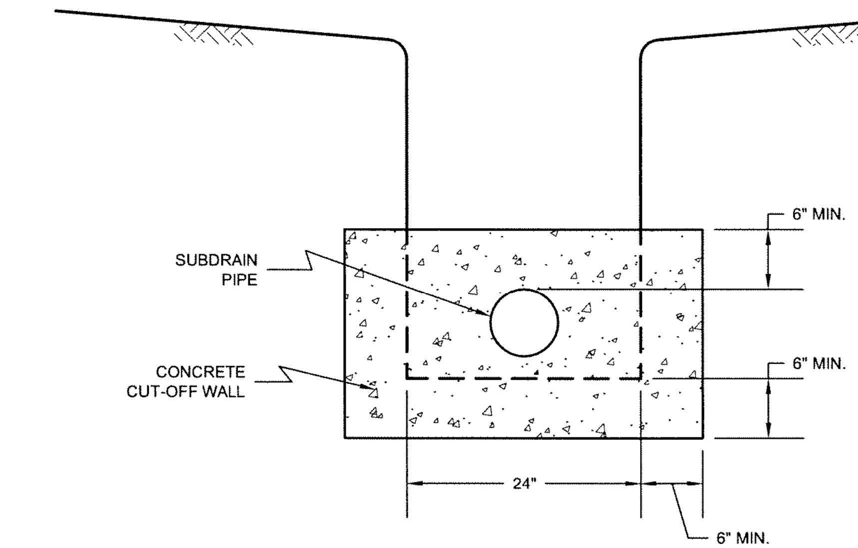
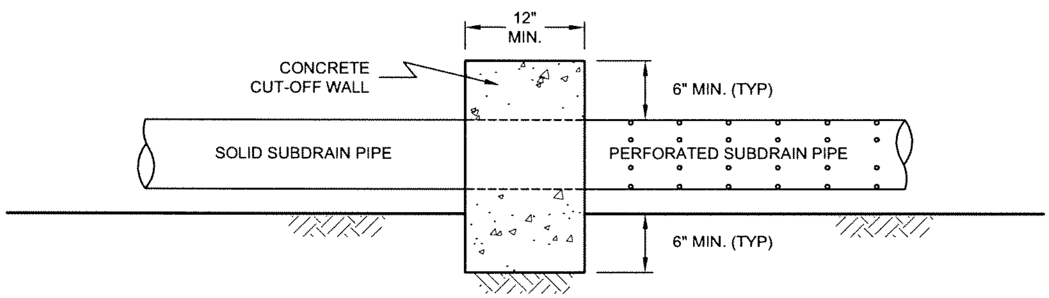
X:/R14TEMP/1_AUTOCAD PLATE TEMPLATE/1_Detail/TCSD

FRONT VIEW



NO SCALE

SIDE VIEW



NO SCALE

TYPICAL SUBDRAIN CUT-OFF WALL DETAIL

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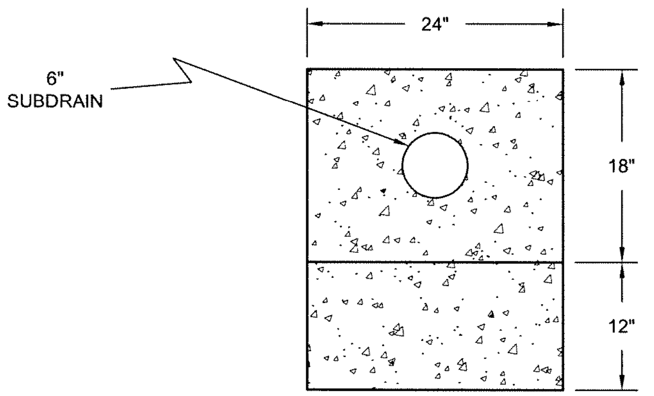
JJB / RA		DSK/GTYPD	DATE 11 - 20 - 2006	PROJECT NO. 06931 - 42 - 01	FIG. 14

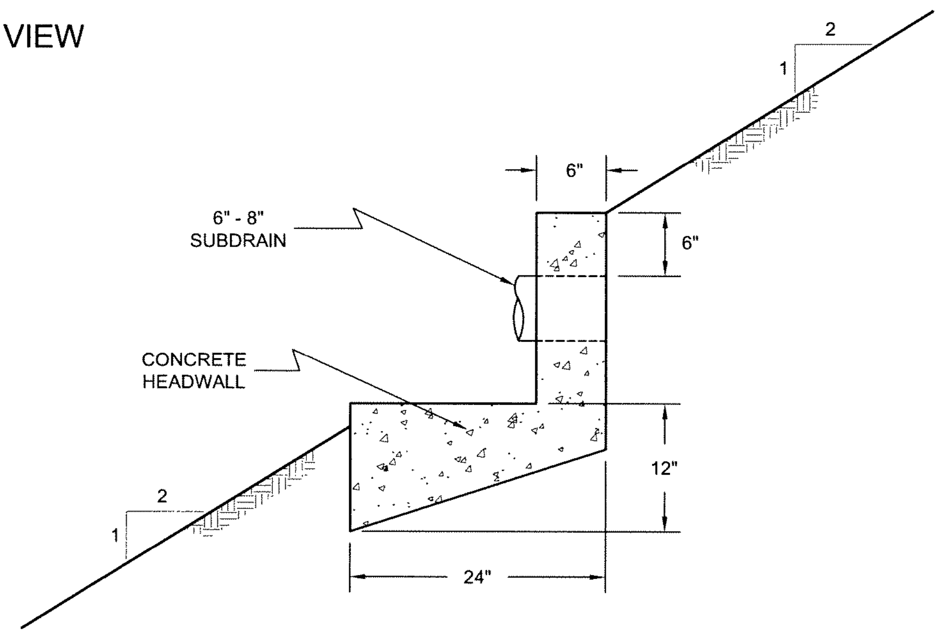
X:/R14TEMP/1_AUTOCAD PLATE TEMPLATE/1_DETAIL/RSCOW

FRONT VIEW



NO SCALE

SIDE VIEW



NOTE: HEADWALL SHOULD OUTLET INTO CONTROLLED SURFACE DRAINAGE

NO SCALE

TYPICAL SUBDRAIN OUTLET HEADWALL DETAIL

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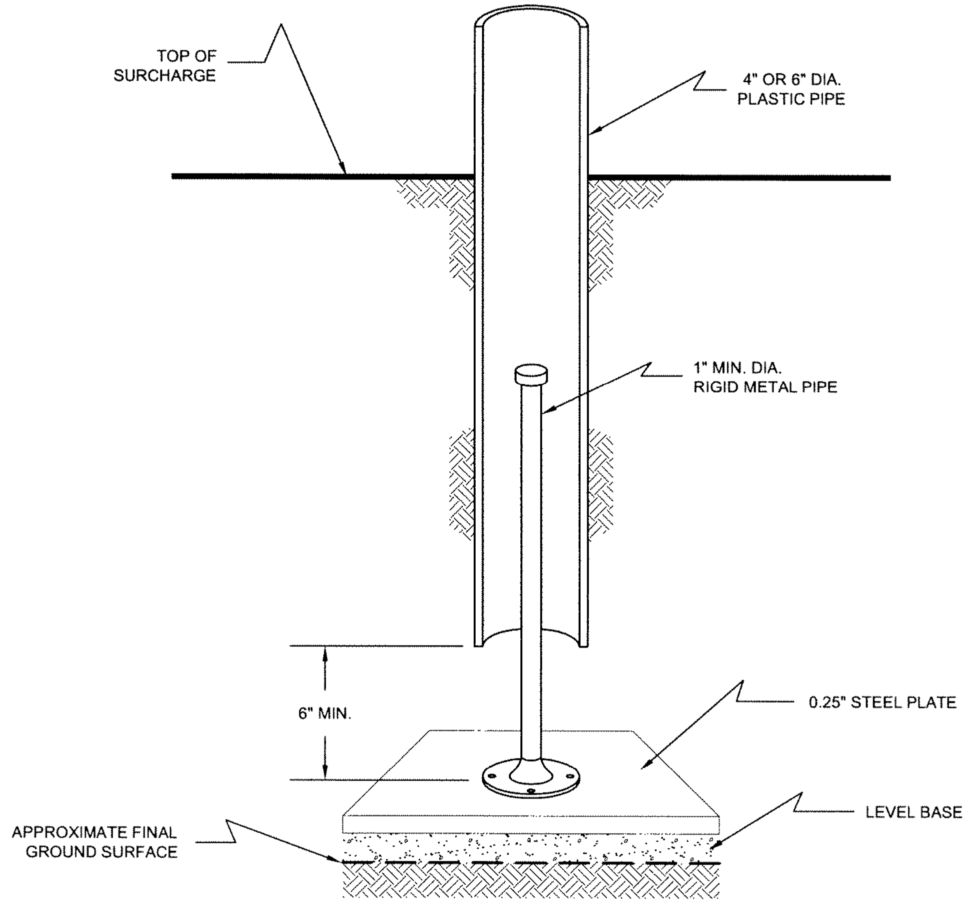
X:/R14TEMP/1_AUTOCAD PLATE TEMPLATE/1_DETAIL/SOHD



NOTES:

1......LOCATION OF SETTLEMENT PLATES SHALL BE CLEARLY MARKED AND READILY VISIBLE (RED FLAG) TO EQUIPMENT OPERATORS.

2......CONTRACTOR SHALL MAINTAIN 10-FOOT HORIZONTAL CLEARANCE FOR HEAVY EQUIPMENT WITHIN 5 FEET (VERTICAL) OF PLATE BASE. FILL WITHIN CLEARANCE AREA SHALL BE HAND COMPACTED TO PROJECT SPECIFICATIONS OR COMPACTED BY ALTERNATIVE APPROVED SOILS ENGINEER.

3......AFTER 5 FEET (VERTICAL) OF FILL IS IN PLACE, THE CONTRACTOR SHALL MAINTAIN 5 FEET HORIZONTAL EQUIPMENT CLEARANCE. FILL IN CLEARANCE AREA SHALL BE HAND COMPACTED (OR APPROVED ALTERNATIVE) IN VERTICAL INCREMENTS NOT TO EXCEED 2 FEET.

4......IN THE EVENT OF DAMAGE TO SETTLEMENT PLATE OR EXTENSION RESULTING FROM EQUIPMENT OPERATING WITHIN PRESCRIBED CLEARANCE AREA, CONTRACTORS SHALL IMMEDIATELY NOTIFY SOILS ENGINEER AND SHALL BE RESPONSIBLE FOR RESTORING THE SETTLEMENT PLATES TO WORKING ORDER.

NO SCALE

SETTLEMENT MONUMENT

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X:/R14TEMP/1_AUTOCAD PLATE TEMPLATE/1_DETAIL/STLMNT

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