F I N A L   R E P O R T

GEOTECHNICAL EVALUATIONS EXISTING WATER STORAGE RESERVOIR PROPOSED SHADOW RUN RANCH RESIDENTIAL DEVELOPMENT PAUMA VALLEY, NORTH SAN DIEGO COUNTY, CALIFORNIA

Prepared for

Ms. Sherrill Schoepe
Shadow Run Ranch, LLC

URS Project No. 27661027.10000

November 25, 2013
Revised December 31, 2013

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Ms. Sherrill Shoepe
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c/o: Ron Deutschendorf
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Subject: Geotechnical Evaluation
Existing Water Storage Reservoir
Proposed Shadow Run Ranch Residential Development
Pauma Valley, North San Diego County, California
URS Project No. 27661027.00001

Dear Ms. Schoepe:

This letter transmits URS Corporation's (URS) geotechnical evaluation for the existing water storage reservoir at the proposed Shadow Run Ranch residential development, Pauma Valley, North San Diego County, California. This study was performed in accordance with our Work Plan, dated August 22, 2013.

The County of San Diego had requested additional investigation to evaluate the stability of the existing earth dam (embankment) at the site for their permitting process. These additional geotechnical evaluations were completed utilizing site specific information developed from geotechnical test borings, laboratory testing, and stability analyses. The evaluations indicate that the earthen embankment will provide containment of the reservoir for static and seismic conditions

This final report has been revised to address comments from the County's consultant (GEI Consultants) dated December 4, 2013. If you have any questions regarding this report, or if we can be of further service, please give us a call.

Sincerely,

URS CORPORATION

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ASTM	ASTM International
CDMG	California Division of Mines and Geology
CGS	California Geological Survey
D.G.	Decomposed Granite
FOS	Factor of Safety
H:V	horizontal to vertical ratio
MSL	Mean Sea Level
NCEER	National Center for Earthquake Engineering Research
pcf	pounds per cubic foot
PGA	Peak Ground Acceleration
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
psf	pounds per square foot
Qaf	Artificial Fill
SPT	Standard Penetration Test
URS	URS Group
USCS	United Soil Classification System
USGS	United States Geological Survey
Wxd	Weathered

SECTION 1 INTRODUCTION

This report presents results of a geotechnical investigation and evaluations by URS Corporation (URS) that were performed for the existing private water storage reservoir at the proposed Shadow Run Ranch residential development, located in Pauma Valley in North San Diego County, California. The project location is shown on Figure 1.

The existing reservoir is approximately 3 acres in surface area and located near and east of Highway 76, about 10 miles east of Interstate 15, near the intersection of Highway 76 and Adams Drive (Figure 2). The reservoir is filled using groundwater pumped from nearby wells on the property. The reservoir provides a water supply for agricultural purposes (to irrigate citrus and avocado orchards on the property) and has been used for many years. The land below the reservoir is proposed for development as single family residences. The existing reservoir will continue to be used for agricultural purposes.

According to the State of California (State) Division of Safety of Dams (DSOD), the reservoir is not subject to State jurisdiction for safety (refer to DSOD letter dated March 19, 2013). The County of San Diego (County) has requested investigation and analyses to evaluate the stability of the existing earth embankment forming the reservoir. The geotechnical evaluations were performed using the results of subsurface explorations, geotechnical laboratory testing, and slope stability analyses, as outlined in the Work Plan prepared by URS, dated August 22, 2013.

1.1 PURPOSE OF EVALUATIONS

The County requested additional information to adequately demonstrate the embankment dam is stable. More specifically, the County has requested an evaluation of:

- Embankment material type and compaction.
- Embankment foundation subsurface conditions, e.g., the thickness of fill soils, and whether or not unsuitable material including boulders and loose rock were removed during construction.
- Central cutoff trench or impervious core for the dam section, if present.

The purpose of the geotechnical investigation was to address the above issues as described further in this report.

1.2 APPROACH FOR EVALUATIONS

Geotechnical borings were performed to provide subsurface information to characterize the embankment and underlying natural geologic materials forming the foundation of the embankment. The subsurface information was used to develop cross sections of the embankment and foundation materials that were used for slope stability analyses.

1.2.1 Subsurface Explorations

Geotechnical borings were advanced at five locations along the crest of the embankment. It had been anticipated that the embankment and foundation may include coarse materials (gravel, cobble and boulder size), so a Becker Hammer drill rig was mobilized for drilling the borings. The Becker Hammer rig is designed to characterize gravelly materials and uses a “closed bit” method (which is somewhat like a pile driver), which does not recover any sample materials. Furthermore, empirical methods using Becker Penetration Test (BPT) resistance data have been developed for use in evaluating the potential for gravelly materials to liquefy during an earthquake (Harder and Seed, 1986). It was intended to use the BPT data in evaluating the potential for the embankment or foundation materials to liquefy. However, the embankment materials encountered during the investigation were comprised of clayey sand with only sparse gravel-sized material. The relatively fine-grained nature of the embankment materials allowed the borings to be advanced “open bit”, which does not provide the BPT data, but does allow observing and sampling drill cuttings returned to the surface. The open bit mode also allowed obtaining relatively undisturbed soil samples with a Standard Penetration Test (SPT) split spoon sampler and a modified California soil sampler. Below the embankment, the borings penetrated weathered granitic rock; rock coring was not required. Disturbed soil samples were collected at regular intervals to the total depth of the borings.

The Becker Hammer borings and undisturbed samples collected from the borings provided data to evaluate the relative density of the embankment fill materials. Although there are no compaction records from construction of the embankment available, the relative density of the embankment materials was evaluated based on the BPT (open bit) data and soil sampler penetration resistance (N-values from the SPT and modified California samples) and the in-situ density of the materials, as determined by laboratory testing.

Before drilling the borings, shallow test pits were excavated to maximum depths of about 14 below the ground surface (bgs) with a heavy duty backhoe. The test pits were located along the crest of the embankment and near the toe of the downstream slope. The test pits were excavated near the proposed boring locations on the crest of the dam. The final locations of the test pits near the downstream toe were adjusted in the field as necessary to avoid stockpiled boulders.

The borings and test pits were located at various locations spanning the width of the access road at the crest of the embankment to confirm whether or not a core trench or impervious core is present. As described below, the dam was constructed as a homogeneous earth dam comprised of a relatively uniform embankment material (i.e, clayey sand commonly described as decomposed granite or “DG”).

1.2.2 Stability Analyses

The results of the field exploration and laboratory testing were used to perform engineering analyses to evaluate the dam embankment and foundation condition and their stability. The stability of the embankment dam was evaluated by performing static and pseudo-static slope stability analyses. The values of Peak Ground Acceleration (PGA) for the pseudo-static analyses were based on a deterministic seismic shaking analysis assuming the Maximum Credible Earthquake (MCE) occurs on the nearest reach of the Elsinore fault, with a seismic reduction factor based on current engineering practice. Limited seismic deformation analyses were also performed.

SECTION 2 GEOTECHNICAL INVESTIGATION

The current geotechnical investigation of the existing dam and reservoir at Shadow Ranch included geologic reconnaissance, subsurface explorations and laboratory testing. Previous investigations were also performed by URS. The previous investigations and the current investigation are summarized in the following sections.

2.1 PREVIOUS INVESTIGATIONS

URS previously performed a fault hazard investigation of the Elsinore Fault within the Alquist-Priolo Earthquake Fault Zone, as mapped on the property by the State of California, 1980 (Figure 2). For that study, exploratory trenches were excavated to document the location of the Elsinore Fault, and to provide fault set-back recommendations for the proposed residential development. Based on the initial fault study (URS, 2001), a branch fault was suspected to project near the existing water storage reservoir. A suspected fault had also been mapped between the existing reservoir and a nearby hillside. Additional fault trenches were excavated to confirm the absence of faulting in the vicinity of the reservoir (URS, 2009). Locations of previous exploratory trenches are shown on Figure 2.

2.2 CURRENT INVESTIGATION

The field explorations were supervised by URS engineering geologists. Boring permits were obtained from the County of San Diego Department of Environmental Health (DEH). The exploration elevations were estimated using available site topography; hand held GPS was also used to determine the approximate locations of the subsurface explorations.

2.2.1 Subsurface Explorations

The subsurface explorations included advancing five (5) test borings (designated Borings B-1, -2, -2A, -3 and -4) with a truck-mounted Becker Hammer rig capable of closed or open bit drilling, SPT sampling and rock coring, if required. The borings were located along the existing vehicle access road around the crest of the embankment and reservoir. Eleven test pits (designated Test Pits TP-1 through TP-11) were excavated with a rubber tired back-hoe. Figure 3 provides the locations of the borings and test pits. Logs of the test borings and test pits are presented in Appendix A as Figures A-2 through A-13. A Key to Logs is presented as Figure A-1 in Appendix A. Photographs of the field explorations and representative samples are also included in Appendix A.

The borings were advanced to a minimum of 10 feet below the bottom of the embankment, as determined by the URS geologist during drilling. Table 1 provides the locations and drilling depths of the borings and the method of drilling and sampling. All of the borings were drilled using the Becker Hammer rig in the open bit mode. BPT (open bit) advancement rates were recorded by the URS geologist onsite. In the open bit mode, samples using an SPT sampler and a modified California sampler were obtained alternating at approximately 5 foot depth intervals. Disturbed samples were also collected from the cuttings generated from each advancement of the BPT open bit (typically 5 foot drives).

Three of the borings (Borings B-1, B-2A and B-4) were constructed as piezometers (stand pipe type monitoring wells). Well construction consisted of 2-inch diameter blank PVC pipe with a length of slotted screen placed near the bottom of the hole, then backfilled with sand and a bentonite seal in accordance with County of San Diego DEH requirements. A locking cap was installed on the top of each well and a flush-mount traffic cover was constructed at the ground surface. The remaining borings were tremie backfilled with bentonite cement. Drill cuttings were spread and disposed of onsite. Groundwater levels were measured periodically for the duration of the geotechnical study, the most recent reading (December 20, 2013) is shown on the boring log.

The test pits were located along the crest and downstream toe of the embankment, and one at the proposed location of the revised spillway (Test Pit TP-8, see Figure 3). The test pits were excavated with a rubber tired backhoe with a 24-inch wide bucket. Disturbed samples and/or hand held drive samples were collected from the sidewalls of the test pits. The maximum depths of the test pits was about 14.5 feet below ground surface (see Table 2). The test pits were backfilled with the excavated materials, which were nominally recompacted with the backhoe.

2.2.2 Soil Sampling

Soil samples were obtained alternating between a modified California drive sampler (2.5-inch inside diameter) with a stainless steel liner, and a SPT sampler (1.375-inch inside diameter and 2-inch outside diameter) in general accordance with ASTM D-1586. Both samplers were driven 18-inches or until sampler refusal into material at the bottom of the boring by a 140-pound hammer falling 30-inches. Materials encountered in the borings and test pits were classified in accordance with the Unified Soil Classification System (USCS).

2.2.3 Laboratory Testing

The samples obtained from the borings and test pits were sealed and transported to URS' geotechnical laboratory for further examination and testing. The testing included: moisture content, bulk density, particle size analyses and compaction tests to characterize and classify the soils. Direct shear tests were performed to evaluate strength characteristics of the soil or rock materials. Atterberg limit tests were performed on representative samples of the fill soil. The laboratory testing was completed according to ASTM testing standards. Results of the lab tests are presented in Appendix B.

Pinhole testing (ASTM D4647) was performed on three representative samples of the fill materials. The pinhole test provides a measure to evaluate the potential for clayey soils to erode by flowing water (piping). Results of the pinhole tests are presented in Appendix B, and are discussed below.

SECTION 3 SITE CONDITIONS

Knowledge of the site conditions at the Shadow Run Ranch embankment dam was developed from a review of the area geology, site reconnaissance, a review of previous investigations, and the results of the current geotechnical investigation.

3.1 SITE GEOLOGY

Cretaceous and Jurassic granitic and metamorphic rocks underlie the natural hillside immediately northeast of the reservoir (Kennedy, 2000). The mesa below the reservoir has been mapped as Quaternary older (alluvial) fan deposits (Vaughn, 1987; Kennedy, 2000). Subsurface explorations performed for this investigation indicate that the embankment is also underlain by granitic rock. The granitic rock was highly to completely weathered. Prior to reservoir construction, alluvial fan deposits (as previously mapped) may have been present as a fairly thin layer (if present at all). Excavations to construct the reservoir may have removed any alluvial fan deposits exposing the underlying granitic rock. Alluvial fan deposits appear to underlie only the northwest part of the reservoir, which was formed mostly by excavating into the natural materials, not building an embankment. The alluvial fan deposits appear to thicken downslope from the reservoir, based on previous trenches (URS, 2009).

Cobbles/boulders of less weathered rock ("boulders of decomposition") were encountered in the explorations within deeply weathered rock. Thin topsoil and colluvium (soil developed on a slope) overlies the weathered granitic rock near the downstream toe, based on the test pits at the slope toe.

3.2 RESERVOIR

The reservoir storage capacity is up to approximately 42 acre-feet. The water level in the reservoir is maintained at elevation 1,085.4 feet mean sea level (MSL) by spilling through a 24-inch diameter outlet pipe on the west side of the reservoir. A new spillway is proposed at elevation 1,082.6 feet MSL which will reduce the reservoir storage capacity to approximately 34.5 acre-feet. The currently proposed spillway is near the location of Boring B-1[1] (see Figure 3).

The reservoir is situated near the base of a northwest trending ridge (Figure 2). The reservoir is not located across a stream channel or natural drainage. The natural drainage course of Frey Creek is several hundred feet north; the creek channel is about 80 feet lower than the reservoir (at its closest point). Runoff from the adjacent hillside is diverted via ditches from entering the reservoir. Therefore, there is little to no natural runoff to the reservoir. The natural terrain bordering the reservoir slopes down to the southwest and south at gentle slope inclinations. The terrain on the northwest side is mostly flat. The site topography is shown in detail in Figure 3.

There are no grading plans or construction reports available for the reservoir embankments. According to the property owner, construction of the reservoir was completed in the early 1960s. The general area of the excavation is shown surrounding a small pond on the 1956 County of San Diego topographic map (original scale 1"=200'), shown on Figure 4. By about 1964, the reservoir had been filled to near its

[1] It is recommended that the spillway be moved east to be in natural ground, as discussed below.

capacity (Figure 5). Older historical aerial photos dated 1946 and 1953 were also obtained (online at http://www.historicaerials.com). These photos (see Figure 5) show initial stages of grading prior to filling the reservoir. Based on a review of these photos:

- The initial excavations for the reservoir are apparent in the 1946 photo. A semi-circular excavation had been made creating west and north facing cut slopes. Material has been stockpiled in the middle of the reservoir area.

- In the 1953 photo, the same general outline of excavated area is apparent, with a berm of fill placed on the east and southeast sides. The north and west parts of the excavation appear to be in cut. It is estimated that the deepest part of the excavation was up to about 15 feet, based on the undisturbed topography shown in Figure 4. The excavated materials appear to have been placed around the margins of the excavation to create an earth fill embankment on the south side of the reservoir. The top of the berm appears to have been wide enough for vehicles. The berm filled in a former drainage swale along and below a dirt road (along the east side). A small amount of water has ponded in the lowest area.

- By 1964, the reservoir has been filled with water and is mostly the same form and shape as the current reservoir. The fill berm/embankment appears to have been moved (re-graded) further south, enlarging the reservoir area. A fresh-appearing excavation is evident on the hillside to the northeast of the reservoir, which may have been the source of some of the soil used for embankment fill. Boulders have been placed around the downstream margin of the reservoir, probably for erosion protection.

A layer of bentonite had been placed along the bottom of the reservoir to improve water retention, according to the property owner. Bottom soundings with storage volumes were prepared in 1998 (see Figure 9 of Work Plan). The bottom soundings were compared with the circa 1998 pre-grading topographic contours (Figure 6 of Work Plan) and the available site topographic map (2-foot contour intervals) to help prepare cross sections of the embankment as shown on Figures 6, 7 and 8.

The dam embankment slopes are inclined at about 2.5 to 1 and 3 to 1 (horizontal to vertical), and are up to about 25 feet high. The top of the dam embankment provides an access road between 13 and 20 feet wide. Mature palm trees line the access road; their root systems appear to be fairly shallow (see Appendix A photos).

3.3 SUBSURFACE CONDITIONS

Interpreted subsurface conditions along four sections oriented transverse to the dam embankment (Sections A-A’, B-B’, C-C’, D-D’ and E-E’) are shown on Figures 6, 7 and 8. Appendix A provides detailed soil and rock descriptions.

3.3.1 Embankment Fill

All five borings along the top of the embankment penetrated fill soil. The thickness of the fill ranged between about 18.5 feet bgs in Boring B-4 to about 31 feet bgs in Boring B-1. The fill soil was described as reddish brown to grey, clayey sand with a few gravel-sized rock fragments (i.e., DG type material).

Sampler penetration resistances (SPT and Mod Cal samplers) ranged between 25 and 53 blows per foot. Beneath the embankment fill, the borings penetrated weathered and decomposed granitic rock, as described below.

The embankment fill was also observed and logged in the test pits excavated along the crest of the embankment (Test Pits TP-1 through -4, and -8 and -9, see Figure 3). Those test pits were excavated to depths up to about 14.5 feet bgs with no significant sidewall instability. A geologist was able to enter the pits up to a depth of about 5 feet bgs and scrape the sidewalls smooth; it was apparent from the stratification observed that the embankment fill had been placed in layers between about 6 and 8 inches thick. The fill appears to have been compacted at least to the total depth of the pits, judging from the sidewall stability and soil behavior. The fill material observed in the test pits was relatively uniform clayey sand with gravel, i.e., DG type-material. The fill material comprising the embankment appears to have been screened to remove large particles and was likely imported from offsite. No large rock fragments or boulders were observed in the fill.

Test Pits TP-5, -6, -7 and -11 (Figure 3) exposed a thin wedge of clayey sand fill with some cobbles at the toe of the embankment slope. These DG type materials (apparently without screening cobbles) appeared to be used in the lower portion of the fill slope.

The test pits were located across the width of the embankment crest to investigate whether or not a central cutoff trench or impervious core had been constructed for the dam section. Based on the test pits, the dam section does not include a central cutoff trench or impervious core. The dam can be described as a homogeneous earth dam constructed of a relatively uniform embankment fill material (i.e., DG soil).

3.3.2 Topsoil/Colluvium

Test Pits TP-5 through -7, and TP-10 and -11 were located at the downstream embankment slope toe. The test pits at these locations were typically excavated through shallow embankment fill into topsoil/colluvium consisting of grey brown silty sand with granitic cobbles and boulders. The natural appearing loamy soil was up to about 2 to 3 feet thick and was apparently not removed from the below the fill at the embankment slope toe. In preparing the subsurface cross sections, it was assumed that a similar thickness of topsoil/colluvium (as observed in the test pits at the slope toe) underlies the lower portion of the embankment, down slope of the crest. Topsoil/Colluvium was not encountered in the borings along the crest. Considering the previous topography and the estimated depth of the cut, topsoil/colluvium appears to have been removed by grading from below the thickest portion of the embankment fill.

Clayey sand colluvium was encountered in Test Pit TP-8 below the fill, at the dam embankment left (easterly) “abutment”. The fill in the abutment area is above the reservoir level.

3.3.3 Older Fan Deposits

The area downslope of the reservoir is underlain by older fan deposits consisting of fine to coarse sand with gravels, cobble and boulders. The thickness of the older alluvium increases with distance downslope and west of the Elsinore Fault.

A previous trench near the embankment fill slope toe (Trench 9, see Figure 3 for location) had been logged (URS, 2009) as "older (Pleistocene) alluvium, consisting of highly weathered to decomposed granitic cobbles and boulders in a matrix of clay" (see Figures 5 and 6 of the Work Plan for Log of Trench 9). Based on this study, our re-interpretation of the log of Trench 9 suggests these materials were likely variably weathered and decomposed granitic rock (rather than deeply weathered alluvial fan gravels), similar to the weathered rock materials encountered at the embankment toe in the test pits for this investigation.

Based on the subsurface explorations, the reservoir embankment is not underlain by older alluvial fan deposits, as previously mapped at the site. Rather, the embankment is underlain by weathered granitic rock, as described below.

3.3.4 Granitic Rock

Highly to completely weathered granitic rock (as DG) was encountered in the test pits along the downstream slope toe, and in the all five of the borings drilled at the crest. Below the fill, the Becker Hammer rig was able to penetrate the weathered granite rock to depths of up to about 15 feet below the embankment fill. The test pits could be excavated 2 to 3 feet into the weathered rock prior to encountering refusal

The Becker Hammer drilling method provided continuous recovery of the disaggregated weathered rock. Relatively undisturbed samples were also obtained of the weathered granitic rock material. The variably weathered rock breaks down during drilling into brown to reddish brown, clayey sand type material with sand- to gravel-size rock fragments, classified as an "SC" type soil. Zones of less weathered granitic rock (weathered in place cobble and boulder-size "core stones") were penetrated in the Becker Hammer borings. Boring B-1 met refusal in the open bit mode, possibly on a granitic core stone.

3.4 GROUNDWATER

Minor groundwater seepage was observed at the bottom of Tests TP-1 and TP-4 at the time of excavation. Both pits were allowed to stand open overnight; the following day 5- to 6-inches of groundwater had accumulated at the bottom of the pits. Seepage levels were approximately 5 to 6 feet below the reservoir level at the time of excavation. The reservoir level typically fluctuates on a weekly basis in response to pumped groundwater supply, and irrigation needs at the ranch. The seepage into the test pits is likely from the reservoir.

Borings B-1, B-2A and B-4 were constructed as monitoring wells. As of late December 2013, ground water had accumulated in Boring B-1 and B-4 at depths of about 29.0 feet and 15.6 feet bgs, respectively. The groundwater at this depth is several feet above the embankment foundation. Groundwater levels appear to have been fairly stable, although the rate of recovery appears to be very slow in the timeframe since the monitoring wells were installed. Boring B-2A has been dry since installation.

The seepage level in Test Pit TP-1 was about 16 feet above the water level measured in Boring B-1 in late December 2013. The difference suggests either the seepage in the test pit may have from an isolated zone of "perched" groundwater in the fill, or the groundwater level in Boring B-1 has not stabilized. For stability analyses, it was conservatively assumed the test pit seepage level was representative of the phreatic surface in the embankment fill, as discussed in Section 4.2.1 below.

There were no signs of surface seepage such as flowing water, moist ground or patches of lush vegetation growth on the downstream slope or toe area of the dam. Seepage was not observed in Trench 9 (excavated along the slope toe in 2009) nor were there indications of groundwater seepage in the current test pits at the toe of the downstream slope. Seeps have not been observed by the ranch manager for more than 25 years.

The depth of the regional groundwater being pumped from local wells to maintain the reservoir is on the order of several hundred feet below the reservoir, depending on the season. The groundwater levels in the monitoring wells are not influenced by the regional groundwater table.

3.5 SEISMICITY

The site is near the Elsinore fault zone, one of several major strike slip fault zones in southern California (Figure 9). Together with the San Andreas system of northwest striking and right lateral faults, the Elsinore fault accommodates a portion of the plate tectonic movement between the North American and Pacific plates.

The Elsinore fault zone comprises a series of right-slip faults extending from the Los Angeles Basin south to the U.S./Mexico Border. Through a long displacement history, the Elsinore fault has produced a series of alternating high and low physiographic features along its length. Some of the regional fault related landforms near the project include Temecula Valley (low area), Agua Tibia Mountain and Palomar Mountain (high areas), Lake Henshaw (low area), and other well expressed fault-related landforms along the length of the fault.

Historically, the Elsinore fault has not produced a major earthquake near Agua Tibia Mountain (in the site vicinity). An earthquake in 1885 may be the nearest large event (estimated magnitude 5.8). The event is placed southeast of the project area in Pauma Valley (Toppozada and others, 2000). The 1910 Temescal Valley earthquake (near Lake Elsinore) is assigned a magnitude 6.0. Smaller magnitude events are generally located in or near Pauma Valley, but none of these small events has produced local damage. Earthquake epicenters for the time frame between 1932 and December 2013 are shown on Figure 9.

For the purpose of estimating seismic shaking at the site, fault rupture was assumed to occur along a single, through going fault segment within the Elsinore fault zone. The values of Peak Ground Acceleration (PGA) were based on a deterministic seismic shaking analysis assuming the Maximum Credible Earthquake (MCE) occurs on the nearest reach of the Elsinore fault (referred to as the Julian segment of the Elsinore fault). The MCE was assumed to result from a 75 km (46.5 miles) long rupture of the entire length of the Julian segment, producing a Magnitude 7.3$\pm$ 0.2 earthquake, at a distance of about 160m (about 500 feet) from the site.

Based on the average of several published attenuation relationships (Abrahamson and others, 2013; Boore and others, 2013; Campbell and Bozorgnia, 2013; Chiou and Youngs, 2013), the deterministic horizontal acceleration spectra for a M 7.5 earthquake (at the 84th percentile) is shown on Figure 10. The averaged PGA is 0.98g at the reservoir site.

SECTION 4 DAM STABILITY EVALUATION

The following section presents discussions, conclusions and recommendations regarding the stability of the earth dam and predicted seismic deformations. The conclusions are based on the interpreted site conditions, the results of the analyses, and professional judgment.

4.1 GEOLOGIC EVALUATIONS

The reservoir dam embankment foundation is underlain by deeply weathered in-place granitic rock. Deep rock weathering requires a stable landform, which is expressed at the site area as the relatively flat mesa below the reservoir. The mesa is underlain at depth by granitic rock, bounded by the Elsinore fault on the west, and encompassing the granitic mountainside on the east. There are no signs of deep seated instability, which would be highly unusual in the granitic rock setting.

Moreover, the older fan deposits in the upper areas of the Shadow Run Ranch site (generally surrounding the reservoir) have been recognized as stable geomorphic surfaces dating back to the late Pleistocene (approximately 15,000 years before present) based on well-developed soil profiles (Vaughn, 1987). There appears to be no geomorphic evidence of past landsliding (e.g., scarps, abrupt slope changes) in the vicinity of the reservoir, as noted during air photo interpretations for the previous fault investigation (URS, 2001; 2009). Considering the site geologic history, the reservoir site is considered geologically stable, and is in a geologic setting that is not prone to deep seated instability.

4.2 SLOPE STABILITY EVALUATIONS

The embankment slope stability analyses were performed on four embankment cross sections (Cross Sections A-A’, B-B’, C-C’, and D-D’). The locations of the cross sections are shown in Figure 3.

4.2.1 Methodology

Slope stability analyses were completed using SLOPE/W, a computer program produced by Geo-Slope International Ltd. (2013) to evaluate the factor of safety against rotational and translational sliding surfaces. The analyses adopted the Spencer Method of Slices for moment and force equilibrium stability. The embankment geometry and interpreted subsurface conditions were input into the SLOPE/W program, and numerous sliding surfaces were checked to determine the critical sliding surface. The assessment of overall stability considered static and seismic conditions.

The reservoir level was assumed to be at the elevation of the proposed new spillway (elevation 1,082.6 feet MSL). The phreatic surface through the embankment was estimated assuming a groundwater level at the reservoir level on the upstream side and a groundwater level approximately 5 feet below the surface at the downstream toe. The phreatic surface was developed using the seepage analyses program, SEEP/W (Geo-Slope, 2013). However, the phreatic surface from SEEP/W was adjusted assuming the seepage level at Test Pit TP-1 (on Section A-A’) and the groundwater level indicated in Boring B-4 (Section D-D’) were representative of the maximum phreatic surface.

The seismic stability of the reservoir embankment was evaluated by performing pseudostatic slope stability analyses and post-liquefaction slope stability analyses. The pseudostatic case assumes that there are no liquefiable materials within the embankment or foundation. As reported in Kramer (1996), Marcuson (1981) suggested that appropriate pseudostatic coefficients for dams should correspond to ⅓ to ½ of the estimated PGA (0.98g for this project). The pseudostatic analyses were performed by assuming the worst case of a sustained horizontal acceleration equal to ½ of the estimated PGA; a sustained acceleration of 0.49g was used. The post-liquefaction analyses were performed assuming that the strength of the potentially liquefiable soils (the colluvium) was reduced to a residual undrained strength; a pseudostatic acceleration was not applied for the post-liquefaction analyses.

SLOPE/W was also used to calculate the yield acceleration for use in simplified seismic deformation analyses. The yield acceleration is defined as the sustained horizontal acceleration that produces a factor of safety equal to 1.0.

4.2.2 Material Properties

The material parameters used in the stability analyses are presented in Table 3. These properties were selected based on the results of laboratory testing and published values for similar materials.

The embankment fill soil was described as dense, well compacted clayey sand with gravel (Unified Soil Classification of “SC”). Atterberg tests were performed on samples of the clayey sand fill (see Appendix B). These tests indicated the clayey sand soil has low plasticity, but would still be considered to behave as “sand-like” material with drained strengths even during seismic loading. The fill soil is predominantly sand based on its gradation. For the post-liquefaction stability analyses, it was conservatively assumed that the entire colluvium layer would liquefy during an earthquake, even though itis recognized that some of the colluvium is actually above the phreatic surface. The residual undrained shear strength of the liquefiable colluvium layer was evaluated using an empirical correlation with sampler blow counts developed by Seed and Harder (1990). The colluvium materials were not encountered in the exploratory borings, thus no blow count data was available. Based on a visual inspection, the soil material was observed to be very loose and a low blow count, $(N_1)_{60\text{-}cs}$, of 8 to 10 blows per foot was assumed. This corresponds to an estimated residual undrained strength of 250 pounds per square foot (psf) for the colluvium in post-liquefaction conditions.

4.2.3 Results

The factors of safety obtained from the slope stability analyses are summarized in Table 4; graphical results are provided in Figures 11 through 39. These figures depict the cross section geometry, the material properties used, the critical sliding surface, and the resulting minimum factor of safety (FS). The grid of points on these figures indicates the center of circular sliding surfaces that were analyzed. Although comparison of the previous topography with the conditions encountered in Boring B-1 indicates that the majority of the colluvium was likely removed during the grading for the reservoir (as depicted in Figure 6), parametric analyses were performed on Section A-A’ assuming that the colluvium did extend to the location of Boring B-1. Wedge shaped slip surfaces through the colluvium were also analyzed for the post-liquefaction case.

The analyses indicate that the earthen embankment for the reservoir will be stable for static (minimum required FS = 1.5) and post liquefaction conditions (minimum required FS = 1.1). The pseudo-static factor of safety for all sections were above 1.1, except for Section A-A' where the phreatic surface was at the level of the seeps observed in Test Pit TP-1. However, the estimated seismic deformations (discussed in Section 4.3) for this cross section are only about 2 inches and are acceptable for the freeboard that is available at the embankment.

4.3 SEISMIC DEFORMATIONS

The Newmark (1965) sliding block method was used to estimate the seismic deformations. A major assumption in this methodology is that the peak ground acceleration exceeds the yield acceleration. The Newmark method applies to slopes that are expected to deform as a single massive block unit. Deformations are calculated using the following equation.

$$\log d = 0.90 + log\left[\left(1 - \frac{a_y}{a_{max}}\right)^{2.53}\left(\frac{a_y}{a_{max}}\right)^{-1.09}\right]$$

where d = estimated downslope movement caused by the earthquake (cm)

a_y = yield acceleration, defined as the horizontal earthquake acceleration that results in a pseudo-static factor of safety that is exactly equal to 1.0.

a_{max} = peak ground acceleration of the design earthquake = 0.98g (for this site)

The Newmark analyses indicate that only small seismic deformations (less than about 2 inches) would occur at Cross Sections A-A', C-C' and D-D', as shown in Table 5. These values are within what would be considered suitable performance for earth dams (embankments).

4.4 PIPING POTENTIAL

The embankment soils are mostly clayey sand (Unified Soil Classification of SC) with some gravel. The clay fraction (binder) has low plasticity. The fill appears to be well compacted and appears to have been placed with moisture control. According to Sherard and others (1963), this type of soil[2] in an earth dam embankment would have "intermediate piping resistance". Two of the three pinhole tests performed were classified as "dispersive" (the other was "non-dispersive") which suggests the fill has some susceptibility to piping.

SEEP/W modeling suggests the maximum hydraulic gradients are located along the interface between the embankment and DG foundation. The maximum gradients are about 0.4 which is at the critical gradient (0.3 to 0.4) required for possible piping failures (Cedergren, 1977). However, the foundation below the highest part of the embankment is in weathered granitics, which are not erodible by piping. The topsoil/colluvium layer does not appear to extend below the full footprint of the dam. Therefore there is no continuous flow path from the toe to the reservoir. The topsoil/colluvium itself is a relatively heterogeneous mixture of clayey sand with cobbles not likely conducive to backward erosion and piping below the dam.

[2] A soil type described as "well-graded coarse sand or sand-gravel mixtures with a binder of clay of medium plasticity (PI greater than 6). Well compacted".

4.5 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are made as part of this geotechnical evaluation of the Shadow Ranch reservoir earth dam:

1. The reservoir site is geologically stable in a geologic setting that is not prone to deep seated instability. There are no geomorphic features suggestive of past landsliding in the vicinity of the reservoir. The embankment fill slope has performed well with no significant instability for over 50 years.

2. The embankment soil is comprised of dense, clayey sand "DG" material with some gravel-sized fragments of granitic rock. The thickness of the embankment fill ranged between about 18.5 and 31 feet. The fill was placed in thin lifts and appears to be dense and well-compacted. The fill soil has a relatively uniform gradation. There were no large rocks, boulders, vegetative material or other debris within the fill. The thickest part of the embankment appears to have been placed directly on weathered granitic rock that had been stripped of topsoil/colluvium. Fill may have been placed on topsoil/colluvium at the downslope toe of the embankment.

3. The embankment foundation is underlain by variably weathered granitic rock. The original ground surface had been lowered to create a cut area; the excavation extended into weathered granitic rock. Boulders and loose rock were removed.

4. The dam section does not include a central cutoff trench or impervious core. The dam is a homogeneous earth dam constructed entirely of a single embankment material.

5. The dam does not have a downstream drain. The dam is low enough to not warrant having a downstream drain. The embankment and foundation have low susceptibility to piping.

6. Groundwater appears to be seeping through the embankment fill in some areas. The groundwater levels indicated in the test pits and one of the two monitoring wells suggest a normal phreatic surface for an earthen dam. The absence of groundwater in one of the wells (Boring B-2A) suggests the fill is relatively impervious and there are no significant groundwater flow paths in the fill.

7. The stability analyses indicate that the earthen embankment for the reservoir will be stable for all static and post liquefaction conditions. One pseudostatic case with conservative assumptions did have a factor of safety less than 1.1; however, the predicted seismically induced deformations are acceptable.

8. Seismic deformations of less than about 2 inches are anticipated at the site and these values are within what would be considered suitable performance for earth dams.

We recommend the proposed spillway be founded in formational materials and should thereby be moved to the general area of Test Pit TP-8.

The existing mature palm trees can be left in place provided they continue to be well maintained and stay healthy. If a palm tree is removed, the former root area should be replaced with compacted soil. Several mature oak trees have also grown near the downstream toe of the embankment. The oak tree root systems are far from the reservoir and do not compromise the integrity of the embankment. If the oak trees are removed, the roots should be fully excavated and replaced with compacted fills.

There have been no indications of seepage at the reservoir in the past. Seepage however, can occur after many years of operation. Even small seeps can increase and lead to an erosive piping failure of the embankment. Regular monitoring of the embankment is an important mitigation measure, especially following a seismic event.

SECTION 5 UNCERTAINTY AND LIMITATIONS

We have observed only a very small portion of the pertinent soil and groundwater conditions. The conclusions and recommendations made herein are based on the assumption that subsurface conditions do not deviate appreciably from those found during the previous and current field investigations.

Geotechnical engineering and geologic sciences are characterized by uncertainty. Professional judgments presented herein are based partly on our understanding of the scope of work, and partly on our general experience. Our engineering work and judgments rendered meet current professional standards; we do not guarantee the performance of the project in any respect.

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Table 1
Summary of Exploratory Borings
Geotechnical Evaluation – Shadow Run Ranch

Boring	Location	Method of Drilling and Sampling	Maximum Depth of Drilling (feet)	Monitoring Well Installation
B-1	Section A-A'	Open bit drilling, with SPT and Mod Cal sampling.	45.1	2-inch Slotted PVC
B-2	Section B-B'	Open bit drilling, with SPT and Mod Cal sampling.	36.5	
B-2a	Section B-B'	Open bit drilling, with SPT and Mod Cal sampling.	28.5	2-inch Slotted PVC
B-3	Section C-C'	Open bit drilling, with SPT and Mod Cal sampling.	31.5	
B-4	Section D-D'	Open bit drilling, with SPT and Mod Cal sampling.	29.4	2-inch Slotted PVC
		Total	**171**	**103**

Table 2
Summary of Test Pits
Geotechnical Evaluation – Shadow Run Ranch

Test Pit	Location	Materials Encountered	Depth of Test Pit (feet)
TP-1	Crest	Fill	12.9
TP-2	Crest	Fill	14.6
TP-3	Crest	Fill	14.5
TP-4	Crest	Fill	14.5
TP-5	Toe	Fill, Topsoil/Colluvium, Decomposed Granite	8
TP-6	Toe	Fill, Topsoil/Colluvium, Decomposed Granite	8.2
TP-7	Toe	Fill, Decomposed Granite	8
TP-8	Crest	Fill, Colluvium	5.5
TP-9	Crest	Fill, Decomposed Granite	9.5
TP-10	Toe	Colluvium, Decomposed Granite	4.8
TP-11	Toe	Fill, Decomposed Granite	7

Table 3
Soil Parameters used in Stability Analyses
Geotechnical Evaluation – Shadow Run Ranch

Parameters	Material Type				
	Embankment Fill (SC)	**Colluvium**	**Colluvium (post liquefaction)**	**Decomposed Granite**	**Rip Rap**
Unit Weight (pcf)	125	125	125	140	135
Friction Angle (degrees)	36	36	0	40	40
Cohesion (psf)	500	0	250	1,000	0
Hydraulic Conductivity, k_x, (cm/sec)	10^{-5}	10^{-3}	10^{-3}	10^{-7}	1
Hydraulic Conductivity Anisotropy (k_x/k_y)	4	4	4	1	1

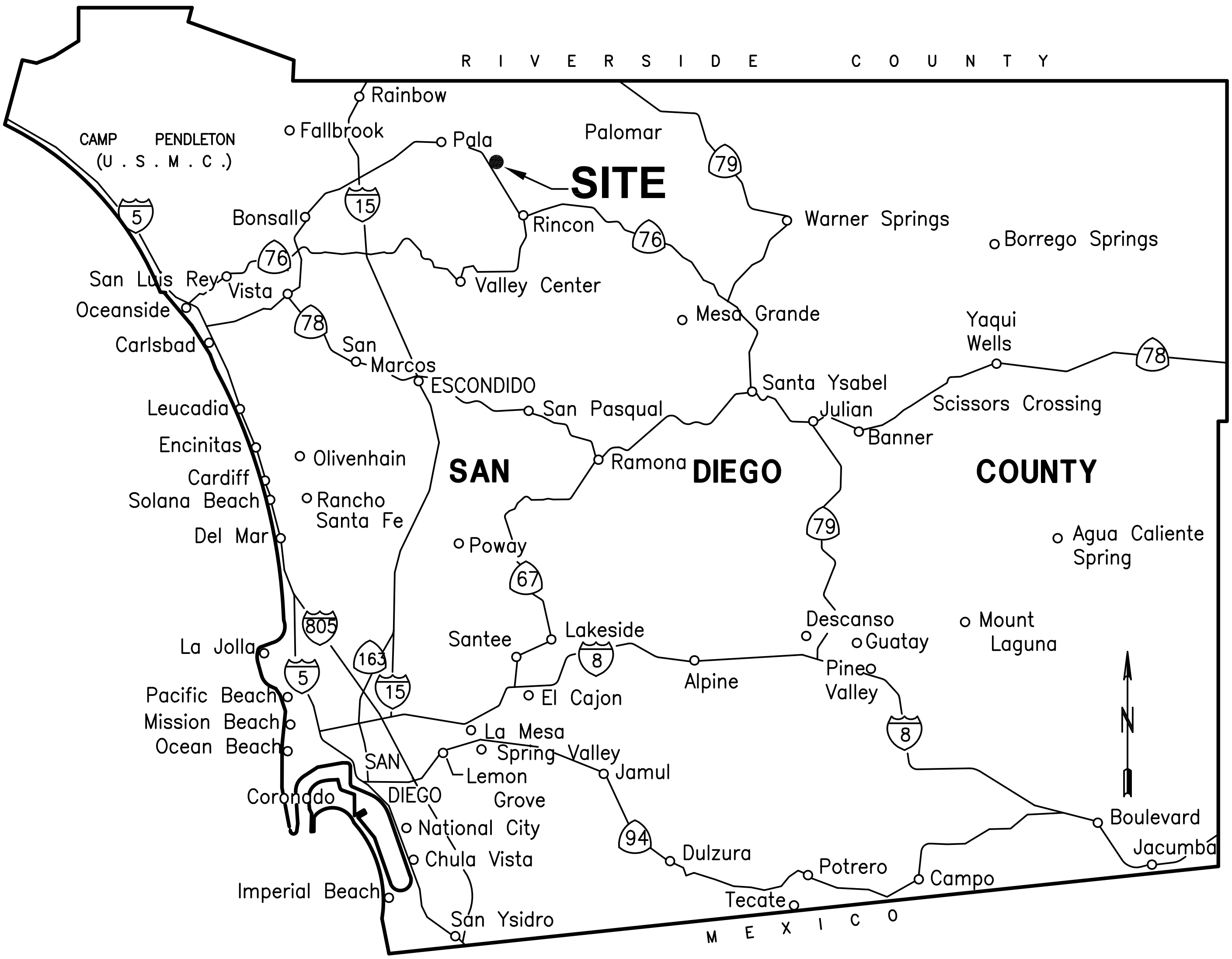
Table 4
Results of Slope Stability Analyses
Geotechnical Evaluation – Shadow Run Ranch

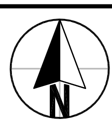
Case		Cross Sections			
	A-A’	A-A’ (with extended colluvium)	B-B’	C-C’	D-D’
Factor of Safety for Static	2.30	2.13	2.45	2.89	4.00
Factor of Safety for Pseudostatic	1.01	0.86	1.12	1.16	1.24
Factor of Safety for Post Liquefaction (Circular Slip Surface)	2.12	1.75	2.07	2.28	3.40
Factor of Safety for Post Liquefaction (Wedge Slip Surface)	2.18	1.50	2.58	1.66	2.32
Yield Acceleration, a_y (g)	0.49	0.41	0.56	0.62	0.74

Table 5
Estimated Seismic Deformations
Geotechnical Evaluation – Shadow Run Ranch

Parameter		Cross Sections			
	A-A’	A-A’ (with extended colluvium)	B-B’	C-C’	D-D’
a_y/a_{max}	0.418	0.500	0.571	0.633	0.755
Deformation (in)	1.15	2.05	0.67	0.41	0.12

Figures





VICINITY MAP
SHADOW RUN RANCH

URS	NOT TO SCALE	CHECKED BY: DLS	DATE: 02-01-13	FIG. NO:
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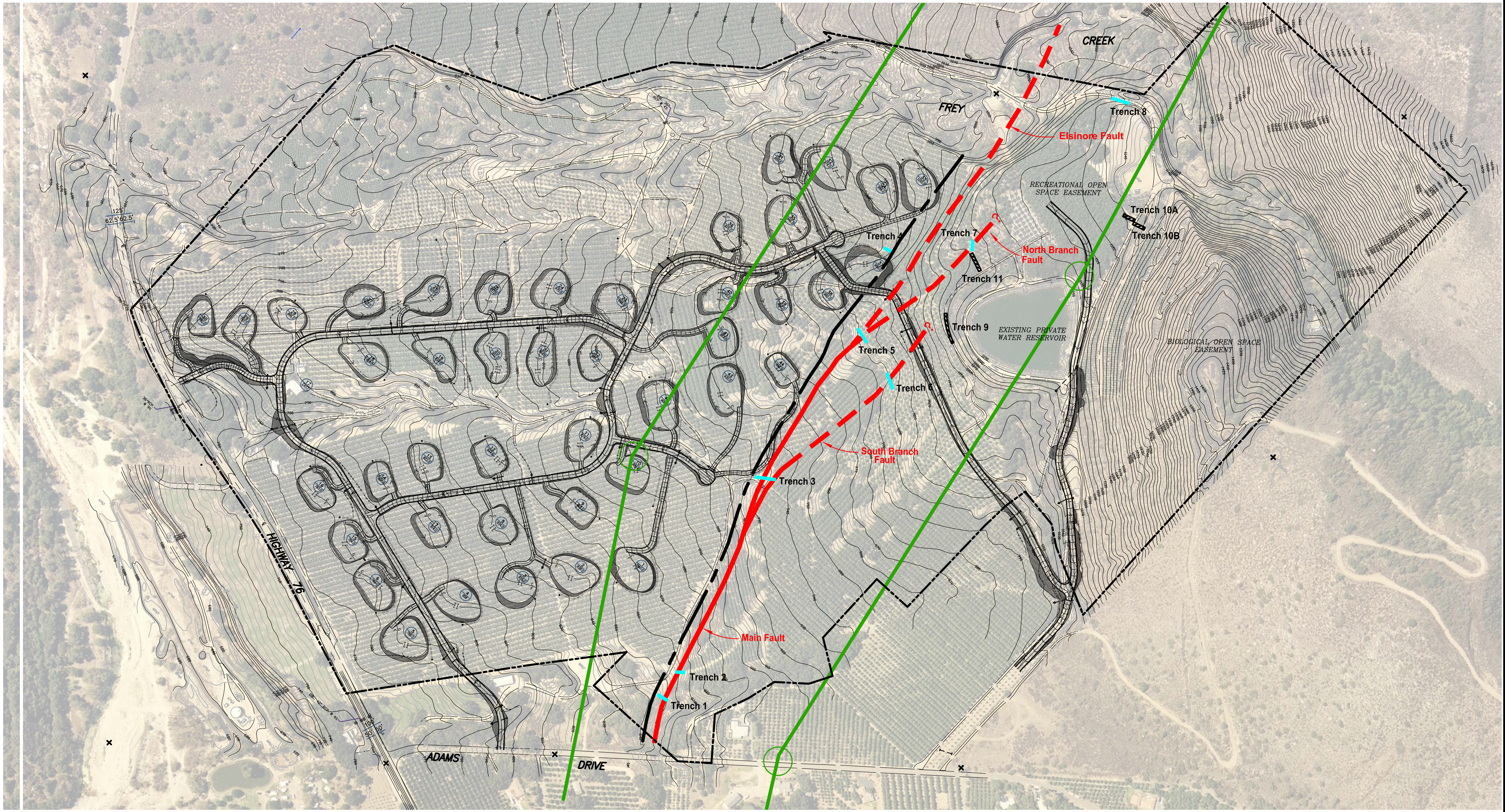

LEGEND:

- FAULT, DASHED WHERE APPROXIMATE
- FAULT SETBACK BOUNDARY (WEST OF MAIN FAULT)
- SUPPLEMENTAL TRENCH LOCATION (URS, 2009)
- PREVIOUS EXPLORATORY TRENCH (URS, 2001)
- EARTHQUAKE FAULT ZONE (APPROXIMATE)

SITE PLAN
MINERAL RESOURCES INVESTIGATION
SHADOW RUN RANCH

URS

200 0 200 400 Feet
SCALE: 1"= 400'

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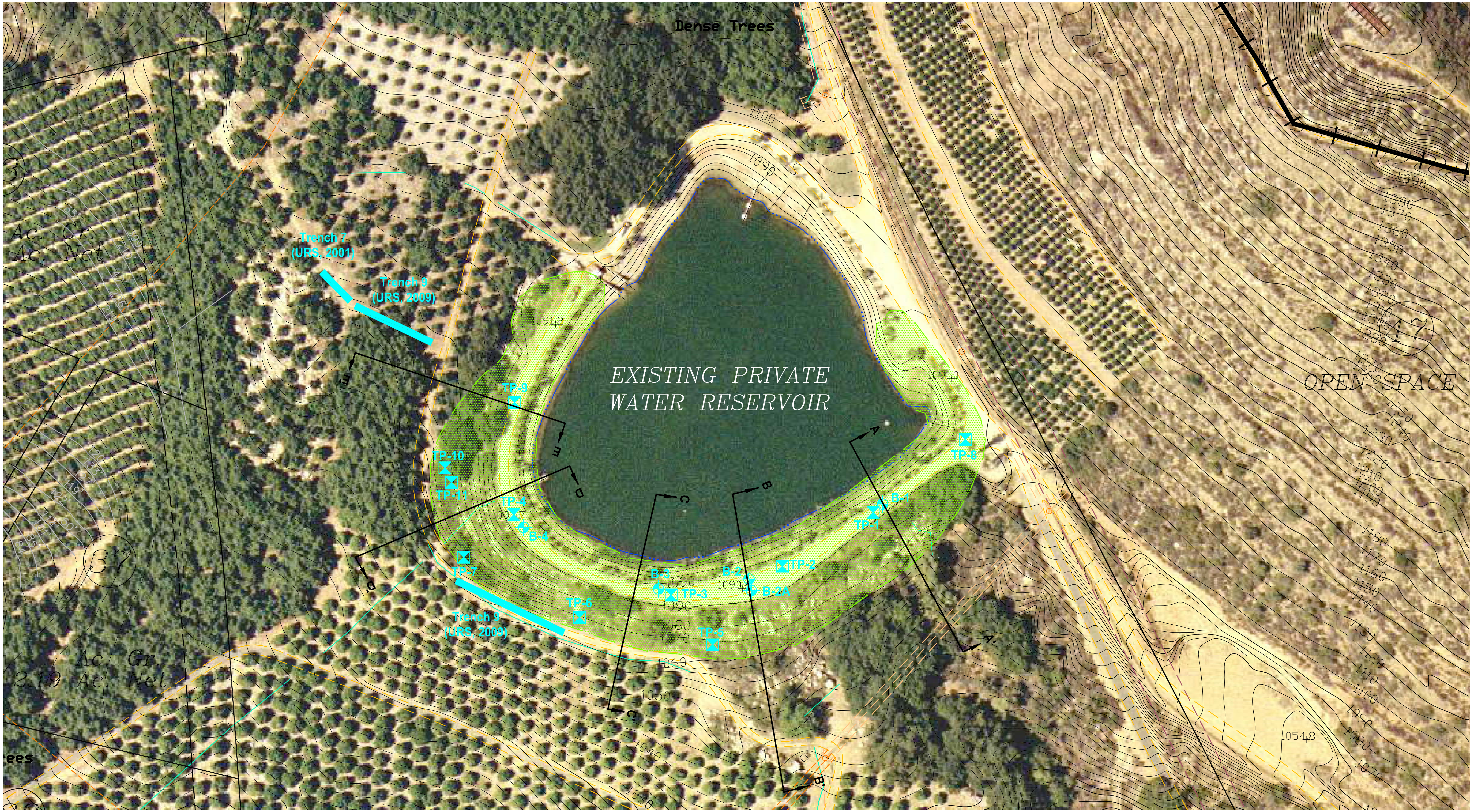

LEGEND

- INDICATES APPROXIMATE LOCATION OF TEST BORING
- INDICATES APPROXIMATE LOCATION OF TEST PIT
- INDICATES APPROXIMATE LOCATION OF CROSS SECTION
- INDICATES APPROXIMATE LOCATION OF PREVIOUS TRENCH (URS, 2001, 2009)
- APPROXIMATE LIMITS OF EMBANKMENT FILL

SITE PLAN
SHADOW RUN RANCH
PAUMA VALLEY, CALIFORNIA

URS

SCALE: 1"= 100' (50 0 50 100 feet)

CHECKED BY: DLS	DATE: 11-25-13	FIG. NO:
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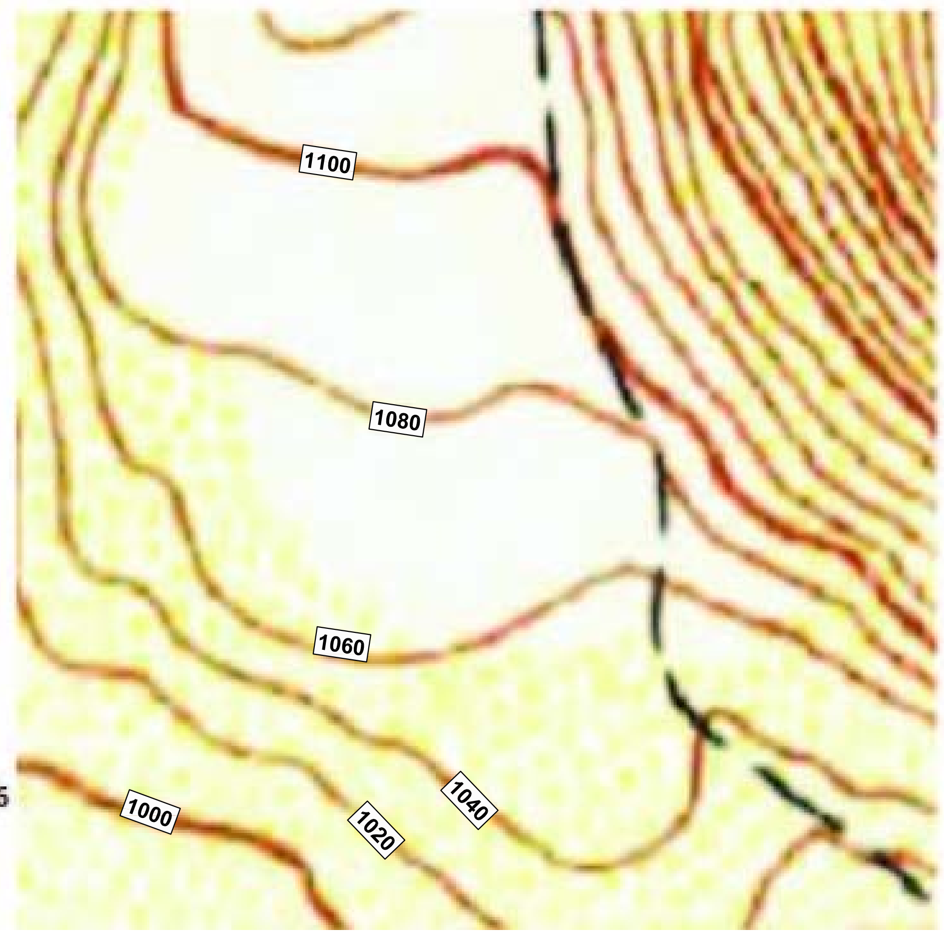
1956



1970

LEGEND

1060 APPROXIMATE GROUND SURFACE ELEVATION TOPOGRAPHIC CONTOUR, FEET MSL

HISTORICAL TOPOGRAPHIC MAPS SHADOW RUN RANCH PAUMA VALEY, CALIFORNIA			
URS	CHECKED BY: DLS	DATE: 12-26-13	FIG. NO:
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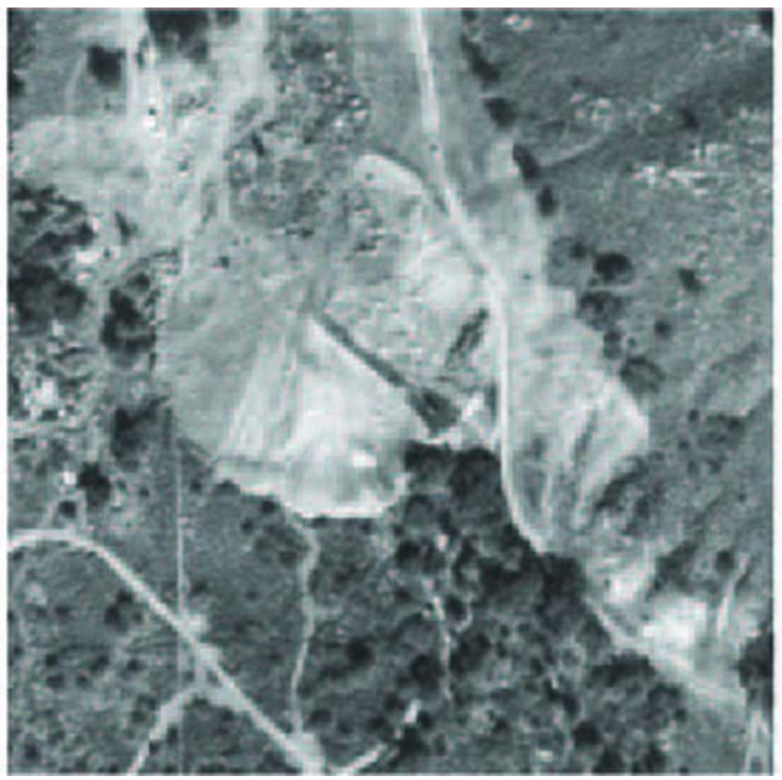
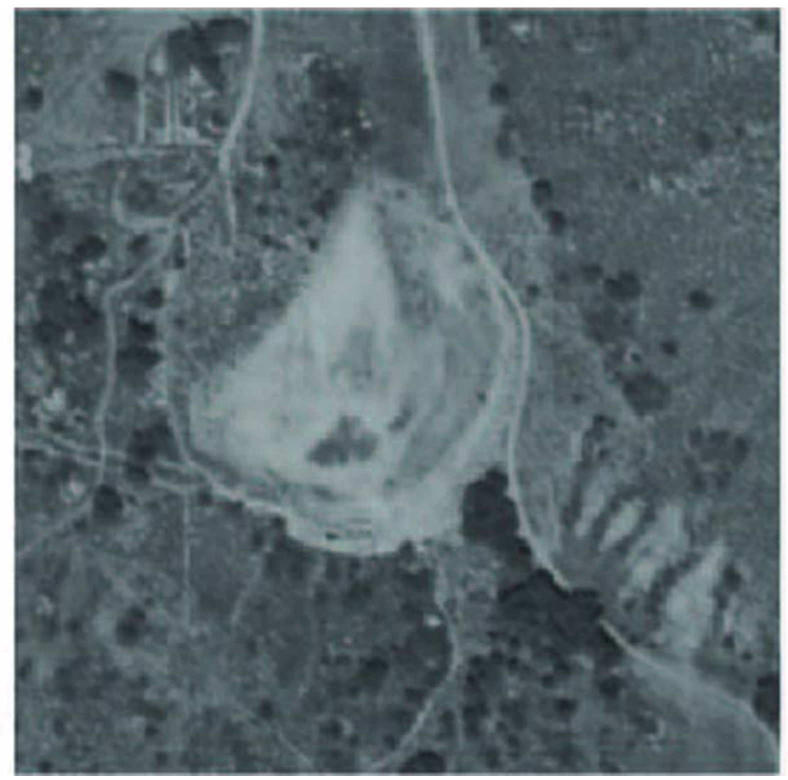
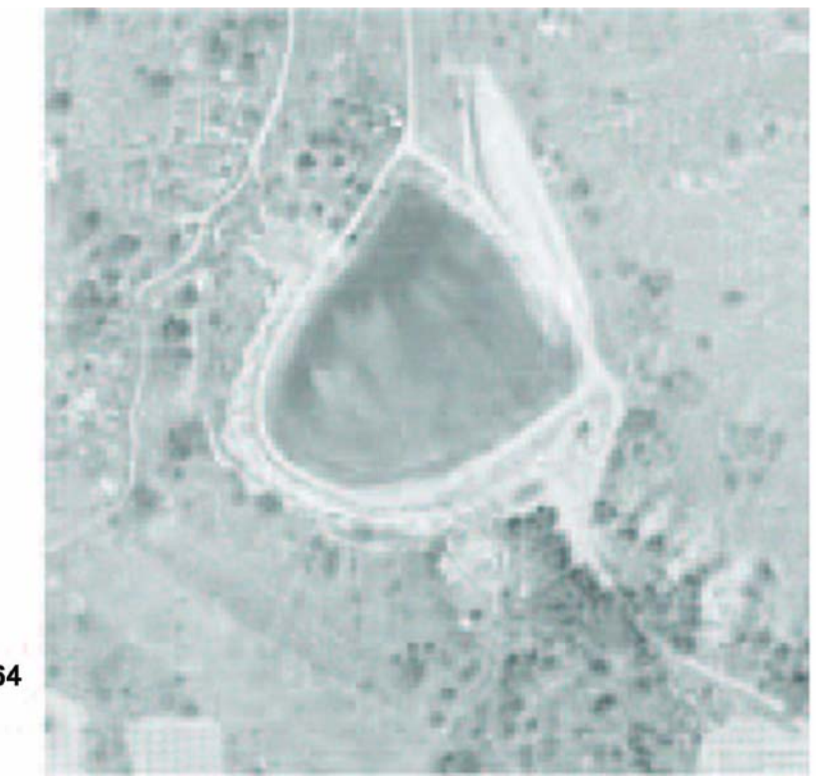
1946



1953



1964



HISTORICAL AERIAL PHOTOGRAPHS
SHADOW RUN RANCH
PAUMA VALEY, CALILFORNIA

URS	CHECKED BY: DLS	DATE: 11-25-13	FIG. NO:
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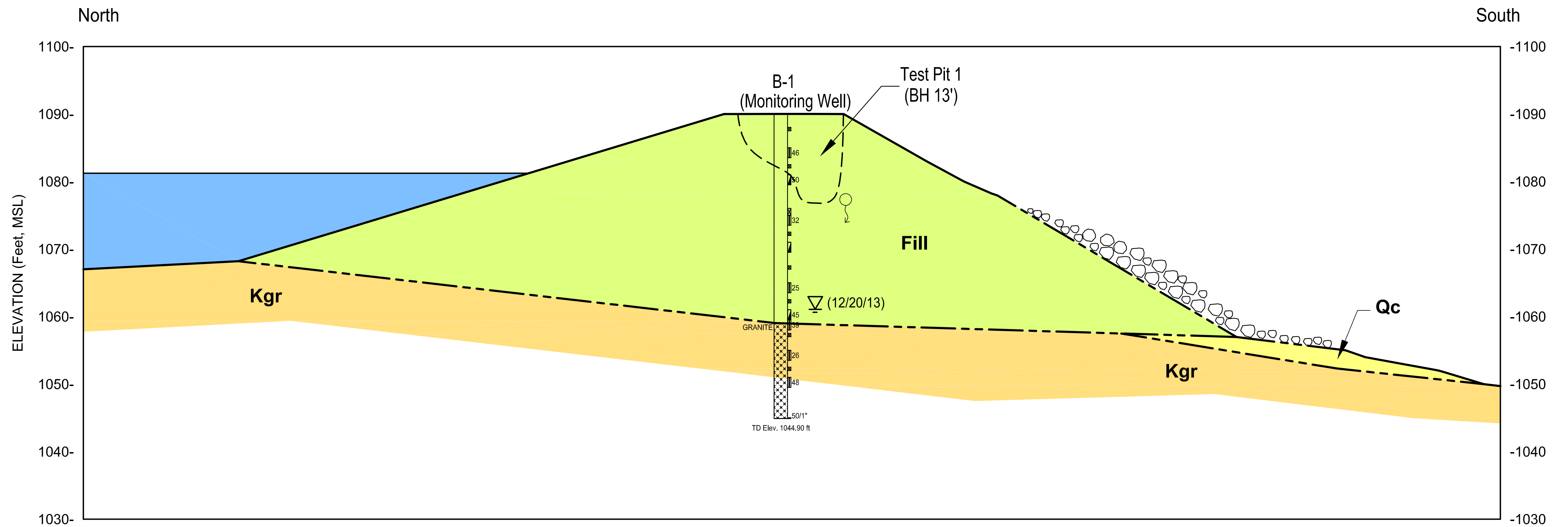
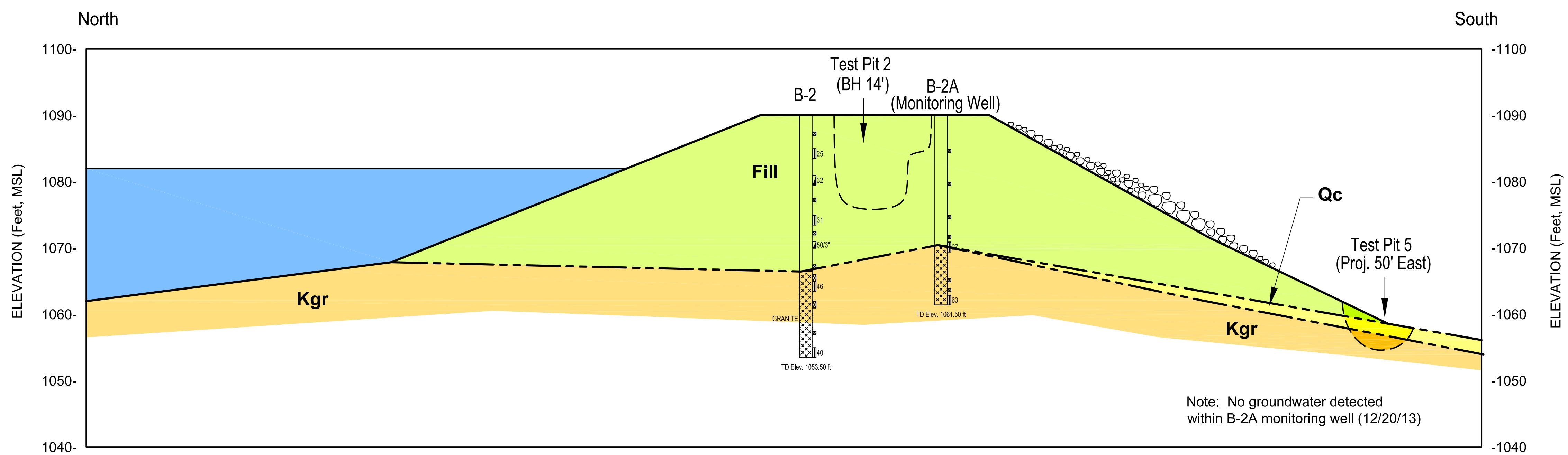


SECTION A-A'



SECTION B-B'

LEGEND

(blue)	Reservoir
Fill	Fill
Qc	Topsoil/Colluvium
Kgr	Granitic Rock
(boulders)	Boulders/Riprap

KEY TO BORING LOG SYMBOLS

TYPICAL SAMPLER GRAPHIC SYMBOLS

- Grab sample
- 2.5" ID sampler
- Standard Penetration sampler

TYPICAL MATERIAL GRAPHIC SYMBOLS

- Decomposed granitic rock

OTHER GRAPHIC SYMBOLS

- Groundwater measured in monitoring well
- Geologic contact (approximate)
- Seepage observed in test pit

GENERALIZED GEOLOGIC CROSS SECTION A-A' AND B-B'
SHADOW RUN RANCH
PAUMA VALLEY, CALIFORNIA

URS

10 0 10 20 feet
SCALE: 1"= 20' (H&V)

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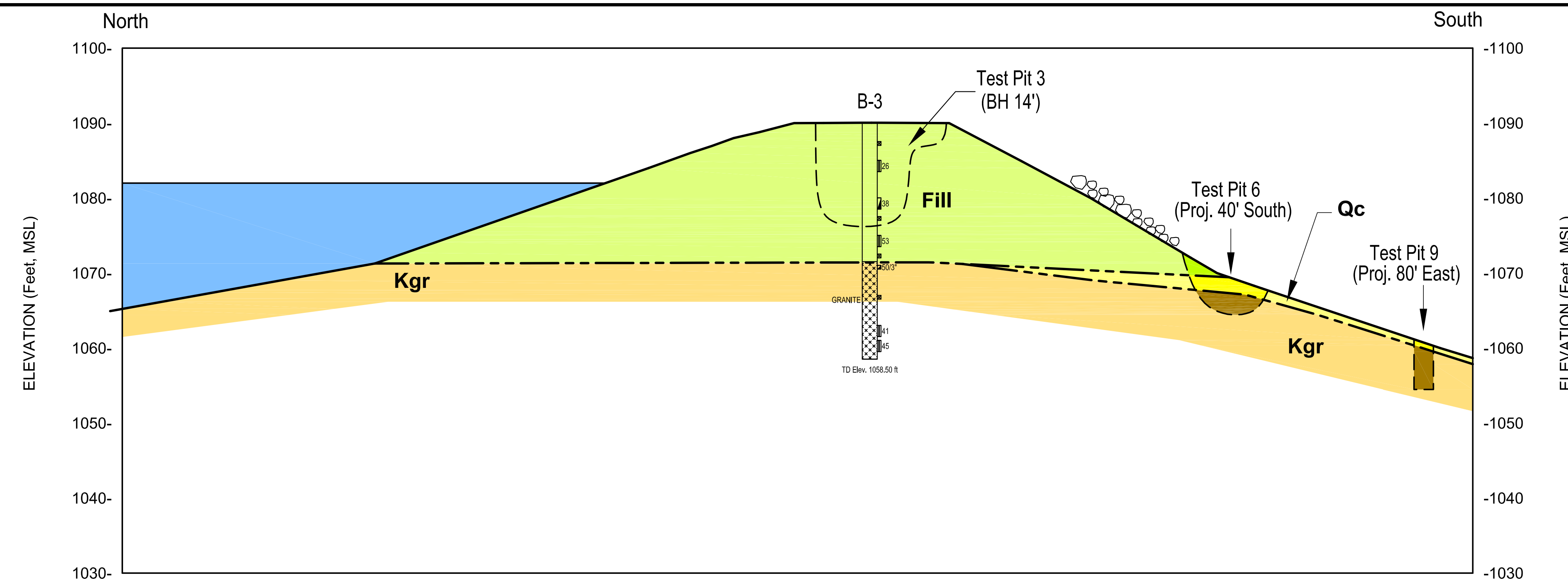
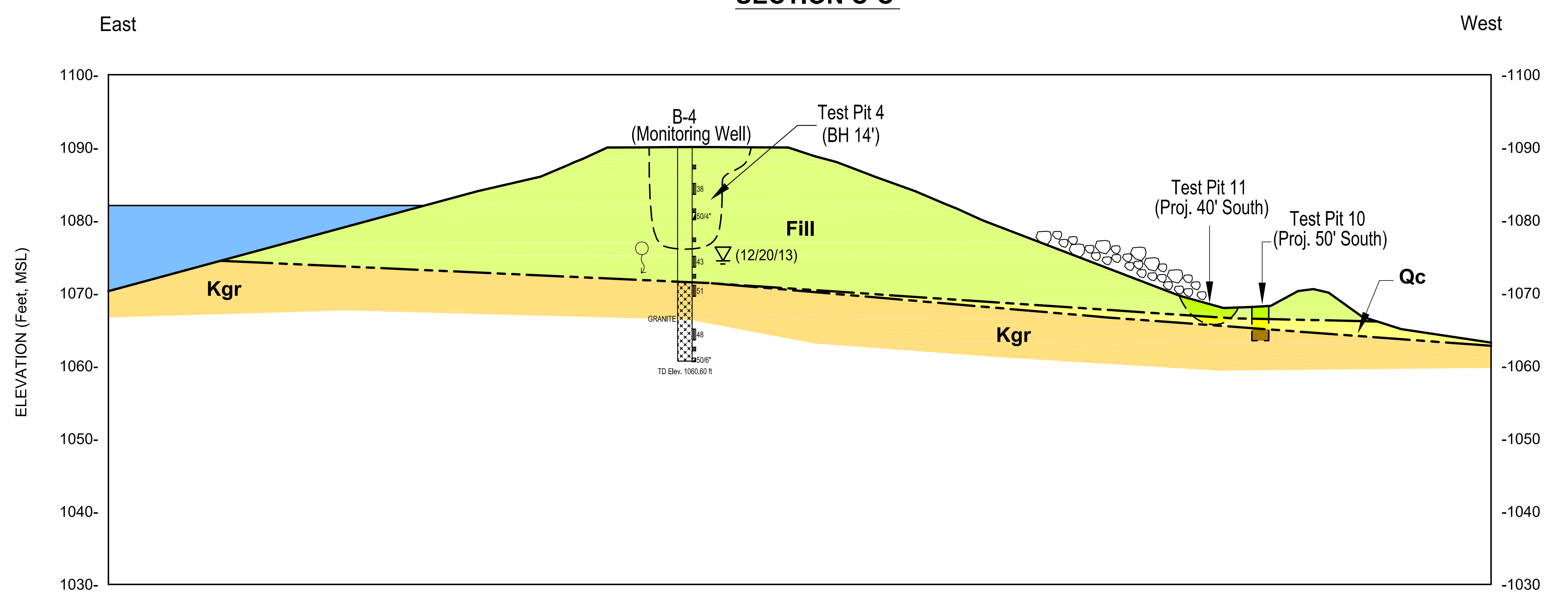


SECTION C-C'



SECTION D-D'

LEGEND

Reservoir	Reservoir
Fill	Fill
Qc	Topsoil/Colluvium
Kgr	Granitic Rock
	Boulders/Riprap

KEY TO BORING LOG SYMBOLS

TYPICAL SAMPLER GRAPHIC SYMBOLS
- Grab sample
- 2.5" ID sampler
- Standard Penetration sampler

TYPICAL MATERIAL GRAPHIC SYMBOLS
- Decomposed granitic rock

OTHER GRAPHIC SYMBOLS
- Groundwater measured in monitoring well
- Geologic contact (approximate)
- Seepage observed in test pit

GENERALIZED GEOLOGIC CROSS SECTION C-C' AND D-D'
SHADOW RUN RANCH
PAUMA VALLEY, CALIFORNIA

URS

10 0 10 20 feet
SCALE: 1"= 20' (H&V)

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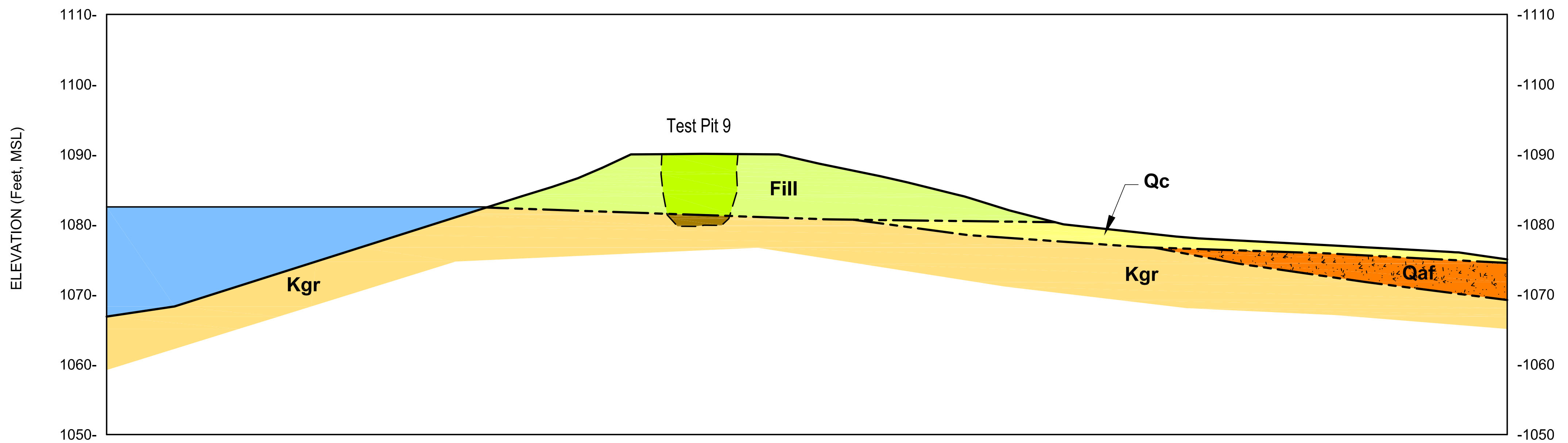


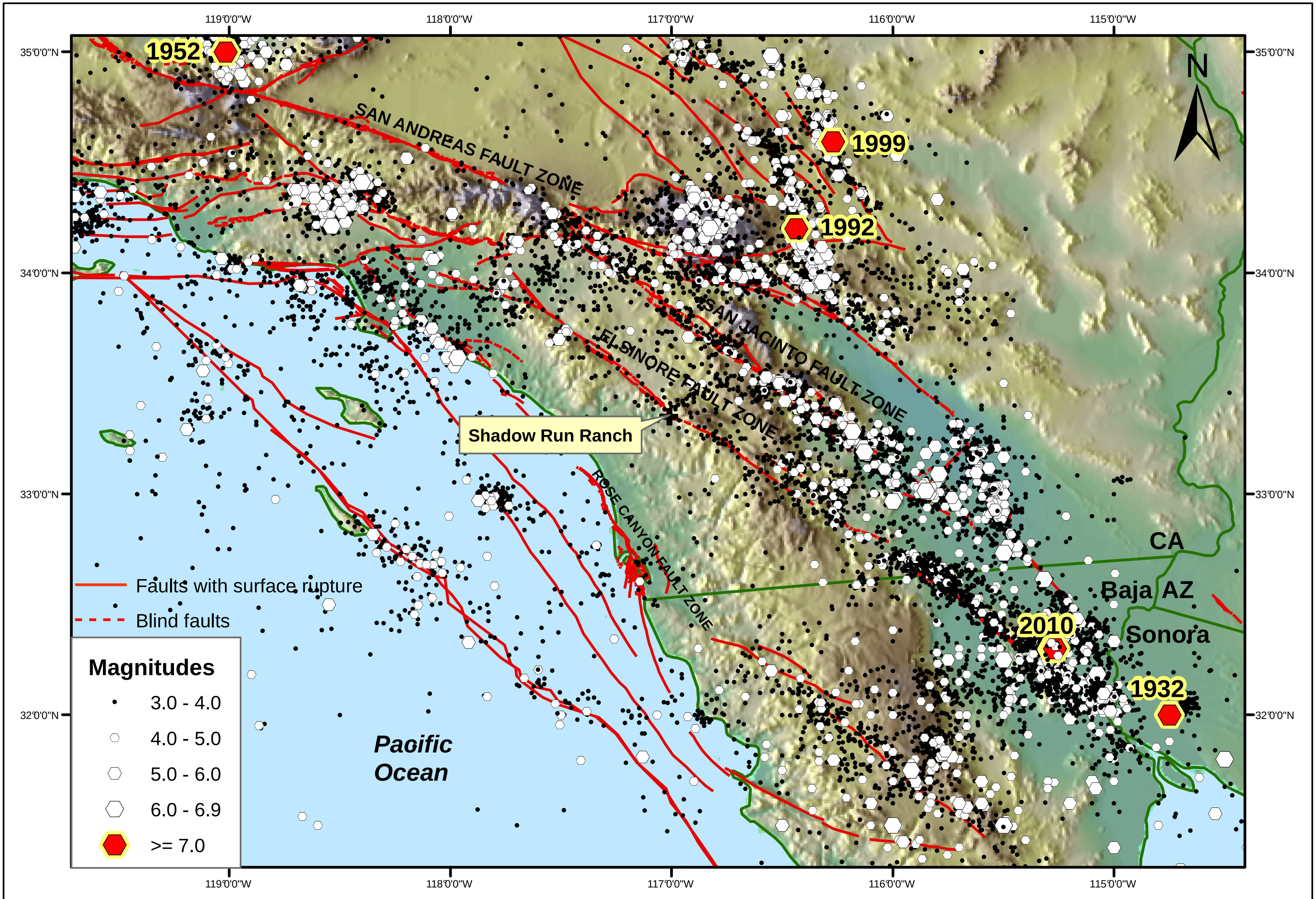
SECTION E-E'

LEGEND

- Reservoir
- Fill — Fill
- Qc — Topsoil/Colluvium
- Qaf — Older Fan Deposits
- Kgr — Granitic Rock
- — - — Geologic contact (approximate)

GENERALIZED GEOLOGIC CROSS SECTION E-E'
SHADOW RUN RANCH
PAUMA VALLEY, CALIFORNIA

URS	10 0 10 20 feet SCALE: 1"= 20' (H&V)	CHECKED BY: DLS	DATE: 12–26–13	FIG. NO:
		PM: DLS	PROJ. NO: 27661027.10000	8



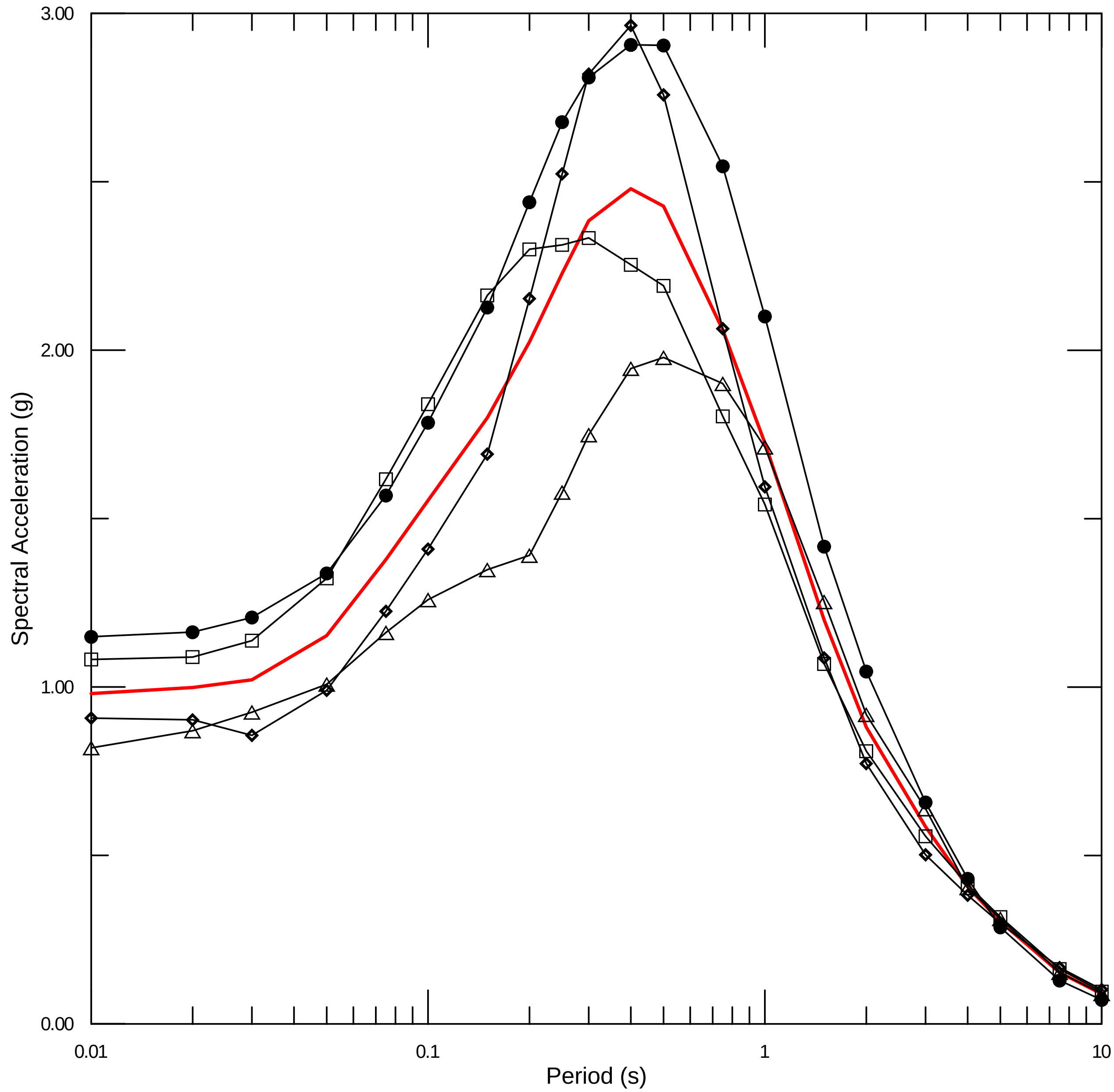
URS

Project No. 27661027

Shadow Run Ranch
California

HISTORICAL SEISMICITY IN THE SITE REGION
(M >= 3.0, 1932 - DECEMBER 2013)

Figure
9



Abrahamson et al. (2013)
Boore et al. (2013)
Campbell and Bozorgnia (2013)
Chiou and Youngs (2013)
Lognormal Average

Site Condition

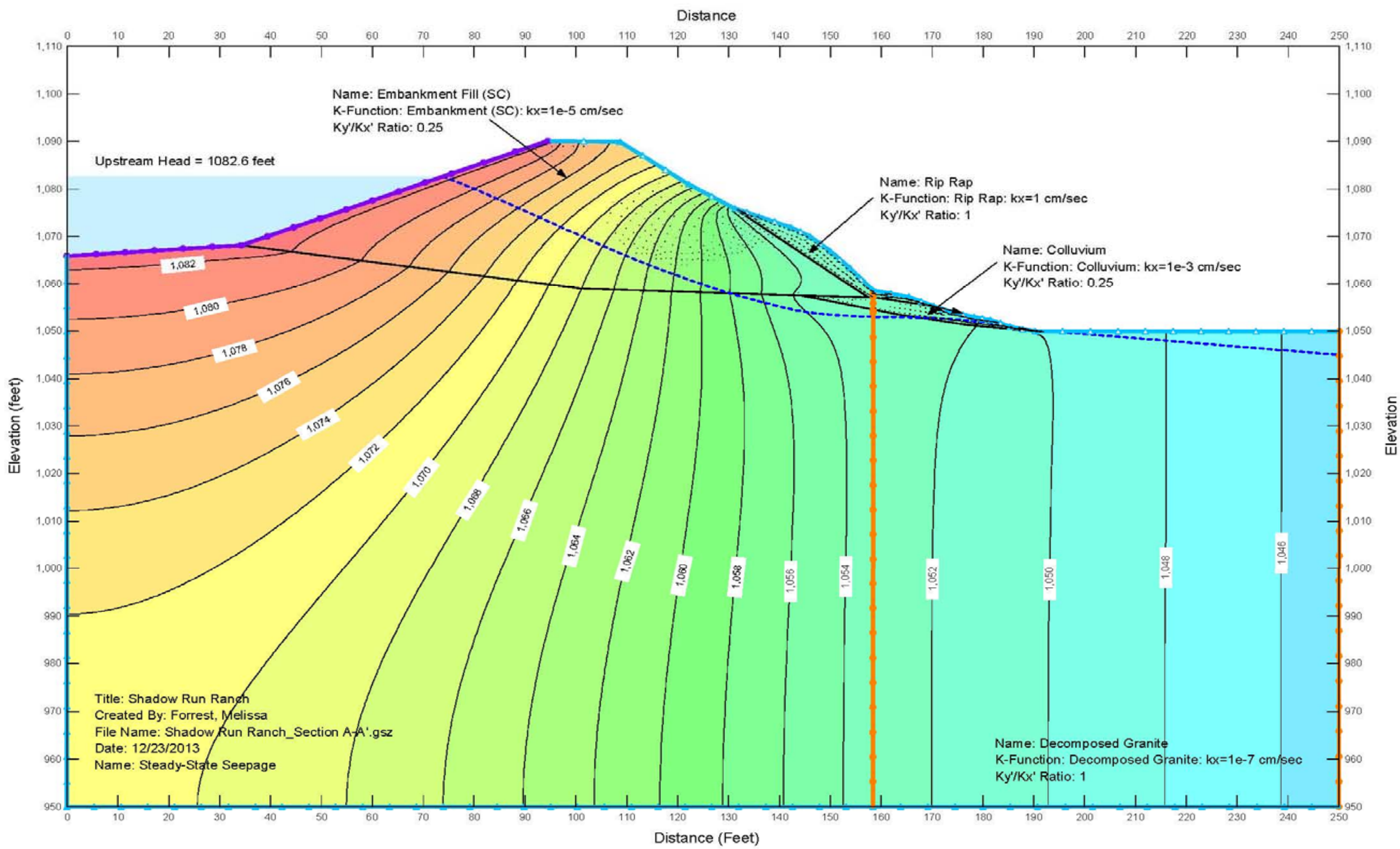
V_S=332 m/sec
$Z_{1.0}$ = 0.43 km (Abrahamsom et al. 2013);
0.43 km (CY 2013)
$Z_{2.5}$ = 1.57 km (CB 2013)

PGA = 0.98 g

Distances

r_r (rupture) = 0.16 km
r_h (horizontal) = 0.16 km
Rupture Width = 13 km

URS	Project No. 27661027 Shadow Run Ranch San Diego, CA	84^{th}-PERCENTILE HORIZONTAL ACCELERATION RESPONSE SPECTRA FOR THE **M** 7.5 ELSINORE FAULT MAXIMUM EARTHQUAKE	Figure 10



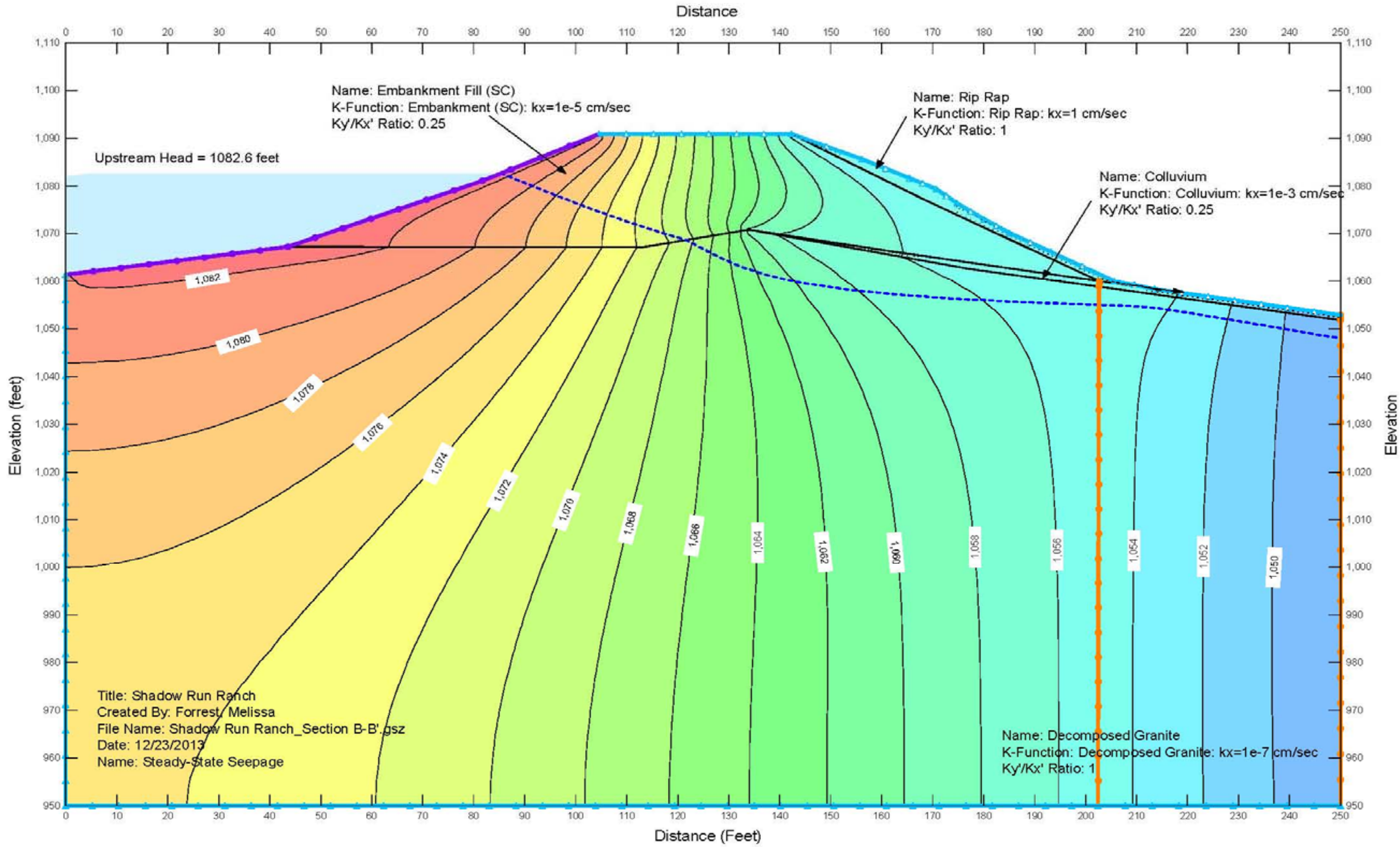
Seepage Analyses of Cross Section A-A'
Steady State Conditions

URS

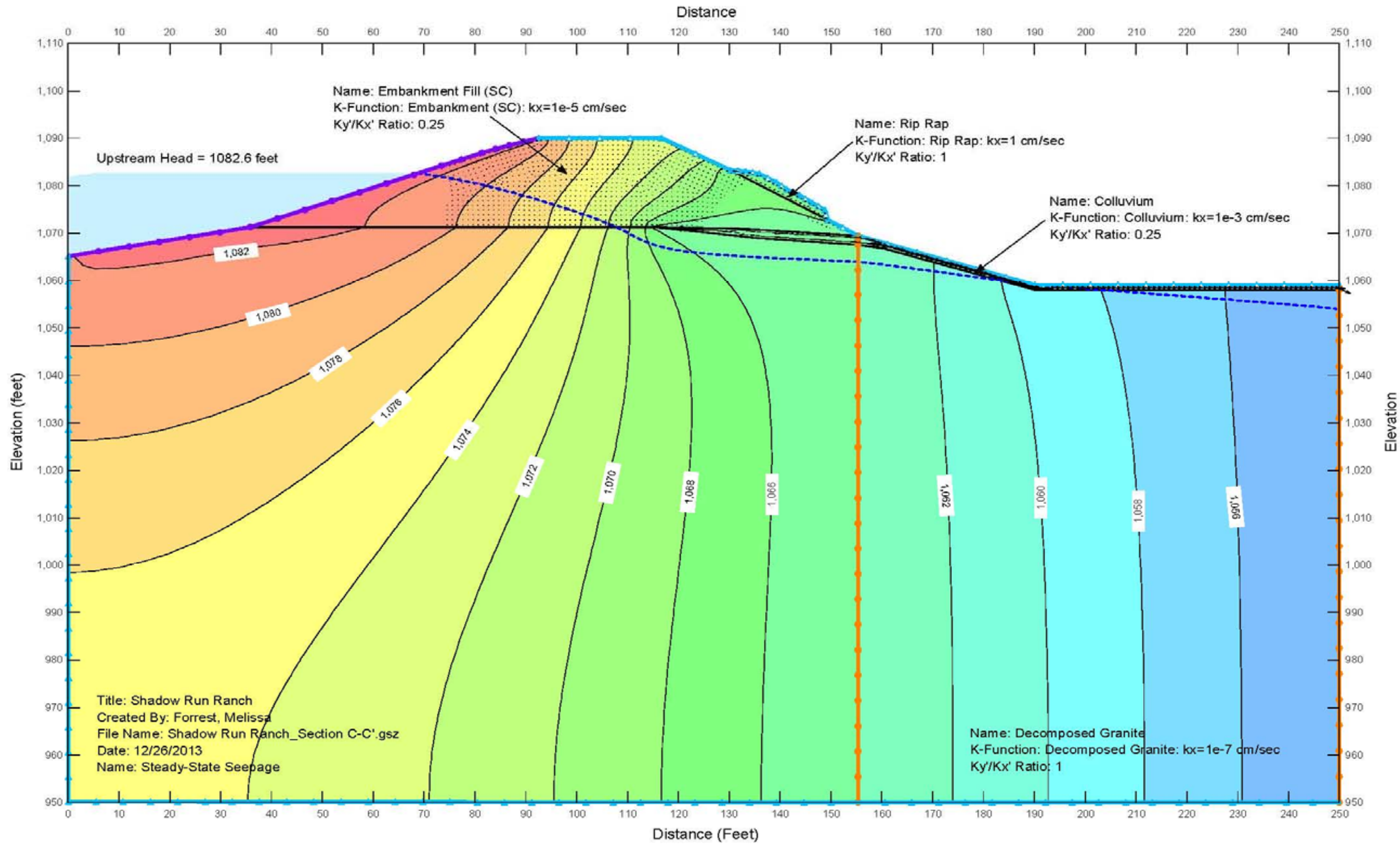
Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

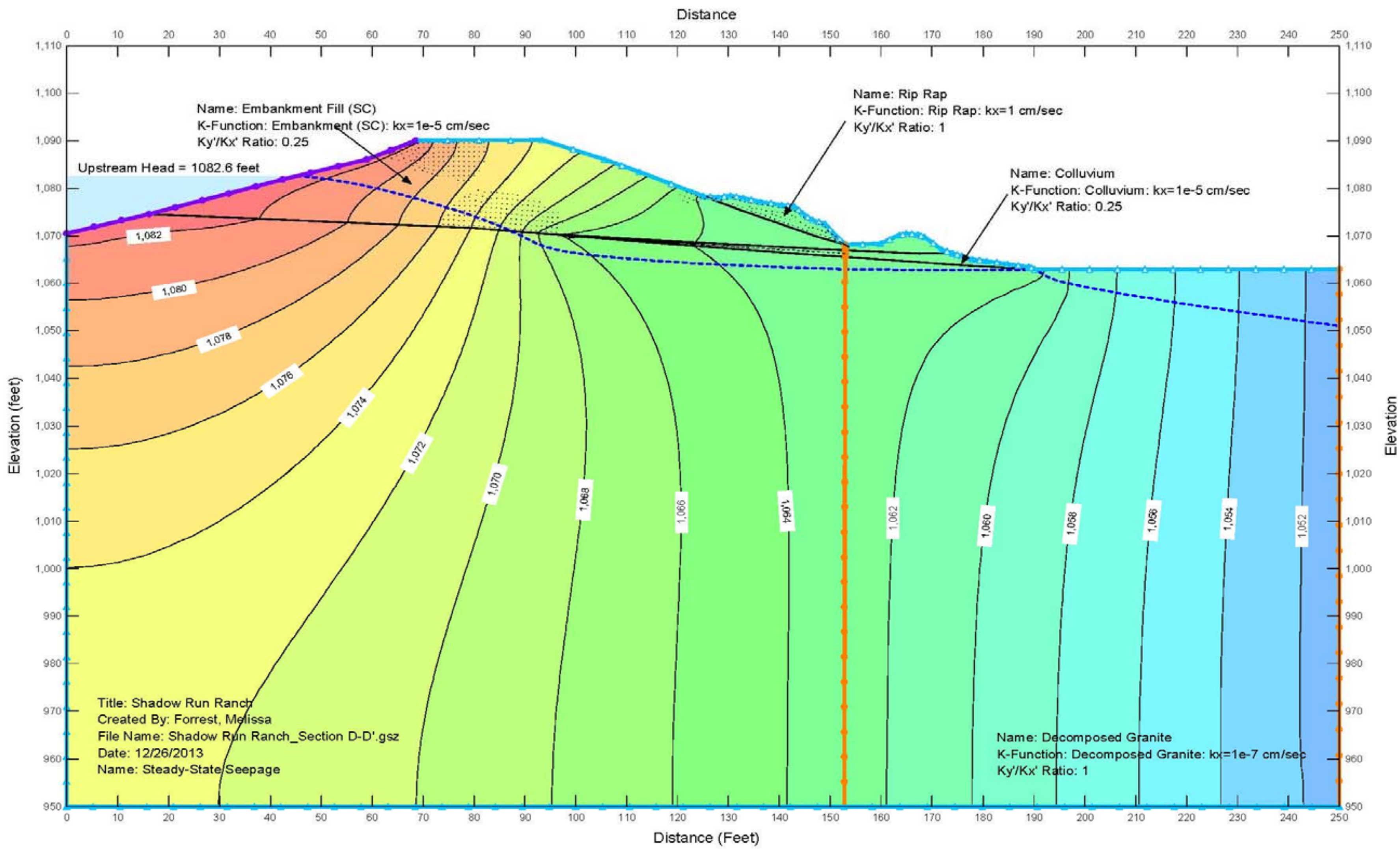
Figure 11



Seepage Analyses of Cross Section B-B' **Steady State Conditions**	URS
Shadow Run Ranch **Pauma Valley, California**	Date: 12/30/2013 **Figure 12**



Seepage Analyses of Cross Section C-C' **Steady State Conditions**	URS
Shadow Run Ranch **Pauma Valley, California**	Date: 12/30/2013 **Figure 13**

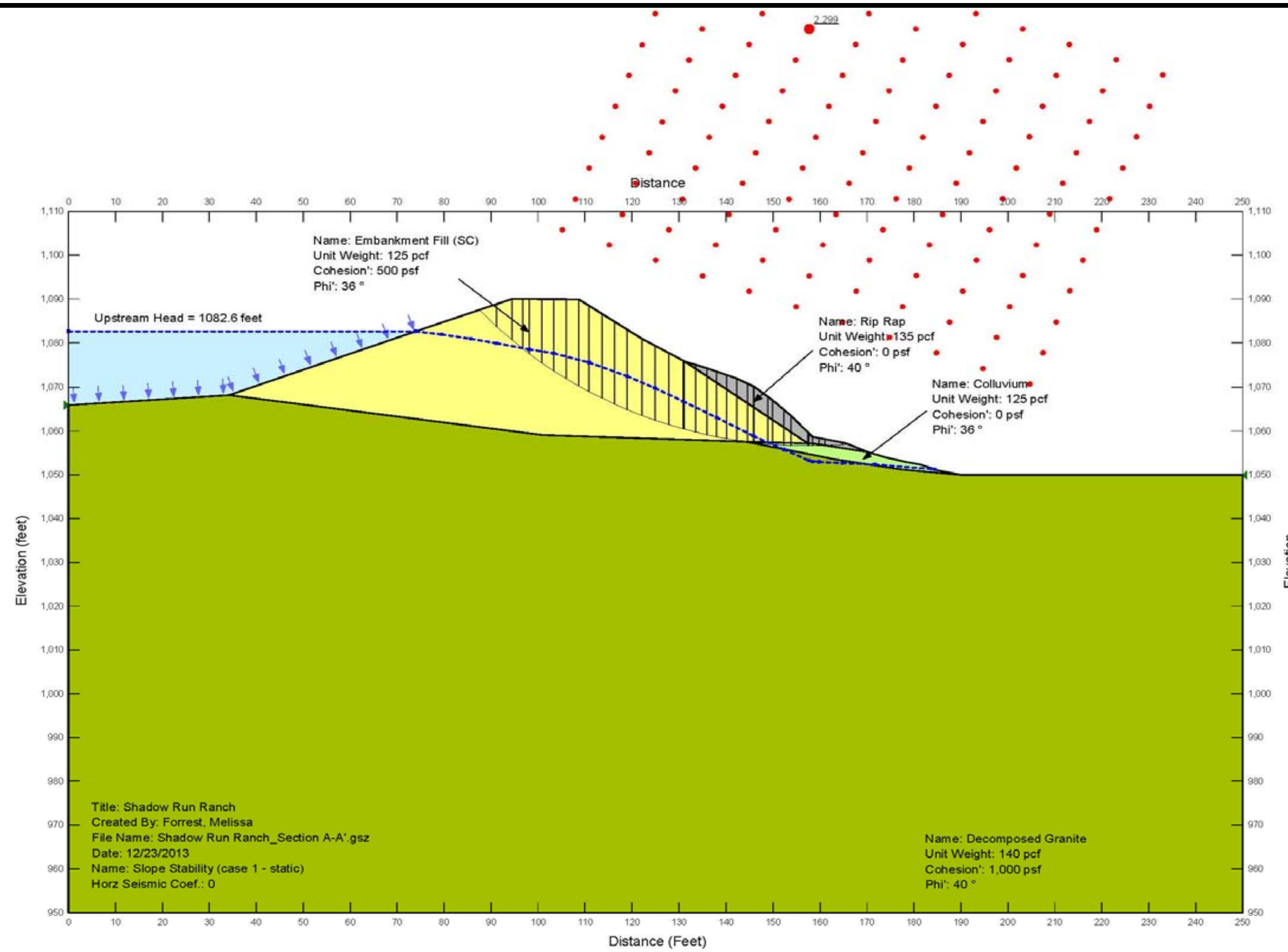


Seepage Analyses of Cross Section D-D'
Steady State Conditions

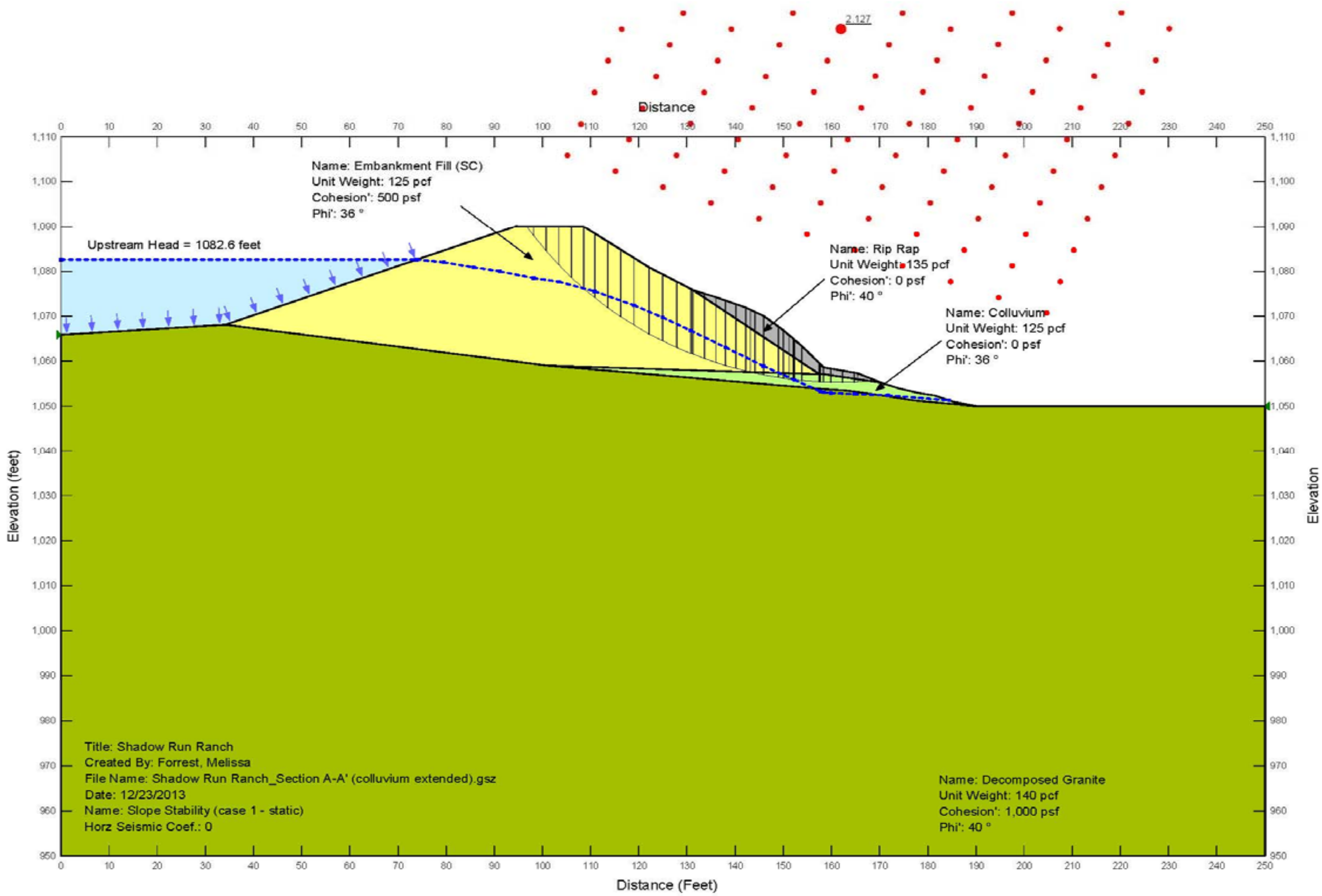
Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

Figure 14



Slope Stability of Cross Section A-A' Static Conditions	URS
Shadow Run Ranch Pauma Valley, California	Date: 12/30/2013 Figure 15



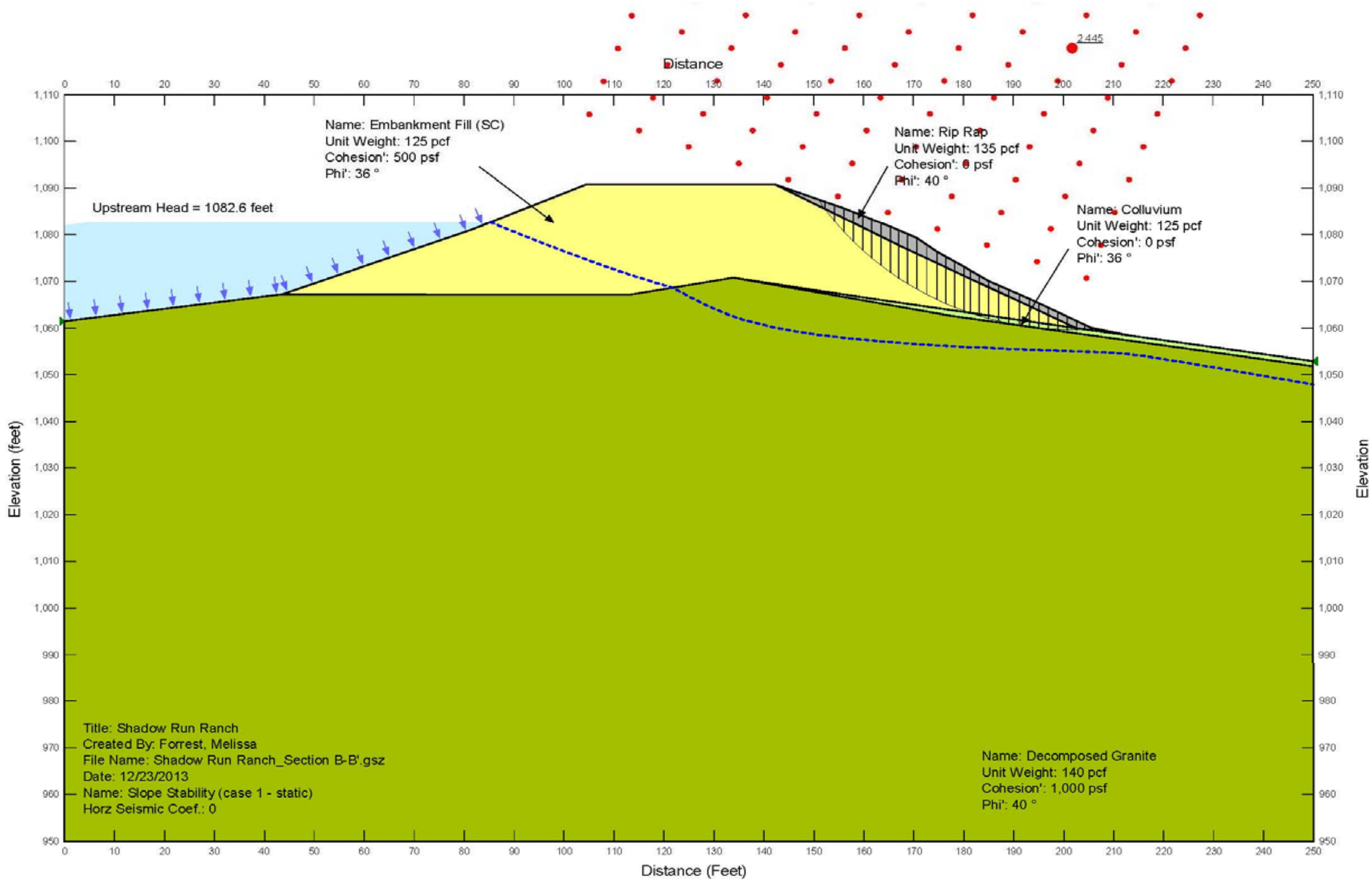
Slope Stability of Cross Section A-A'
(Extended Colluvium Layer) - Static Conditions

URS

Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

Figure 16



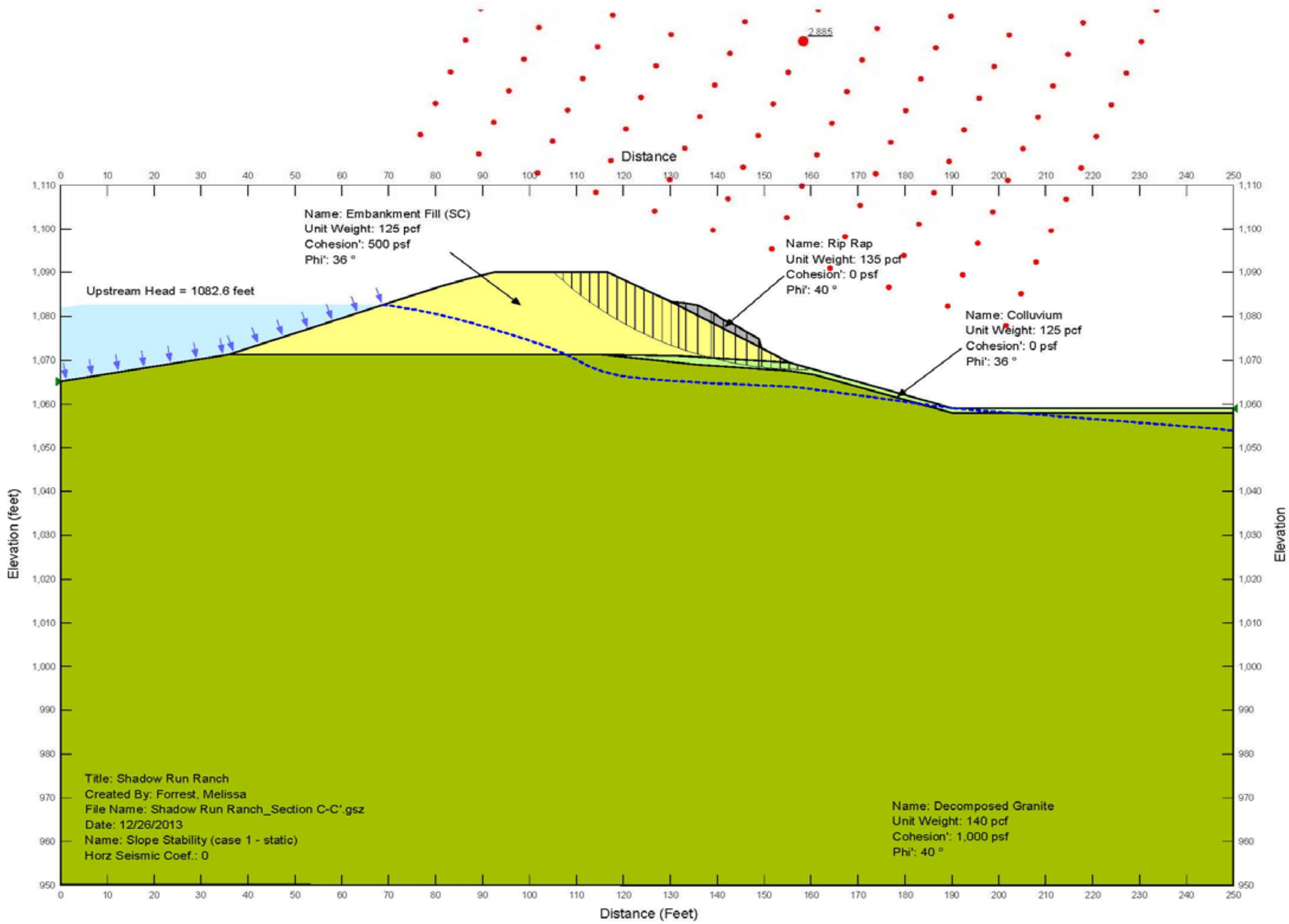
Slope Stability of Cross Section B-B'
Static Conditions

URS

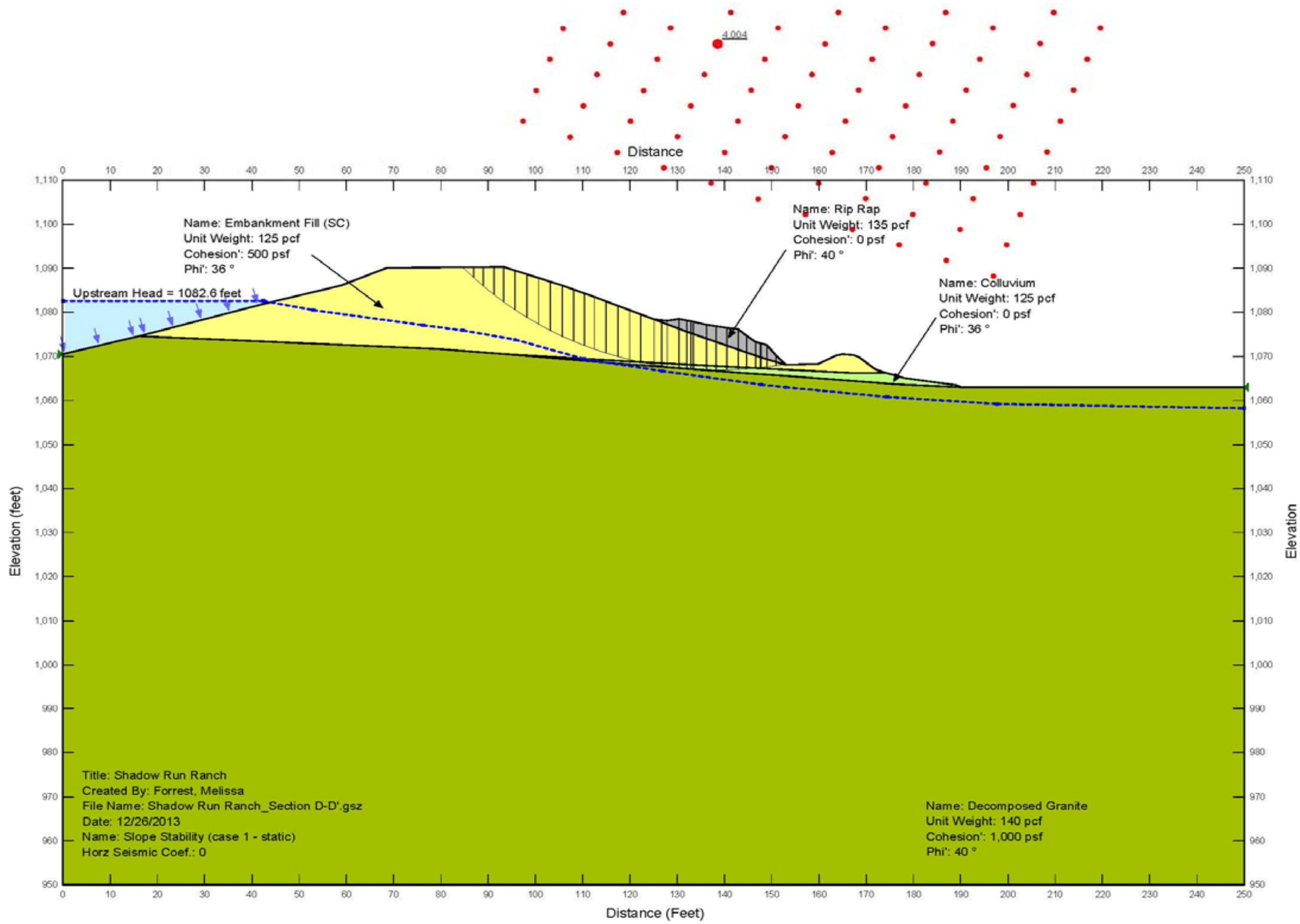
Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

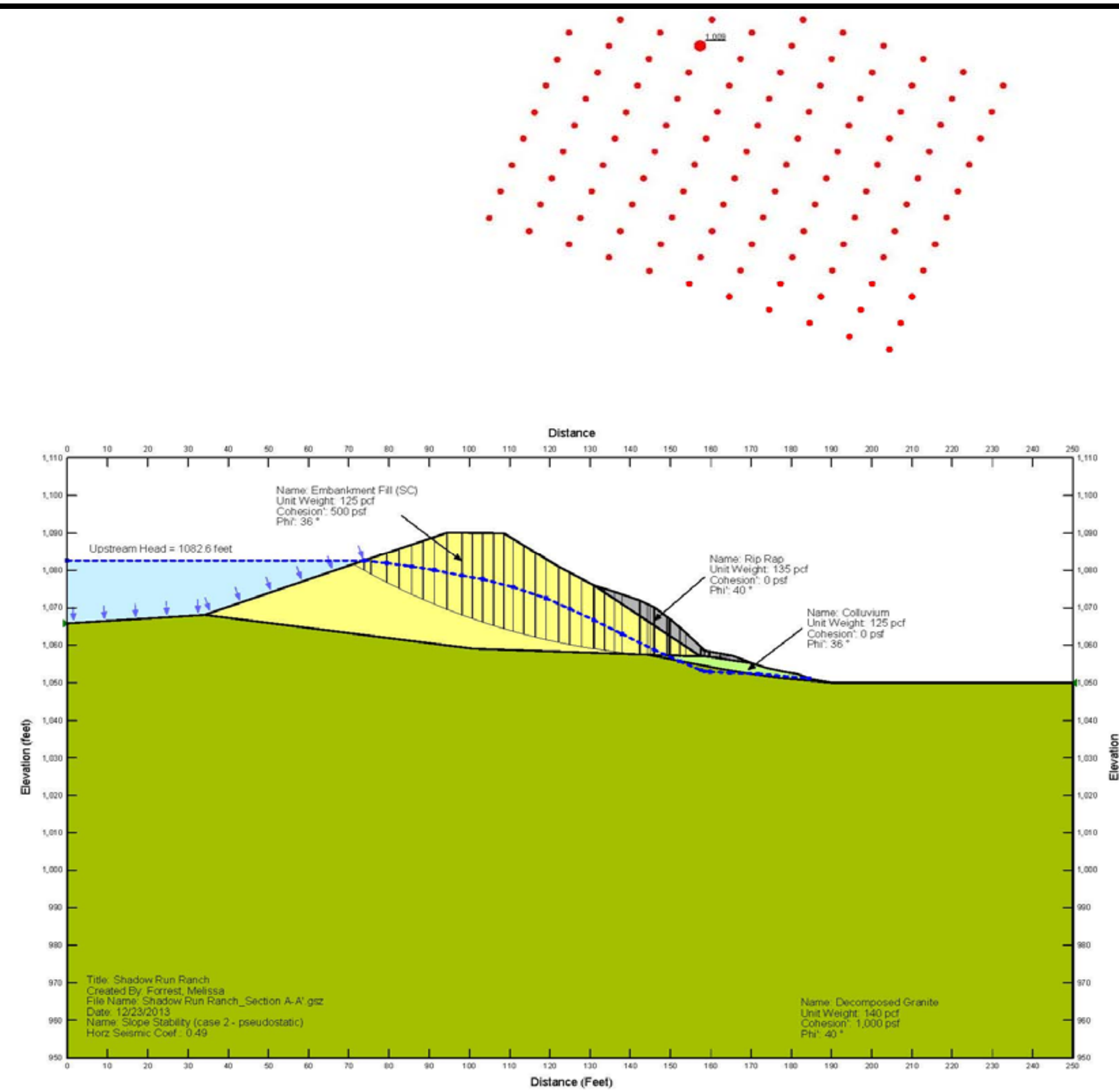
Figure 17



Slope Stability of Cross Section C-C' **Static Conditions**	URS
Shadow Run Ranch **Pauma Valley, California**	Date: 12/30/2013 **Figure 18**



Slope Stability of Cross Section D-D' **Static Conditions**	URS
Shadow Run Ranch **Pauma Valley, California**	Date: 12/30/2013 **Figure 19**



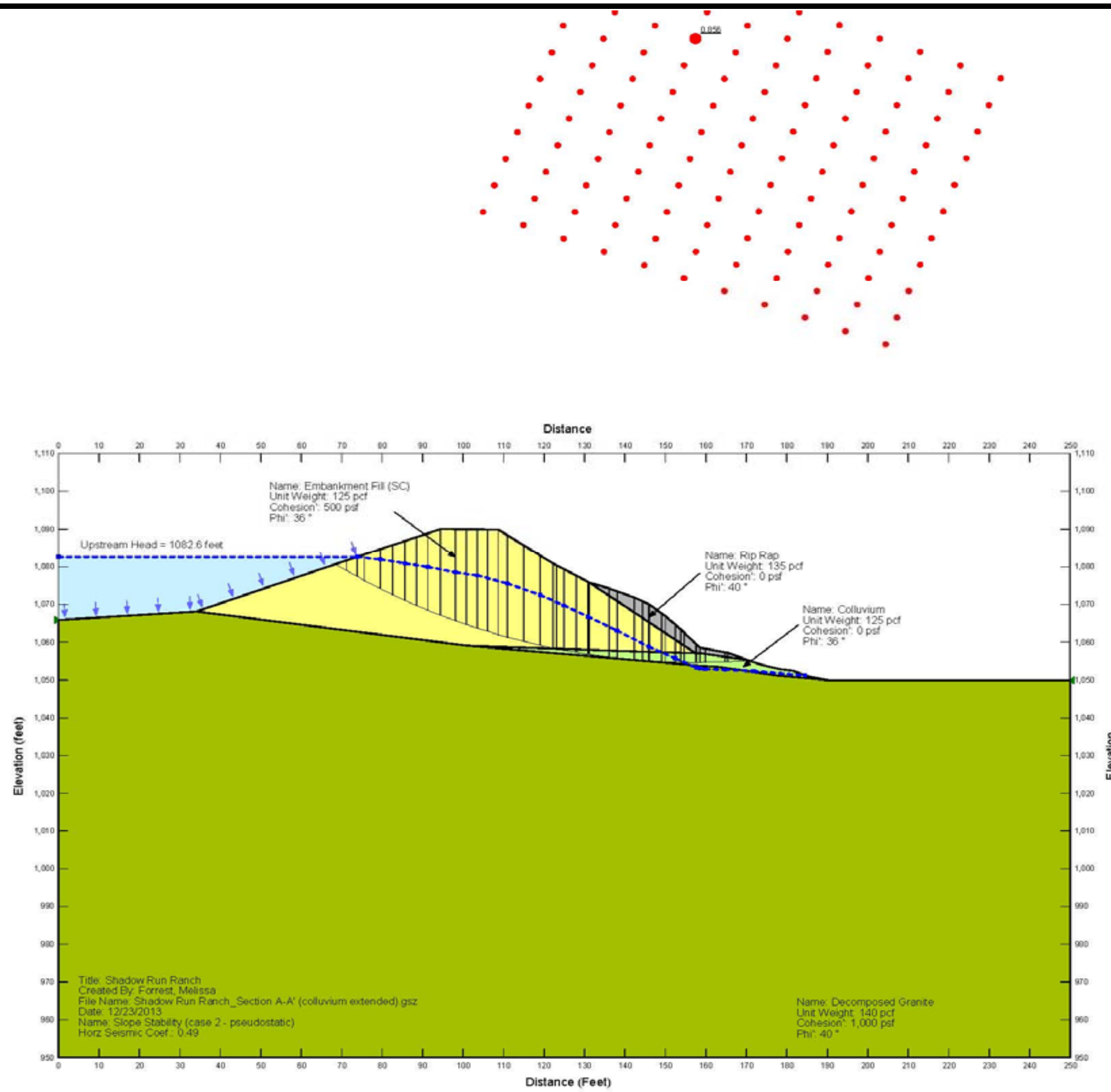
Slope Stability of Cross Section A-A'
Pseudo-Static Conditions

URS

Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

Figure 20

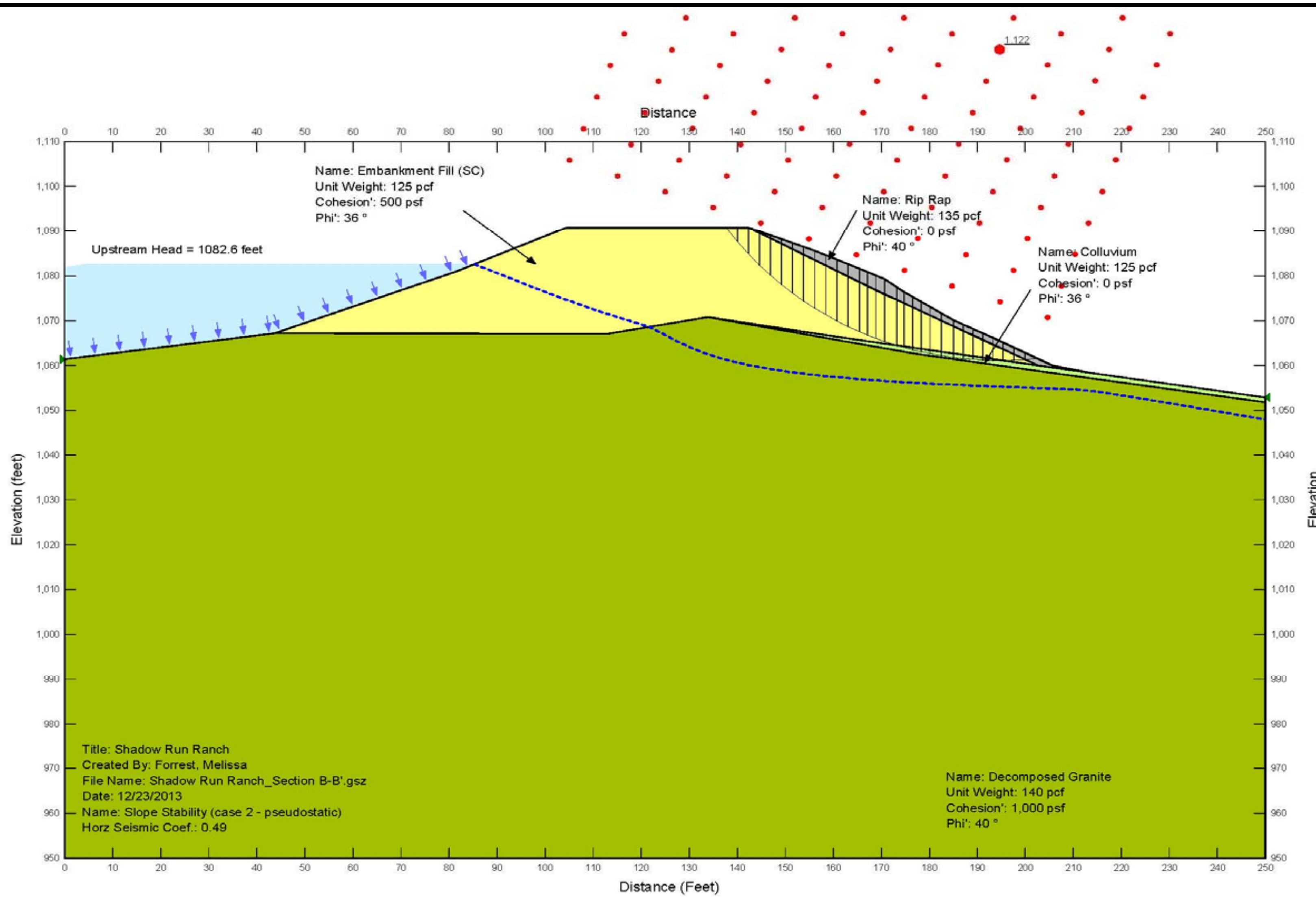


Slope Stability of Cross Section A-A'
(Extended Colluvium Layer) - Pseudo-Static Conditions

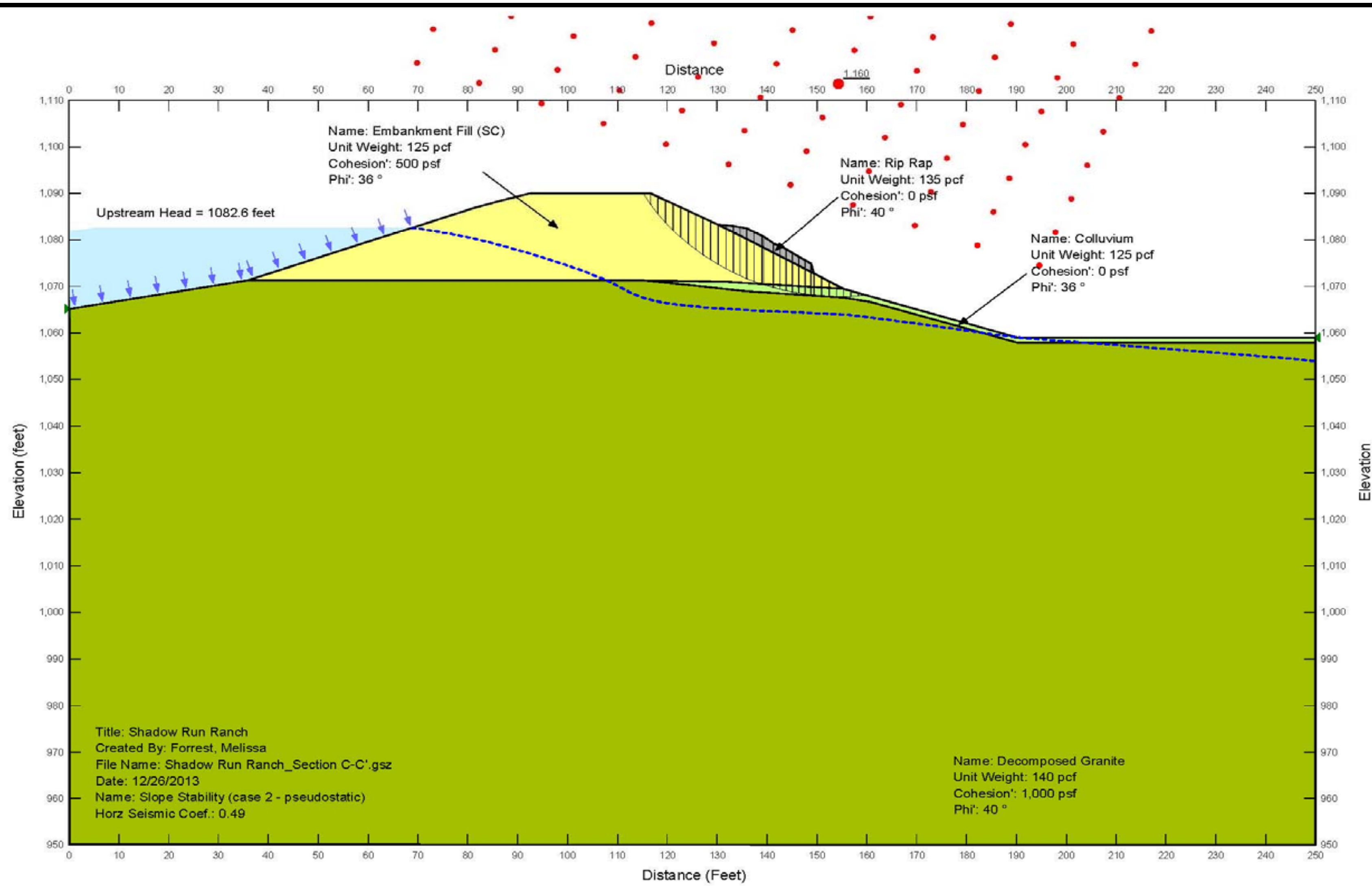
Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

Figure 21



Slope Stability of Cross Section B-B' Pseudo-Static Conditions	URS
Shadow Run Ranch Pauma Valley, California	Date: 12/30/2013 Figure 22



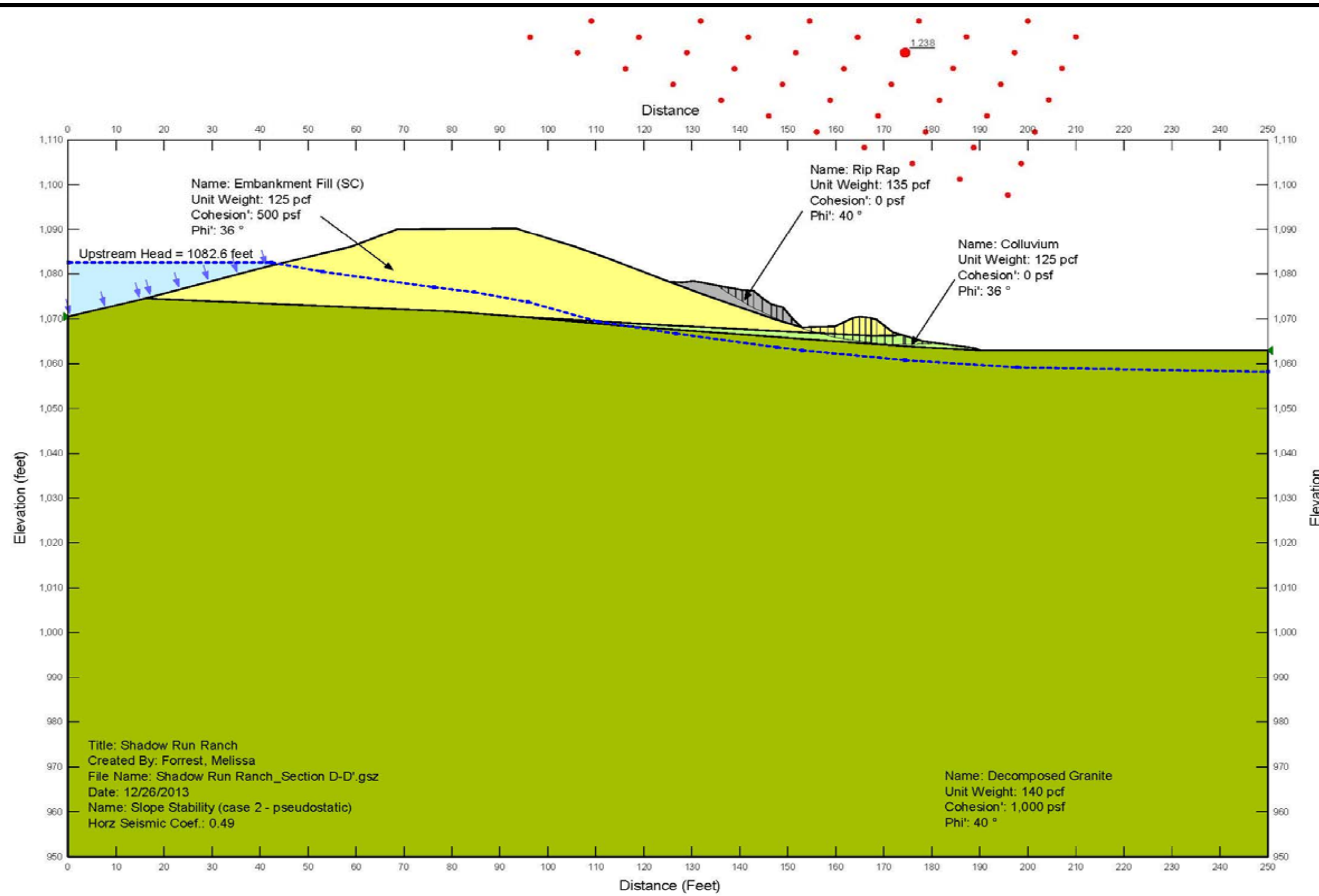
Slope Stability of Cross Section C-C'
Pseudo-Static Conditions

URS

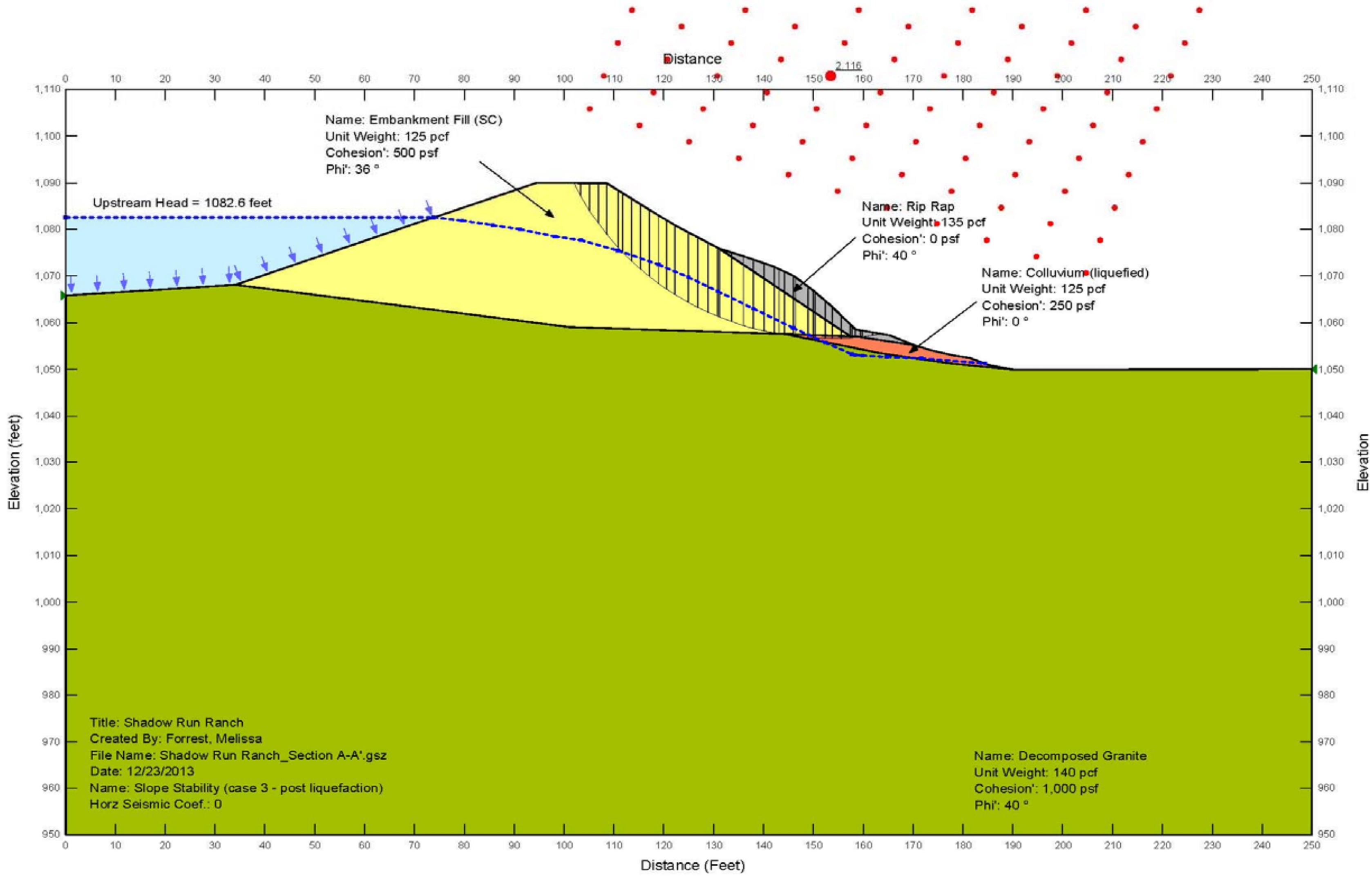
Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

Figure 23



Slope Stability of Cross Section D-D' **Pseudo-Static Conditions**	URS
Shadow Run Ranch **Pauma Valley, California**	Date: 12/30/2013 **Figure 24**



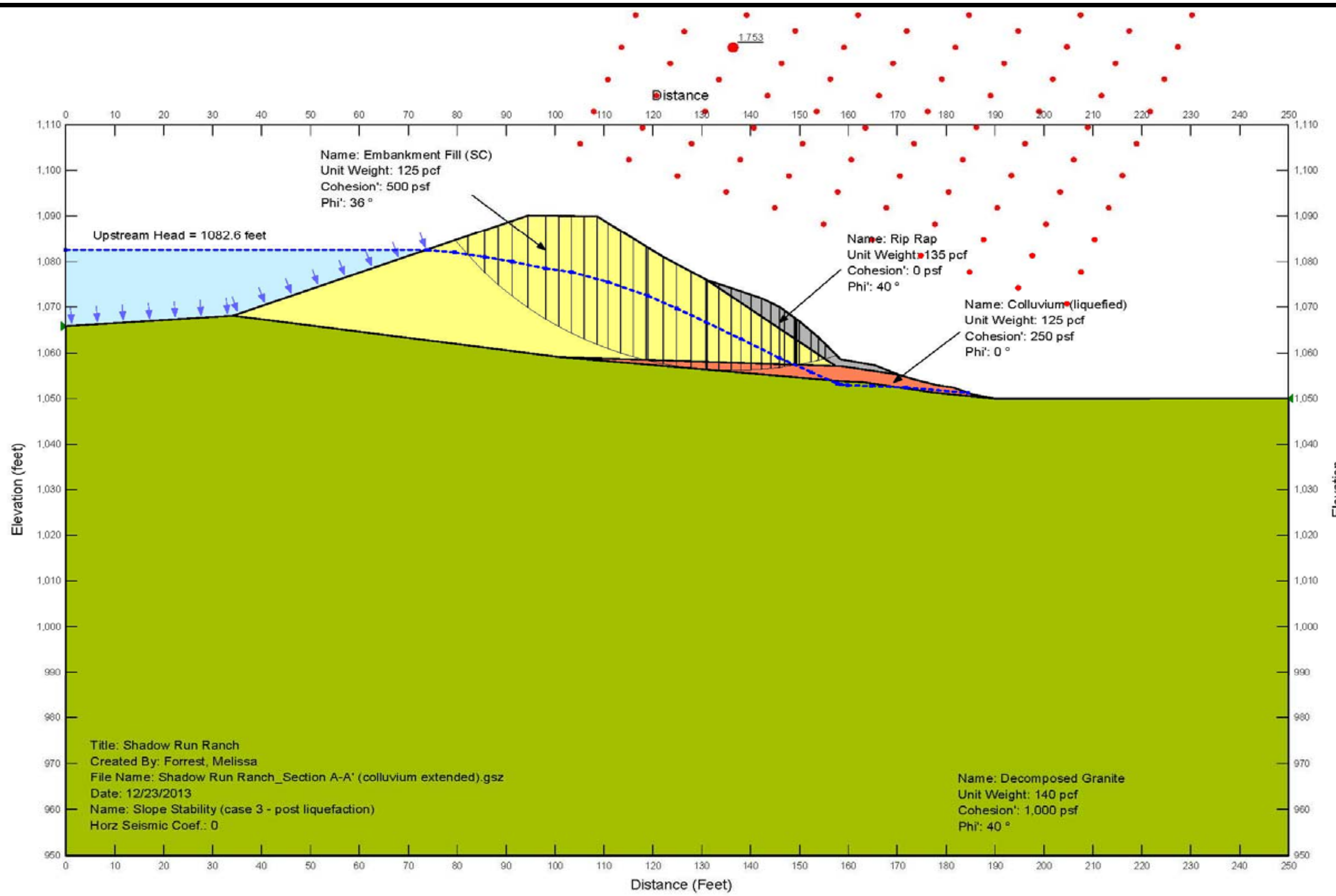
Slope Stability of Cross Section A-A'
(Circular Slip Surface) - Post-Liquefaction Conditions

URS

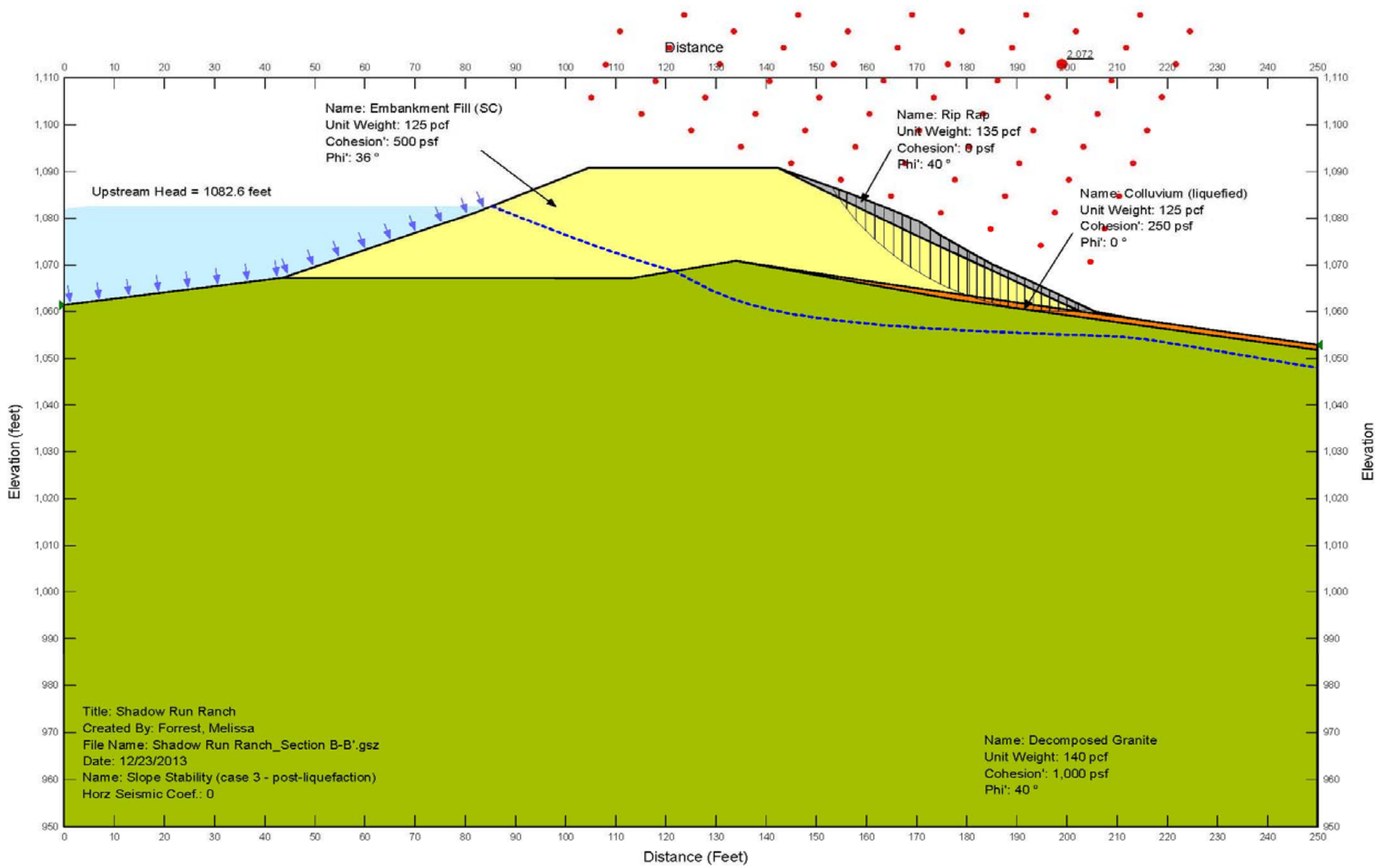
Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

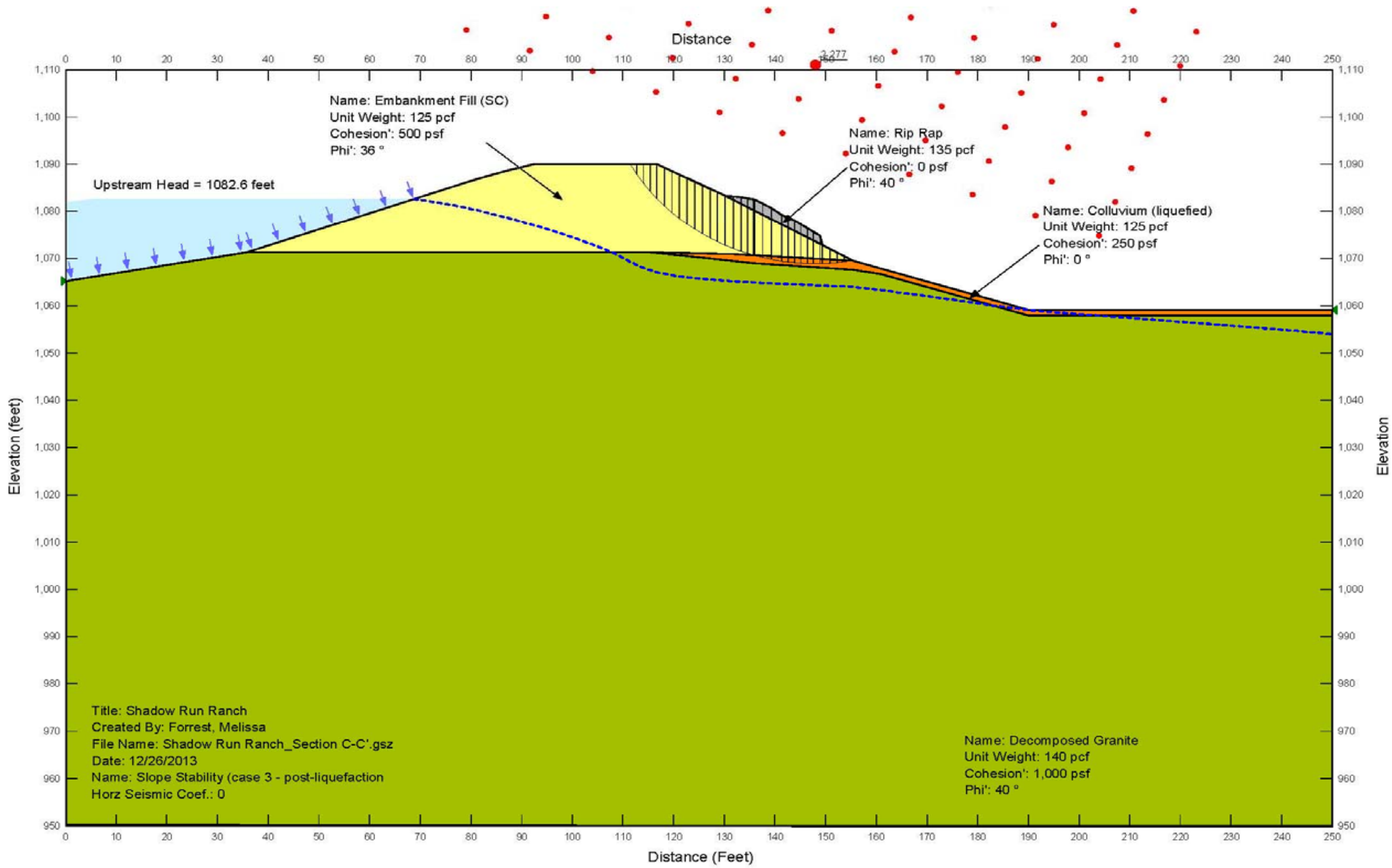
Figure 25



Slope Stability of Cross Section A-A' (Circular Slip Surface with Extended Colluvium Layer) - Post-Liquefaction Conditions	URS
Shadow Run Ranch Pauma Valley, California	Date: 12/30/2013 **Figure 26**



Slope Stability of Cross Section B-B' **(Circular Slip Surface) - Post-Liquefaction Conditions**	URS
Shadow Run Ranch **Pauma Valley, California**	Date: 12/30/2013 **Figure 27**



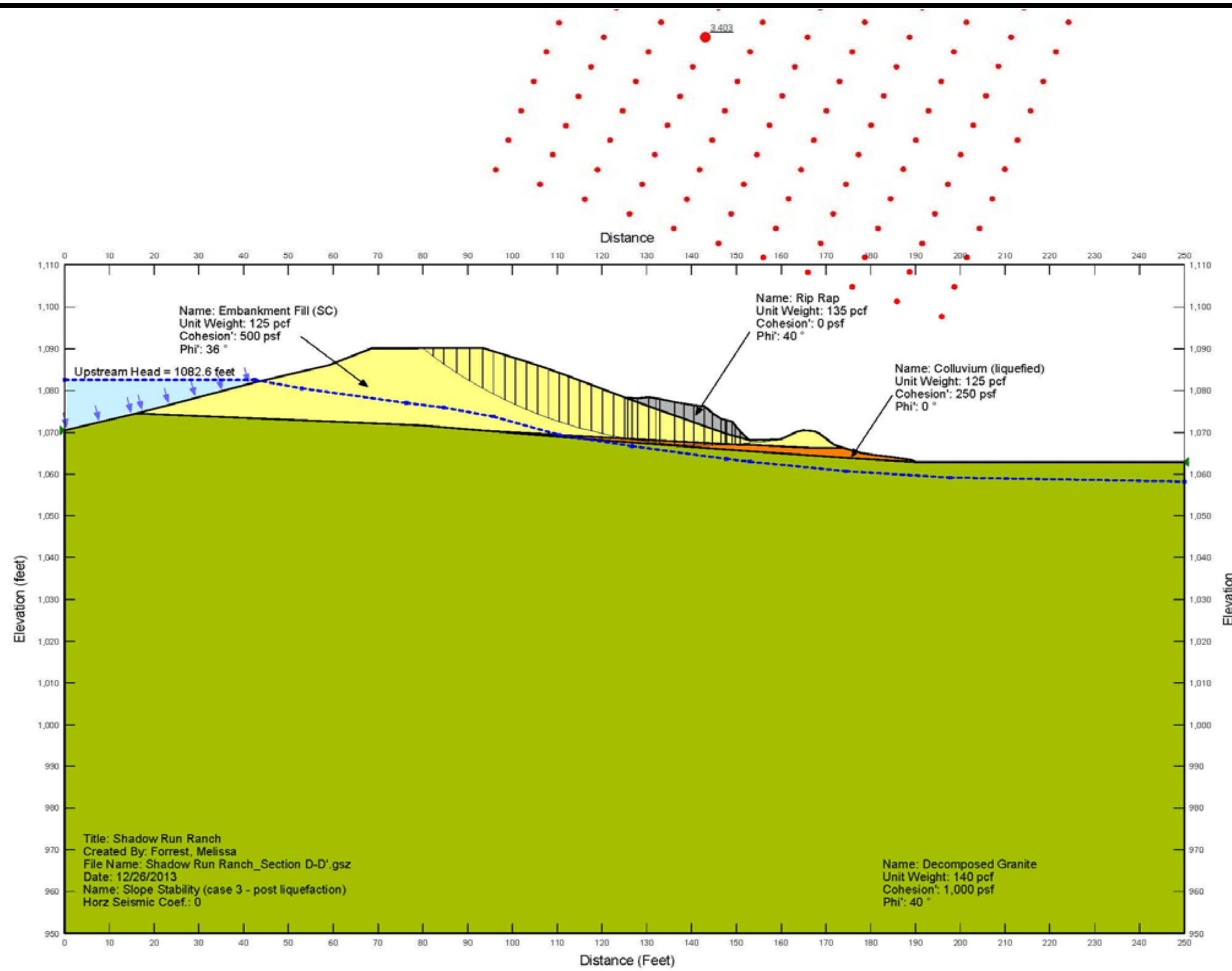
Slope Stability of Cross Section C-C'
(Circular Slip Surface) - Post-Liquefaction Conditions

URS

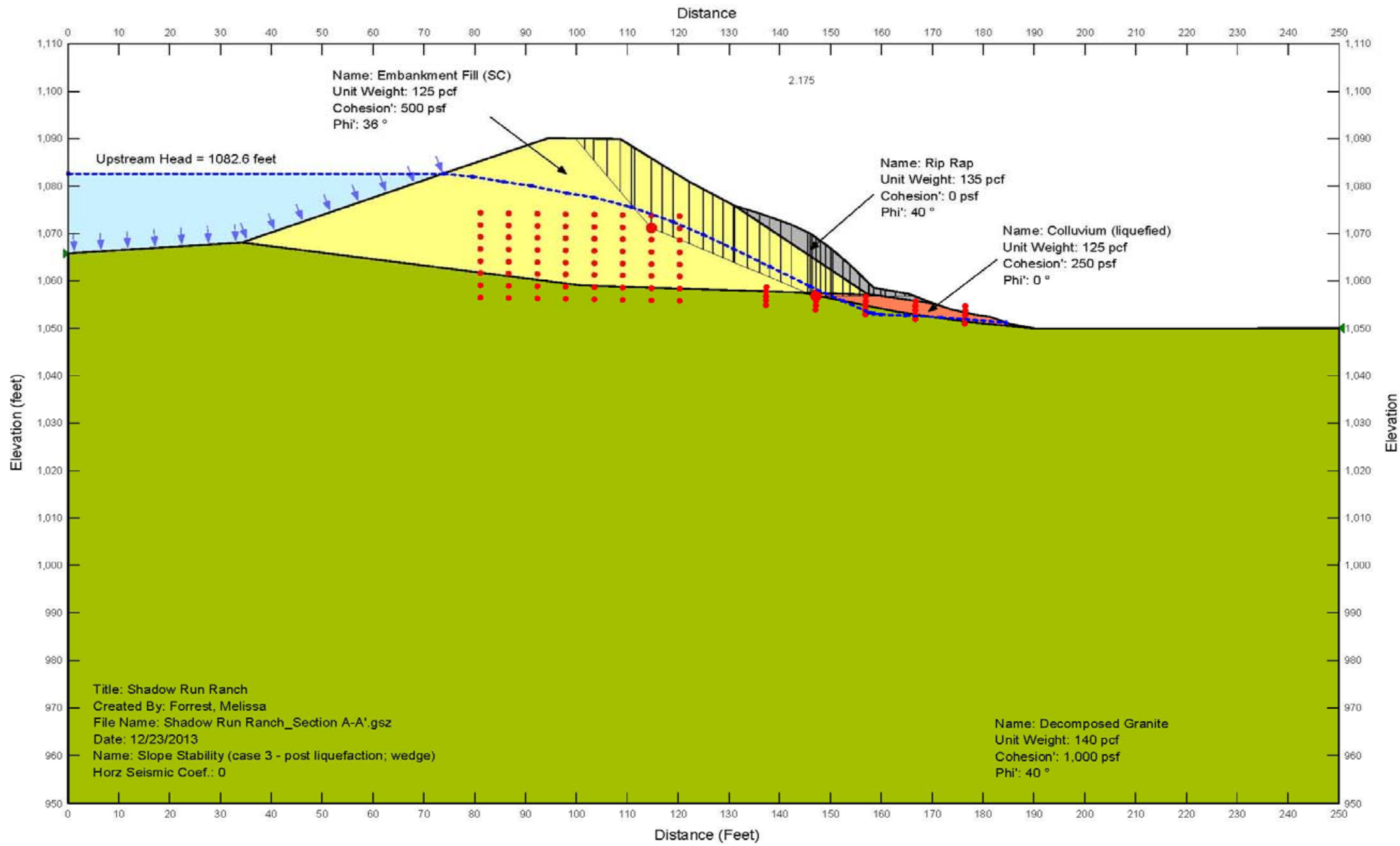
Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

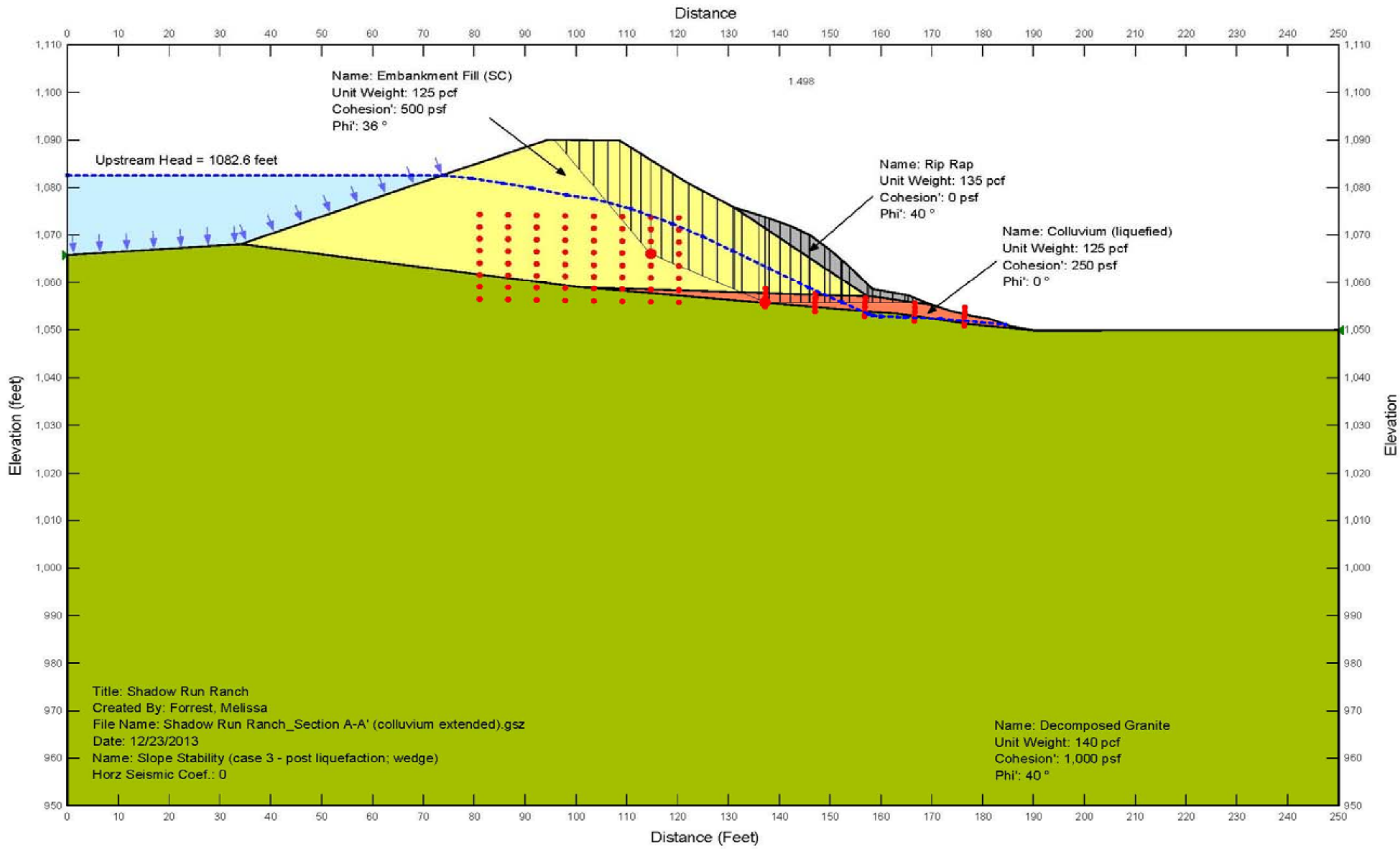
Figure 28



Slope Stability of Cross Section D-D' **(Circular Slip Surface) - Post-Liquefaction Conditions**	URS
Shadow Run Ranch **Pauma Valley, California**	Date: 12/30/2013 **Figure 29**



Slope Stability of Cross Section A-A' **(Wedge Slip Surface) - Post-Liquefaction Conditions**	URS
Shadow Run Ranch **Pauma Valley, California**	Date: 12/30/2013 **Figure 30**



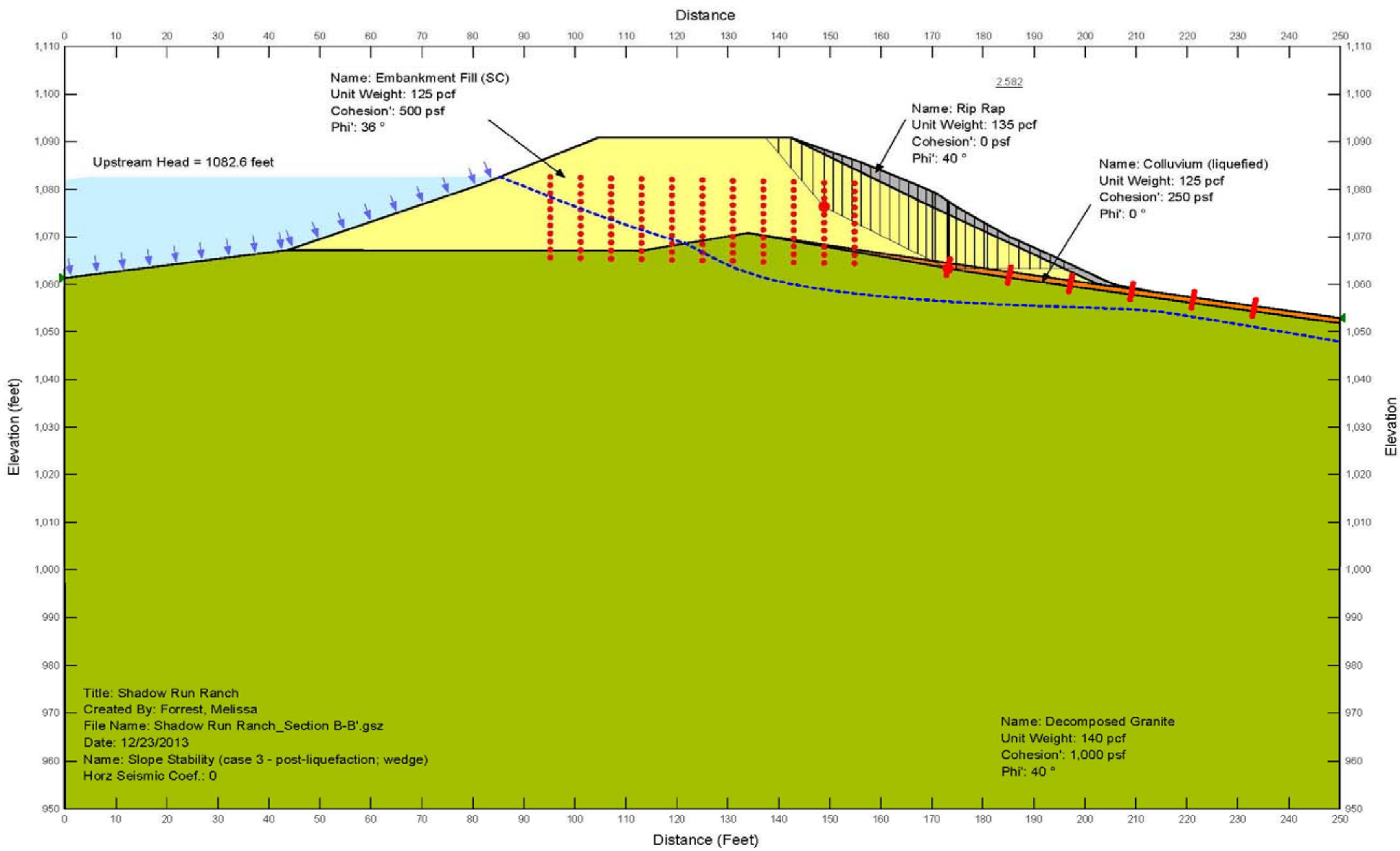
Slope Stability of Cross Section A-A'
(Wedge Slip Surface with Extended Colluvium Layer) - Post-Liquefaction Conditions

URS

Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

Figure 31



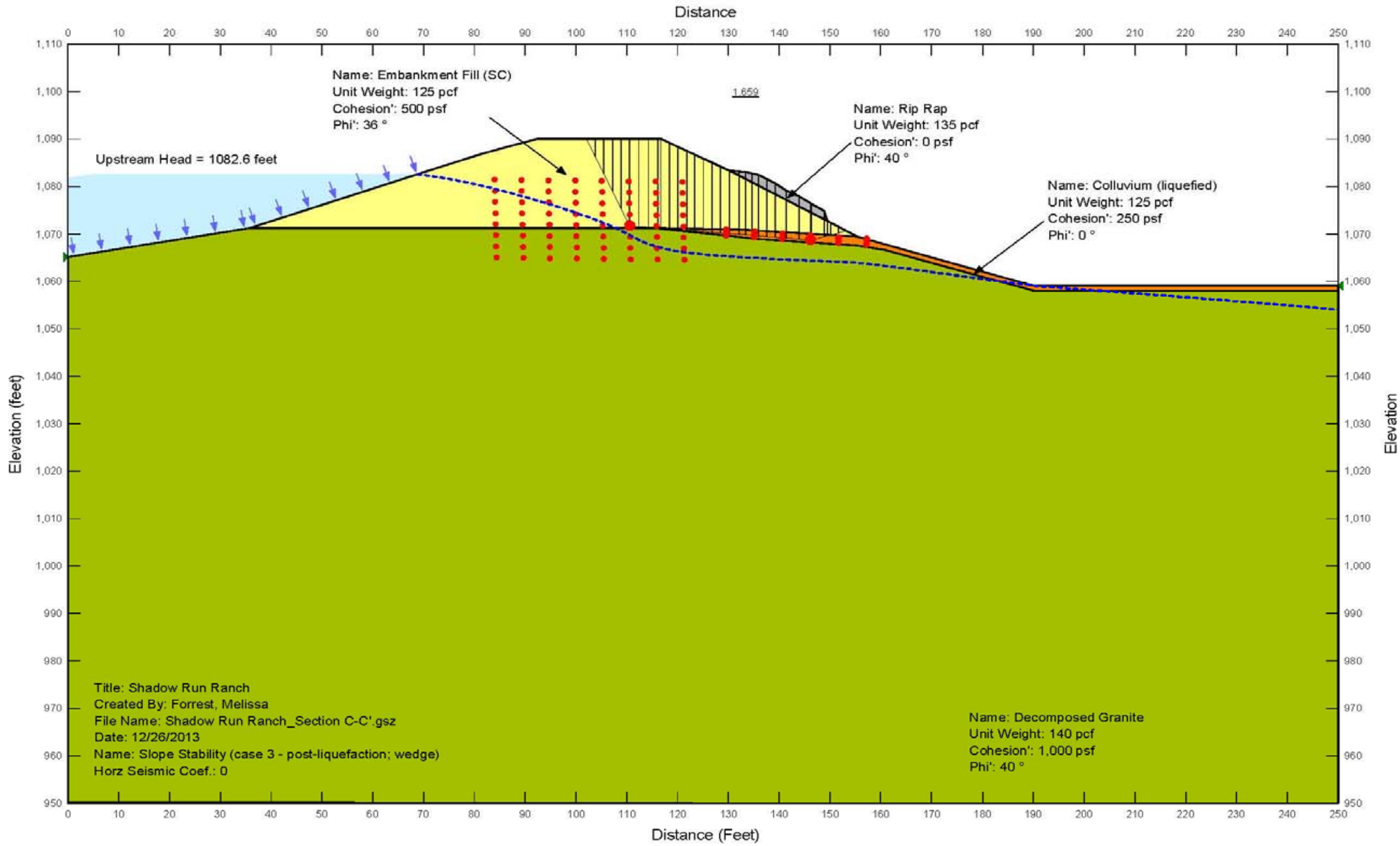
Slope Stability of Cross Section B-B'
(Wedge Slip Surface) - Post-Liquefaction Conditions

URS

Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

Figure 32



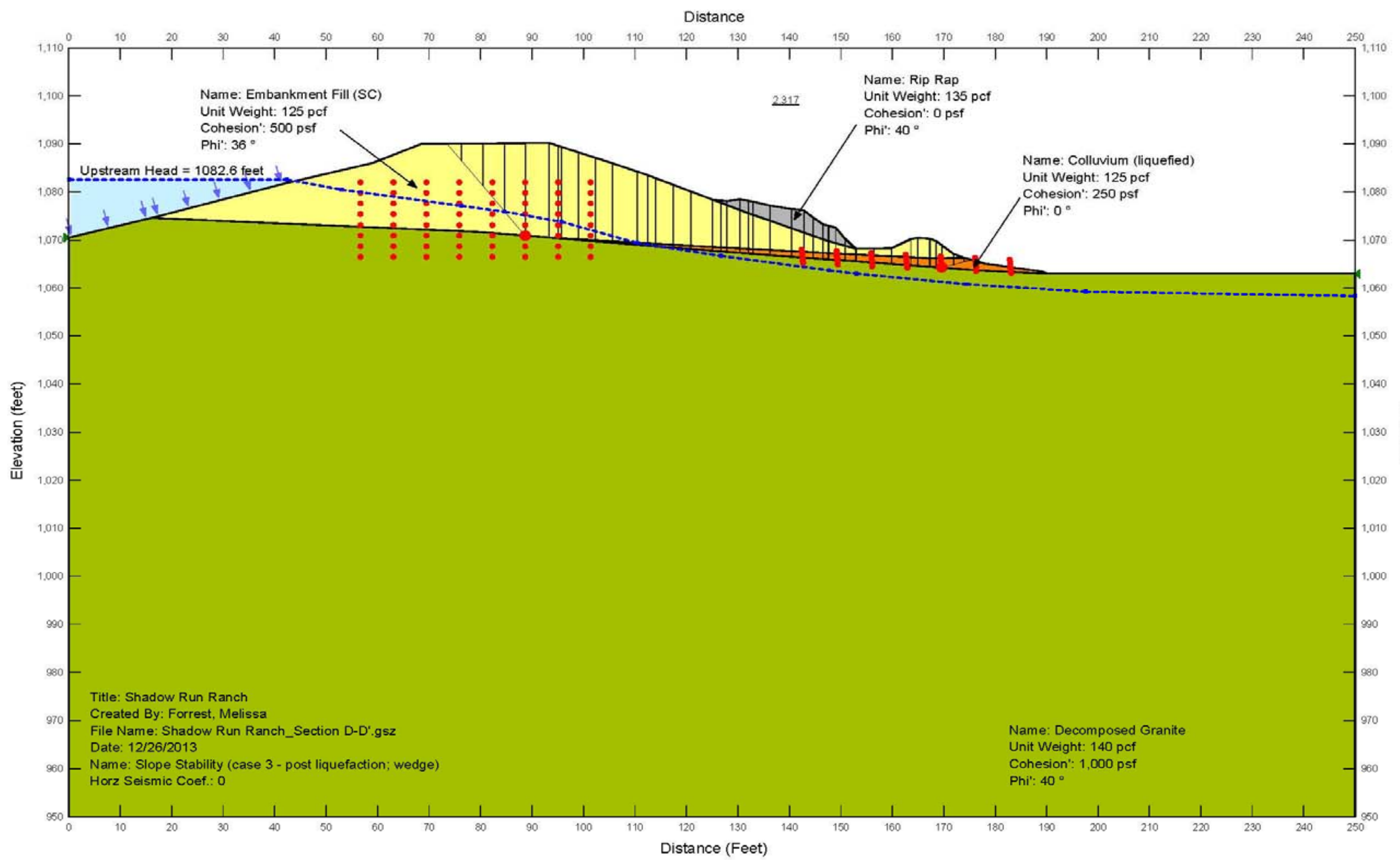
Slope Stability of Cross Section C-C'
(Wedge Slip Surface) - Post-Liquefaction Conditions

URS

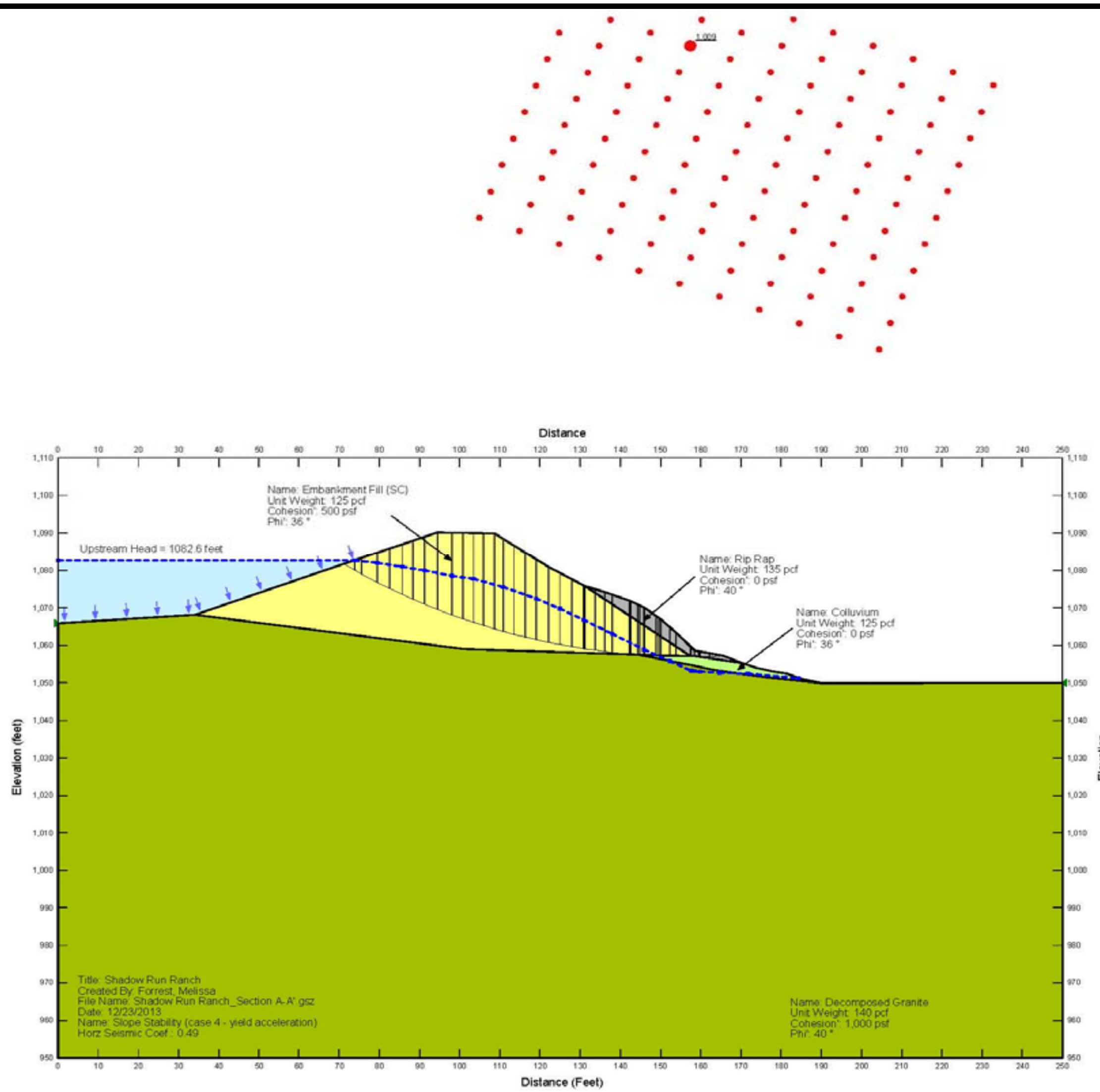
Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

Figure 33



Slope Stability of Cross Section D-D' **(Wedge Slip Surface) - Post-Liquefaction Conditions**	**URS**
Shadow Run Ranch **Pauma Valley, California**	Date: 12/30/2013 **Figure 34**



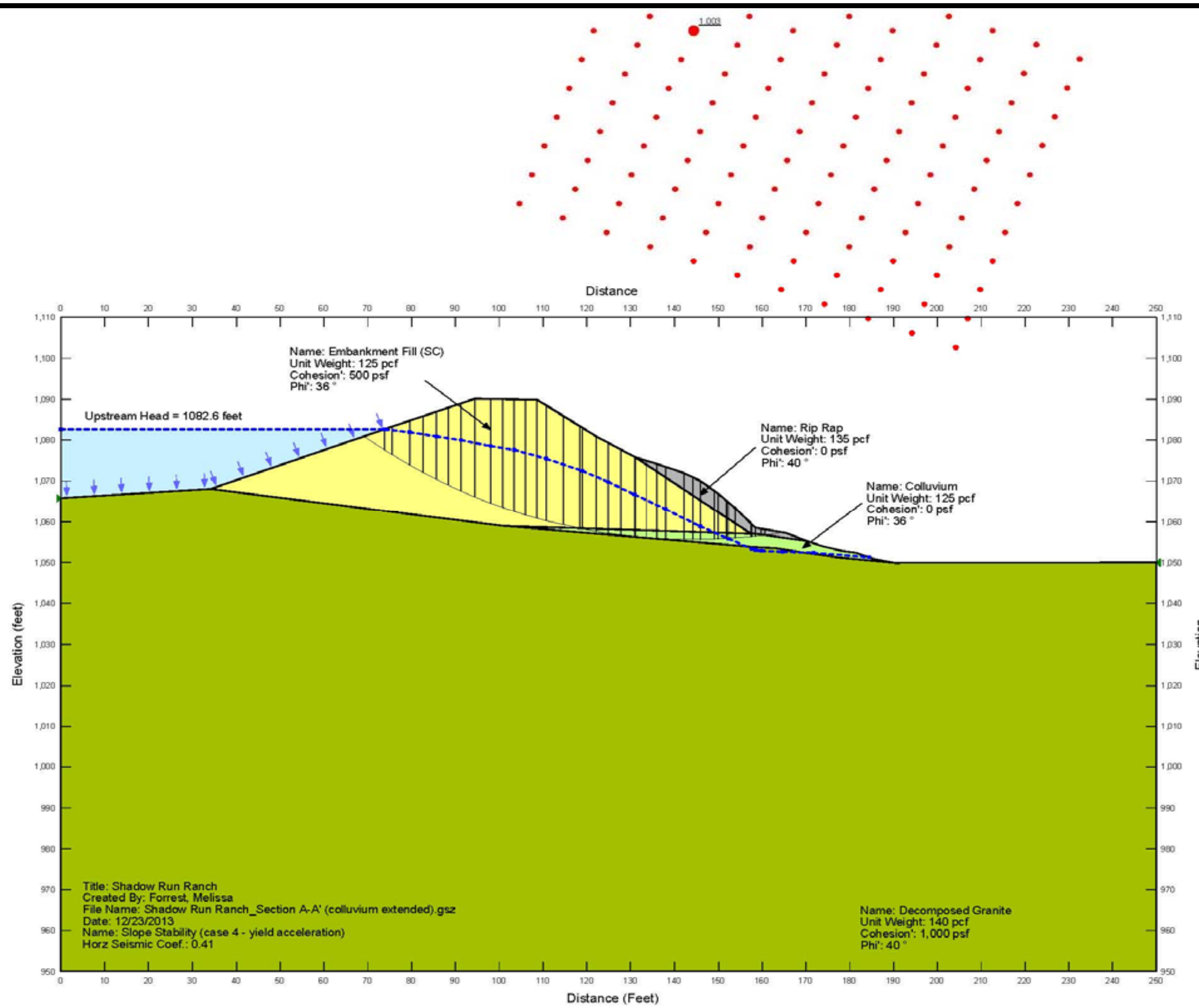
Slope Stability of Cross Section A-A'
Yield Acceleration

URS

Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

Figure 35



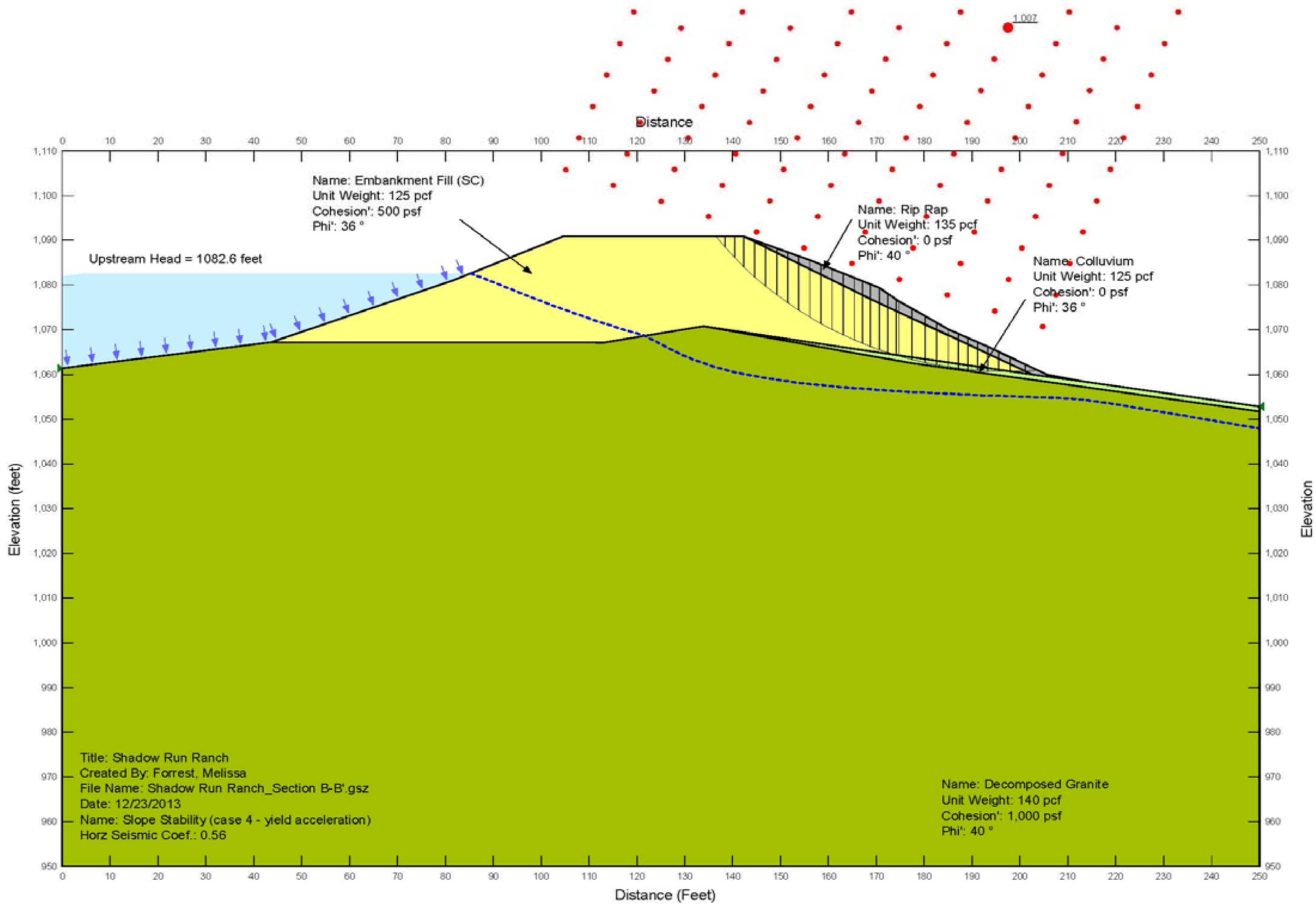
Slope Stability of Cross Section A-A'
(Extended Colluvium Layer) - Yield Acceleration

URS

Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

Figure 36



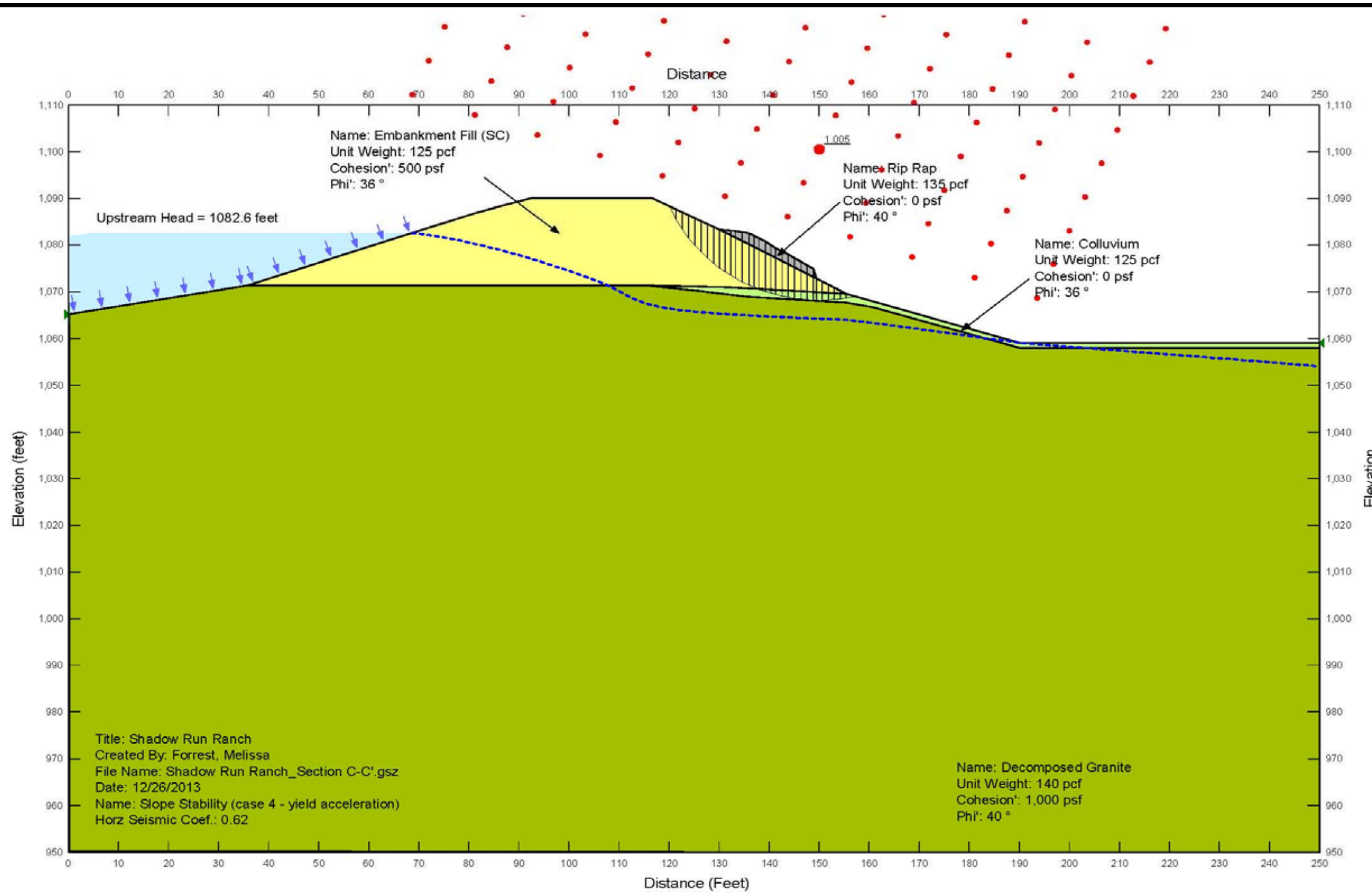
Slope Stability of Cross Section B-B'
Yield Acceleration

URS

Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

Figure 37



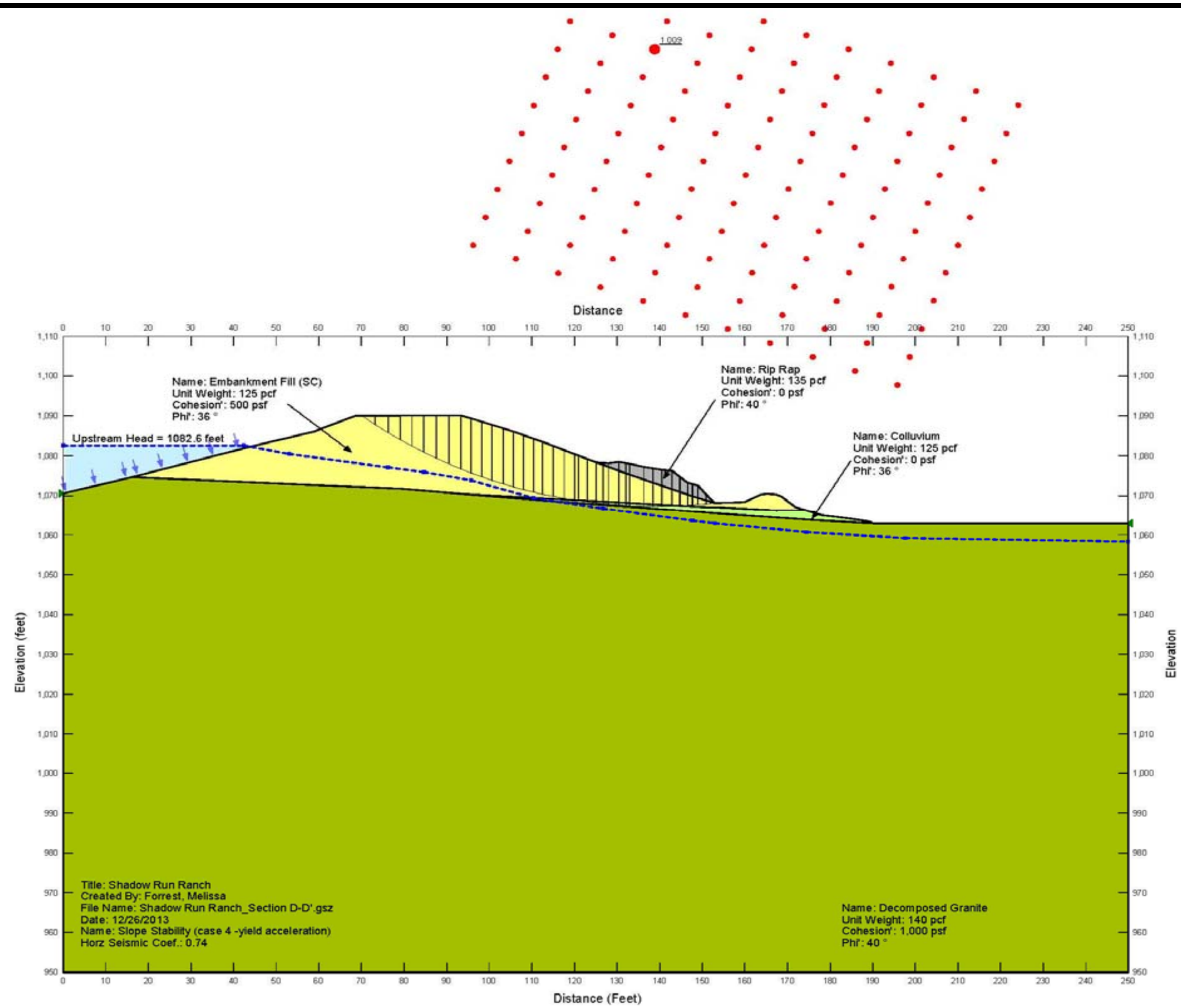
Slope Stability of Cross Section C-C'
Yield Acceleration

URS

Shadow Run Ranch
Pauma Valley, California

Date: 12/30/2013

Figure 38



Slope Stability of Cross Section D-D'
Yield Acceleration

Shadow Run Ranch
Pauma Valley, California

URS

Date: 12/30/2013

Figure 39

The subsurface explorations were performed between September 17 and 26, 2013. The explorations included advancing five (5) test borings (named Borings B-1, -2, -2A, -3 and -4) with a truck-mounted Becker rig capable of closed/open bit drilling, SPT\Mod Cal sampling and rock coring, if required. The borings were located along the existing vehicle access road around the reservoir. Eleven test pits (named TP-1 through TP-11) were excavated with a rubber tired back-hoe. Figure 3 provides the locations of the borings and test pits. Logs of the test borings and test pits are presented in Appendix A as Figures A-2 through A-13. A Key to Logs is presented as Figure A-1 in Appendix A. Photographs of the field explorations are included below.

All of the borings were drilled using the Becker Hammer rig in the open bit mode. BPT (open bit) advancement rates were recorded by the URS geologist onsite. In the open bit mode, we collected samples using an SPT sampler and a modified California sampler alternating at approximately 5 foot intervals.

URS geologists obtained relatively intact samples alternating between a modified California drive sampler (2.5-inch inside diameter) with a stainless steel liner, or using a Standard Penetration Test (SPT) sampler (1.375-inch inside diameter and 2-inch outside diameter) in general accordance with ASTM D-1586. Both samplers were driven 18-inches or until sampler refusal into material at the bottom of the boring by a 140-pound hammer falling 30-inches. Grab samples were also collected from the cuttings generated from each advancement of the BPT open bit (typically 5 foot drives). Soil samples were logged in accordance with the Unified Soil Classification System (USCS).

Three of the borings (B-1, B-2A and B-4) were constructed as "open hole" piezometers (i.e., stand pipe type monitoring wells). Well construction consisted of 2-inch diameter blank PVC pipe with a length of slotted screen placed to the bottom of the hole, then backfilled with sand and a bentonite seal in accordance with County of San Diego requirements. A locking cap was installed on the top of each well and a flush-mount traffic cover was constructed at the ground surface. The remaining borings were tremie backfilled with bentonite cement. Drill cuttings were spread and disposed of onsite. Groundwater levels were measured periodically for the duration of the geotechnical study.

Becker hammer drill rig at Boring B-2. The reservoir level is about at the proposed final level.

Positioning the 140-lb hammer for SPT sampling (left). The Becker "open bit" on the right.

Boring B-1: Sample 13 collected with an SPT sampler at 30.5 feet. Sample was collected at the top of highly/completely weathered granitic rock.

Boring B-2: Sample 8 collected from the Becker cyclone at 22.5 feet. Sample was clayey sand ("dg" soil) collected at the bottom of the embankment fill.

Boring B-2: Sample 9 collected from the Becker cyclone at 24 feet. Sample was collected at the top of highly/completely weathered granitic rock, which breaks down to clayey sand with gravel-sized rock fragments. The Becker open bit allowed continuous logging of the cuttings from the cyclone.

Boring B-4: Sample 9 collected with an SPT sampler at 25 feet. Highly/completely weathered granitic rock about 6 feet below the dam foundation

Backhoe test pits were excavated at the top of the embankment. This is Test Pit TP-2, looking east.

Palm tree roots were exposed at the top of Test Pit TP-2.

Test Pit TP-2: The embankment fill soil had been placed in thin lifts. The test pit sidewalls appeared stable with no sloughing to the total depth of the excavation (about 14.5 feet bgs),

Test Pit TP-5 was excavated at the toe of the embankment fill slope.

Test Pit TP-5 exposed gravelly/cobbly clayey sand fill, overlying topsoil/colluvium. The test pit met refusal on weathered granitic boulders.

Test Pit TP-6 was in gravelly fill overlying topsoil/colluvium at the embankment toe.

Project: Shadow Run Ranch
Project Location: Pauma Valley, California
Project Number: 27661027.10000

Key to Log

Elevation feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Drives per foot	Bounce Pressure, psi	Graphic Log	MATERIAL DESCRIPTION	Well Completion Detail	REMARKS AND OTHER TESTS
1	2	3	4	5	6	7	8	9	10	11

COLUMN DESCRIPTIONS

1. **Elevation:** Elevation in feet referenced to mean sea level (MSL) or site datum.
2. **Depth:** Depth in feet below the ground surface.
3. **Sample Type:** Type of soil sample collected at depth interval shown; sampler symbols are explained below.
4. **Sample Number:** Sample identification number.
5. **Blows per foot:** Number of blows required to advance driven sampler 12 inches beyond first 6-inch interval, or distance noted, using a 140-lb hammer with a 30-inch drop.
6. **Drives per foot:** Number of blows required to advance Becker hammer open bit.
7. **Bounce Pressure:** Bounce pressure reading in pounds per square inch (psi)
8. **Graphic Log:** Graphic depiction of subsurface material encountered; typical symbols are explained below.
9. **Material Description:** Description of material encountered; may include color, moisture, grain size, and density/consistency.
10. **Well Completion Detail:** Schematic of piezometer or well installation or hole backfill; graphic symbols are explained below.
11. **Remarks and Other Tests:** Comments and observations regarding drilling or sampling made by driller or field personnel.

MC	Water content of soil sample measured in laboratory, expressed as percentage of dry weight of specimen
DUW	Dry unit weight of soil sample measured in laboratory, in pounds per cubic foot.
SA	Sieve Analysis, %<#200 sieve
COMP	Laboratory Compaction Test
DS	Direct Shear Test
PI	Plasticity Index (LL-PL), %

TYPICAL SOIL GRAPHIC SYMBOLS

- Fill
- Granitic rock

TYPICAL WELL GRAPHIC SYMBOLS

- Concrete
- 4" diameter blank PVC casing in bentonite chips
- 4" diameter blank PVC casing in filter pack No. 3 sand
- 4" diameter PVC casing with 0.020" slotted well screen in #3 sand
- Well cap

TYPICAL SAMPLER GRAPHIC SYMBOLS

- Grab sample
- 2.5" ID sampler
- Standard Penetration sampler

OTHER GRAPHIC SYMBOLS

- ∇ First water encountered at time of drilling and sampling (ATD)
- ▼ Water level measured at specified time after completion of drilling and sampling
- Minor change in material properties within a stratum
- — — Inferred or gradational contact between strata

GENERAL NOTES

1. Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive; actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.
2. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.
3. All wells were completed with a traffic-rated well vault set in concrete. Well casings were secured with locking plugs and locks.

URS

Project: Shadow Run Ranch	# Log of Boring B-1
Project Location: Pauma Valley, California	
Project Number: 27661027.10000	

Date(s) Drilled	**9/23/13**	Logged By	**D. Rector**	Checked By	**D. Schug**
Drilling Method	**Becker**	Drill Bit Size/Type	**6.6-inches, 4-inch tooth open bit**	Total Depth of Borehole	**45.1 feet**
Drill Rig Type	**Drill Systems AP1000 with Becker hammer**	Drilling Contractor	**Great West Drilling**	Approximate Surface Elevation	**1090 feet MSL**
Groundwater Level and Date Measured	**29 feet bgs (12/20/13)**	Sampling Method(s)	**SPT/2.5" ID/Grab**	Hammer Data	**140 lbs/30-inch drop**
Borehole Completion	**Bentonite chips**	Location	**N 33.34907 W -117.00983**		

Elevation feet	Depth, feet	SAMPLES: Type	SAMPLES: Number	SAMPLES: Blows per foot	SAMPLES: Drives per foot	SAMPLES: Bounce Pressure, psi	Graphic Log	MATERIAL DESCRIPTION	Well Completion Detail	REMARKS
1090	0				14			FILL Moist, brown to reddish brown, clayey sand with few angular gravels (decomposed granitic soil)		
					19	14				
			1							
					19	12				
					19	14				
					26					
1085	5		2							
				46						MC(9)
					20	15				
			3		21	16				
					21	16				
					19	16				
			4							
				50						
1080	10									MC(10), DUW(122), SA(17)
					40	16				
					48	16				
					46	16				
					57	17		Becomes gray, medium grained clayey sand with few gravels		
			5							
					60	17				
1075	15		6							
				32						MC(10), PI(19)
					67	17				
			7		71	18				
					59	18				
			8							
1070	20			>50						
					57	18				
					56	18				
			9							
					43	18				
					46	18				
					41	18				
1065	25		10							
				25						MC(11), PI(13)
					36	16				
			11		42	16				
					44	16				
			12		40	18				
				45						
1060	30									MC(12), DUW(125),

Project: Shadow Run Ranch
Project Location: Pauma Valley, California
Project Number: 27661027.10000

Log of Boring B-1

Elevation feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Drives per foot	Bounce Pressure, psi	Graphic Log	MATERIAL DESCRIPTION	Well Completion Detail	REMARKS
1060	30									SA(29), Pinhole Test
			13	39	66	18				
								GRANITIC ROCK Highly to completely weathered, dense to very dense. Breaks down to mottled gray and reddish brown, clayey SAND (SC) with gravel-size rock fragments		
			14		75	19				
					67	19				
					55	19				
1055	35				62	17				
			15	26						
					67	18				
			16		69	18				MC(16), SA(37)
					71	19				
			17	48	70	20				
1050	40									
					67	21				
					71	22		Becomes whitish gray sand to gravel-size rock fragments		
					9	24				
					96	25				
1045	45		18	50/1"	169	29		Refusal at 45.1 feet. Near refusal in open bit, approach maximum bounce pressure at 29 psi.		
1040	50									
1035	55									
1030	60									
1025	65									

Project: Shadow Run Ranch

Project Location: Pauma Valley, California

Project Number: 27661027.10000

Log of Boring B-2

Date(s) Drilled	**9/25/13**	Logged By	**D. Rector**	Checked By	**D. Schug**
Drilling Method	**Becker**	Drill Bit Size/Type	**6.6-inches, 4-inch tooth open bit**	Total Depth of Borehole	**36.5 feet**
Drill Rig Type	**Drill Systems AP1000 with Becker hammer**	Drilling Contractor	**Great West Drilling**	Approximate Surface Elevation	**1090 feet MSL**
Groundwater Level and Date Measured	**None encountered**	Sampling Method(s)	**SPT/2.5" ID/Grab**	Hammer Data	**140 lbs/30-inch drop**
Borehole Completion	**Bentonite chips**	Location	**N 33.34884 W -117.01031**		

Elevation feet	Depth, feet	SAMPLES Type	SAMPLES Number	Blows per foot	Drives per foot	Bounce Pressure, psi	Graphic Log	MATERIAL DESCRIPTION	REMARKS
1090	0							FILL	
					9	10		Moist, brown to reddish brown, clayey sand with few angular gravels (decomposed granitic soil)	
					12	10			
			1		12	10			
					16	10		Moist, brown to reddish brown, clayey sand with few angular gravels (decomposed granitic soil)	
1085	5				13	10			
			2	25	12	10			MC(10), SA(31)
					12	11			
					13	11			
					12	11			
			3	32	12	10		Becomes gray	MC(11), DUW(122), DS, PI(21)
1080	10								
			4		32	13			
					31	16			
1075	15				37	14			
			5	31	67	13			PI(10)
					47	16			
			6		51	16			
					61	16			
			7	50/3"	81	18			MC(9),
1070	20								
					46	15			
					43	16			
					45	16			
			8		79	18			
								GRANITIC ROCK	
			9		73	19		Highly to completely weathered, dense to very dense. Breaks down to mottled gray and reddish brown, clayey SAND (SC) with gravel-size rock fragments	
1065	25		10	46	81	18		Clay rich	MC(9), SA(30)
								With gravel and boulder-size corestones	
					110	20			
			11		170	25			
					18	21			
1060	30								

Project: Shadow Run Ranch
Project Location: Pauma Valley, California
Project Number: 27661027.10000

Log of Boring B-2

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Drives per foot	Bounce Pressure, psi	Graphic Log	MATERIAL DESCRIPTION	REMARKS
1060	30							Grades to completely weathered rock. Breaks down to reddish brown, clayey SAND (SC)	
					45	21			
					51	22			
			12		66	22			
					69	22			
1055	35		13						
				40					MC(7)
								Bottom of boring at 36.5 feet	
1050	40								
1045	45								
1040	50								
1035	55								
1030	60								
1025	65								

Project: Shadow Run Ranch	**Log of Boring B-2A**
Project Location: Pauma Valley, California	Sheet 1 of 1
Project Number: 27661027.10000	

Date(s) Drilled	**9/25/13**	Logged By	**D. Rector**	Checked By	**D. Schug**
Drilling Method	**Becker**	Drill Bit Size/Type	**6.6-inches, 4-inch tooth open bit**	Total Depth of Borehole	**28.5 feet**
Drill Rig Type	**Drill Systems AP1000 with Becker hammer**	Drilling Contractor	**Great West Drilling**	Approximate Surface Elevation	**1090 feet MSL**
Groundwater Level and Date Measured	**None encountered as of 12/20/13**	Sampling Method(s)	**SPT/Grab**	Hammer Data	**140 lbs/30-inch drop**
Borehole Completion	**Bentonite chips**	Location	**N 33.34882 W -117.01032**		

Elevation, feet	Depth, feet	Samples: Type	Samples: Number	Blows per foot	Drives per foot	Bounce Pressure, psi	Graphic Log	MATERIAL DESCRIPTION	Well Completion Detail	REMARKS
1090	0							FILL Moist, brown to reddish brown, clayey sand with few angular gravels (decomposed granitic soil)		
					10	12				
					39	15				
					59	15				
					63	16				
1085	5				67	16				
		Grab	1		39	16				
					26	14				
					30	15				
					30	15				
1080	10				20	15				
		Grab	2		25	15				
					41	15				
					45	15				
					56	16				
1075	15				37	16				
		Grab	3		42	16				Pinhole Test
					41	16		Becomes grayish brown		
					59	16				
		Grab	4		48	16				
		SPT	5		21	15				
1070	20			27				GRANITIC ROCK Highly to completely weathered, dense to very dense. Breaks down to mottled gray and reddish brown, clayey SAND (SC) with gravel-size rock fragments		
					25	15				
					24	15				
					35	16		Corestone, highly weathered granitic rock		
					250	26				
1065	25	Grab	6		126	25		Becomes light grayish brown to light brown		
					66	17				
		Grab	7		56	17				
		SPT	8	63						MC(3)
								Bottom of boring at 28.5 feet		
1060	30									

Figure A-4

URS

Date(s) Drilled	**9/26/13**	Logged By	**D. Rector**	Checked By	**D. Schug**
Drilling Method	**Becker**	Drill Bit Size/Type	**6.6-inches, 4-inch tooth open bit**	Total Depth of Borehole	**31.5 feet**
Drill Rig Type	**Drill Systems AP1000 with Becker hammer**	Drilling Contractor	**Great West Drilling**	Approximate Surface Elevation	**1090 feet MSL**
Groundwater Level and Date Measured	**None encountered**	Sampling Method(s)	**SPT/2.5" ID/Grab**	Hammer Data	**140 lbs/30-inch drop**
Borehole Completion	**Bentonite chips**	Location	**Top of embankment**		

Elevation feet	Depth, feet	SAMPLES: Type	SAMPLES: Number	SAMPLES: Blows per foot	SAMPLES: Drives per foot	SAMPLES: Bounce Pressure, psi	Graphic Log	MATERIAL DESCRIPTION	REMARKS
1090	0							FILL Moist, brown to reddish brown, clayey sand with few angular gravels (decomposed granitic soil)	
					10	10			
					15	11			
			1		13	11			MC(7), SA(22)
					16	14			
1085	5				20	14			
			2	26	29	14			MC(8)
					24	14			
					18	14		Few gravels	
1080	10				21	14			MC(5), Pinhole Test
			3	38	16	14			
					19	15		Becomes gray brown	
					31	15			
			4		41	16			
					63	16			
1075	15				52	17			
			5	53	80	15			
					63	16			
			6		41	15			
					52	16		GRANITIC ROCK Highly to completely weathered, dense to very dense. Breaks down to mottled gray and reddish brown, clayey SAND (SC) with gravel-size rock fragments	
			7	50/3"	49	15			
1070	20								
					40	15			
					26	15			
			8		26	15		Cobble-sized corestone	
					39	16			
1065	25				60	17			
					140	20			
					65	19			
			10	41	68	16			
			11	45	59	16			
1060	30								

Project: Shadow Run Ranch
Project Location: Pauma Valley, California
Project Number: 27661027.10000

Log of Boring B-3

Elevation feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Drives per foot	Bounce Pressure, psi	Graphic Log	MATERIAL DESCRIPTION	REMARKS
1060	30								
								Bottom of boring at 31,5 feet	
1055	35								
1050	40								
1045	45								
1040	50								
1035	55								
1030	60								
1025	65								

URS

Figure A-5

Project: Shadow Run Ranch
Project Location: Pauma Valley, California
Project Number: 27661027.10000

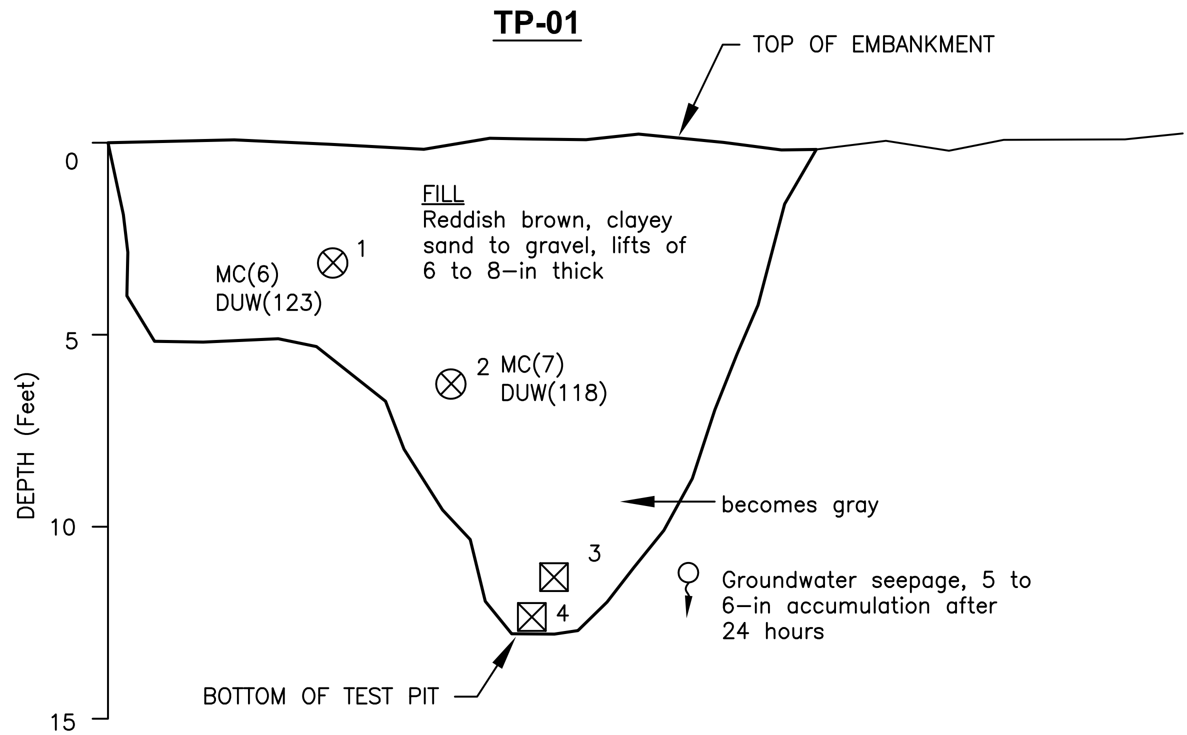
Log of Boring B-4

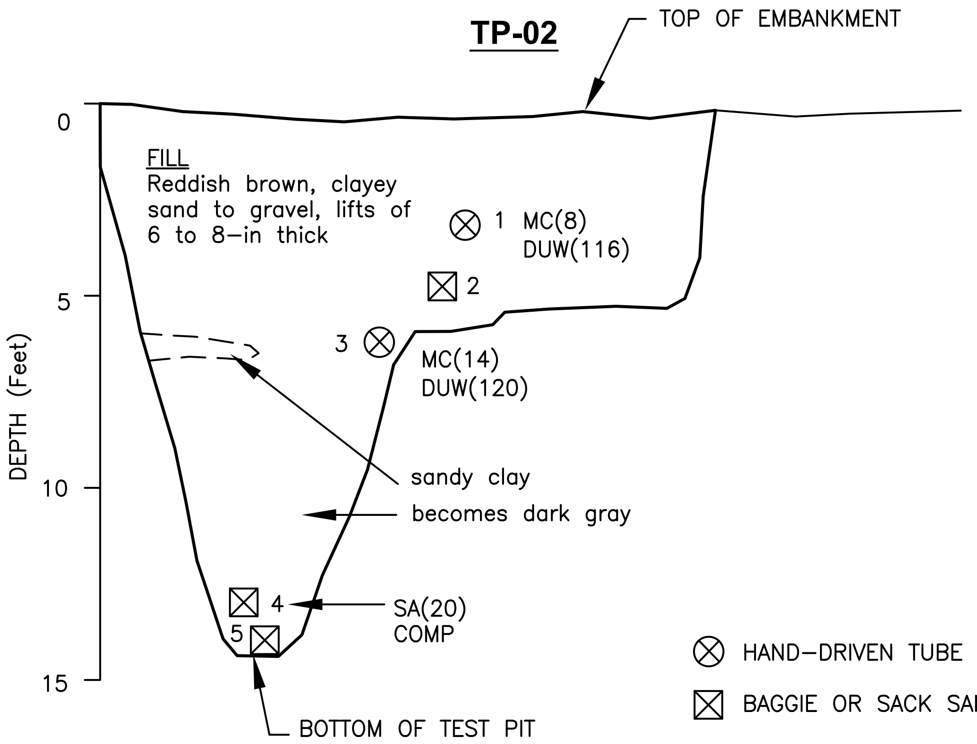
Date(s) Drilled	**9/26/13**	Logged By	**D. Rector**	Checked By	**D. Schug**
Drilling Method	**Becker**	Drill Bit Size/Type	**6.6-inches, 4-inch tooth open bit**	Total Depth of Borehole	**29.4 feet**
Drill Rig Type	**Drill Systems AP1000 with Becker hammer**	Drilling Contractor	**Great West Drilling**	Approximate Surface Elevation	**1090 feet MSL**
Groundwater Level and Date Measured	**15.6 feet bgs (12/20/13)**	Sampling Method(s)	**SPT/2.5" ID/Grab**	Hammer Data	**140 lbs/30-inch drop**
Borehole Completion	**Bentonite chips**	Location	**N 33.3487 W -117.01116**		

Elevation feet	Depth, feet	SAMPLES Type	SAMPLES Number	Blows per foot	Drives per foot	Bounce Pressure, psi	Graphic Log	MATERIAL DESCRIPTION	Well Completion Detail	REMARKS
1090	0							FILL Moist, brown to reddish brown, clayey sand with few angular gravels (decomposed granitic soil)		
					32	14				
					32	14				
			1		40	16				
1085	5				46	16				
			2	38	21	16				MC(6), SA(28)
					46	16				
			3		46	16		Few gravels		
			4	50/4"	36	16				
1080	10									SA(7), DUW(119)
					38	16		Becomes gray brown		
					40	16				
			5		34	16				
					27	16		Becomes brown to reddish brown		
1075	15				25	16				
			6	43	51	16				MC(9)
					45	16				
			7		48	16				
					51	16		GRANITIC ROCK Highly to completely weathered, dense to very dense. Breaks down to mottled gray and reddish brown, clayey SAND (SC) with gravel-size rock fragments		
			8	51	19	16				
1070	20									MC(9), DUW(120)
					18	16				
					20	16				
					26	16				
					39	17				
1065	25				47	17				
			9	48	57	16				MC(7)
					62	16				
			10		57	17				
					63	18				
			11	50/6"				Bottom of boring at 29.4 feet		
1060	30									

Figure A-6

URS





⊗ HAND-DRIVEN TUBE SAMPLE

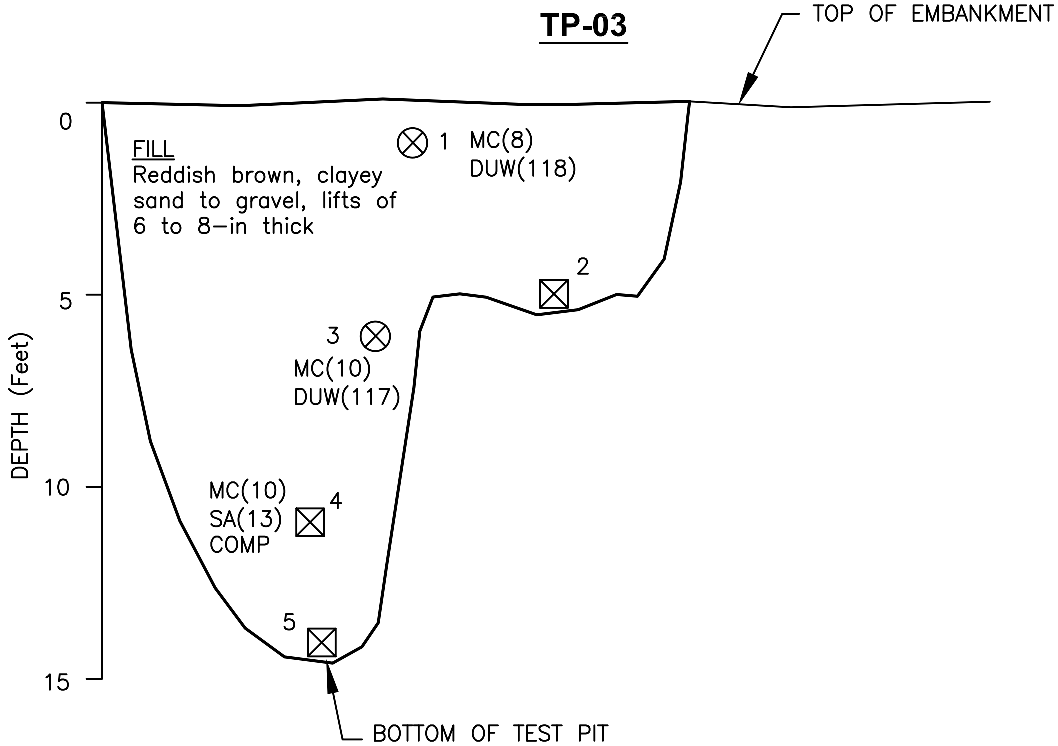
⊠ BAGGIE OR SACK SAMPLE

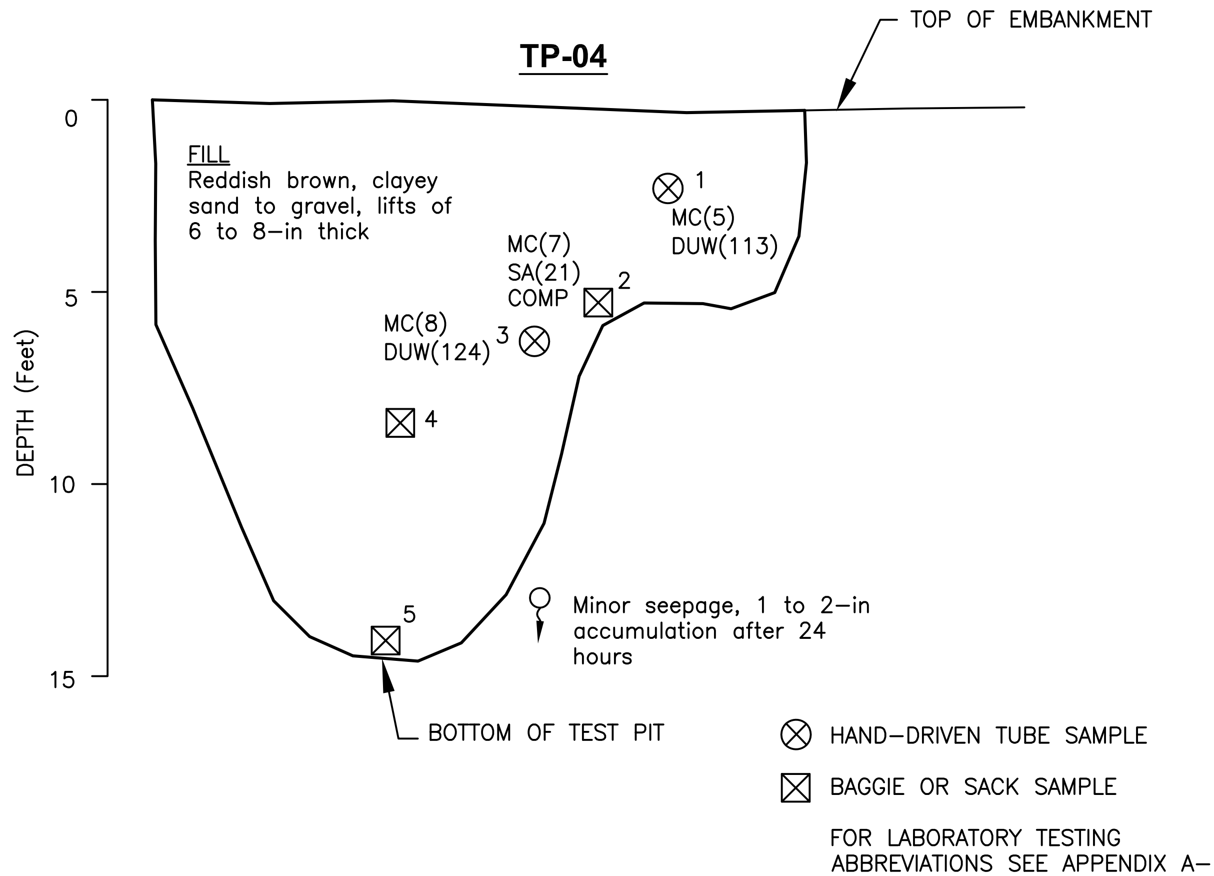
FOR LABORATORY TESTING ABBREVIATIONS SEE APPENDIX A-1

LOG OF TEST PITS TP-01 AND TP-02
SHADOW RUN RANCH
PAUMA VALLEY, CALIFORNIA

URS	2.5 0 2.5 5 feet SCALE: 1"= 5' (H&V)	CHECKED BY: DLS	DATE: 11-25-13	FIG. NO:
		PM: DLS	PROJ. NO: 27661027.10000	A-7

X:\27661027\Shadown Run Test Pits.dwg Nov 25, 2013 - 7:54am





LOG OF TEST PITS TP-03 AND TP-04
SHADOW RUN RANCH
PAUMA VALLEY, CALIFORNIA

URS	2.5 0 2.5 5 feet SCALE: 1"= 5' (H&V)	CHECKED BY: DLS	DATE: 11-25-13	FIG. NO:
		PM: DLS	PROJ. NO: 27661027.10000	**A-8**

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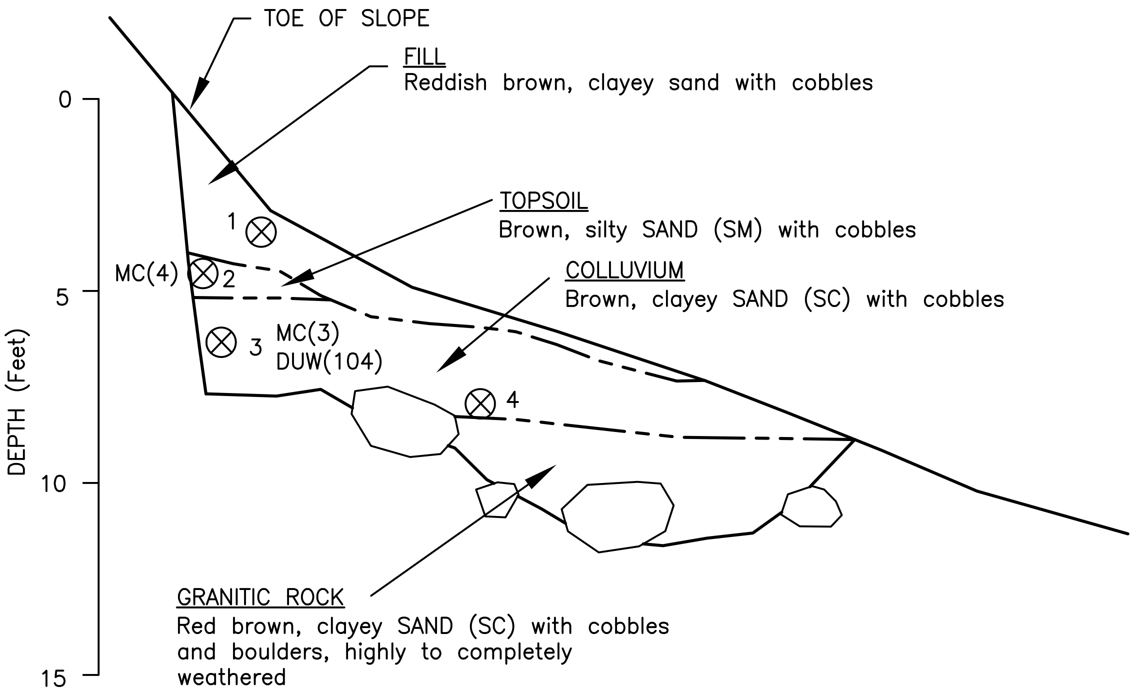
TP-05



⊗ HAND-DRIVEN TUBE SAMPLE

FOR LABORATORY TESTING ABBREVIATIONS SEE APPENDIX A-1

LOG OF TEST PIT TP-05
SHADOW RUN RANCHY
PAUMA VALLEY, CALIFORNIA

URS	2.5 0 2.5 5 feet SCALE: 1"= 5' (H&V)	CHECKED BY: DLS	DATE: 11-25-13	FIG. NO:
		PM: DLS	PROJ. NO: 27661027.10000	**A-9**

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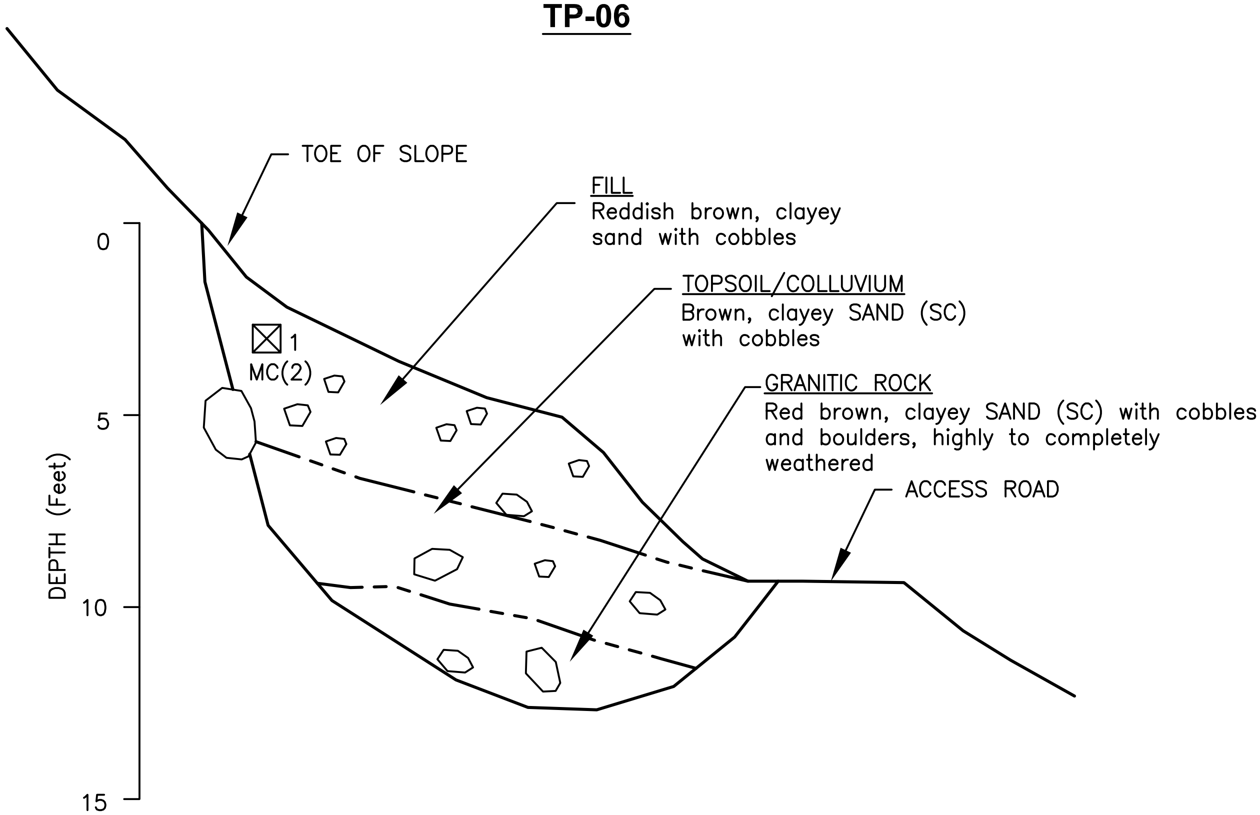
TP-06



⊗ HAND–DRIVEN TUBE SAMPLE

⊠ BAGGIE OR SACK SAMPLE

FOR LABORATORY TESTING ABBREVIATIONS SEE APPENDIX A–1

LOG OF TEST PIT TP-06
SHADOW RUN RANCH
PAUMA VALLEY, CALIFORNIA

URS	2.5 0 2.5 5 feet SCALE: 1"= 5' (H&V)	CHECKED BY: DLS	DATE: 11–25–13	FIG. NO:
		PM: DLS	PROJ. NO: 27661027.10000	A-10

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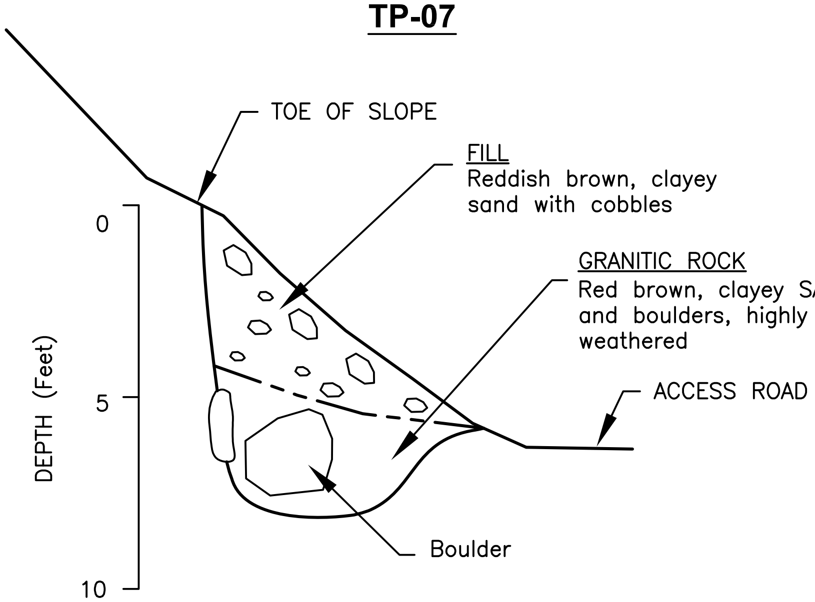
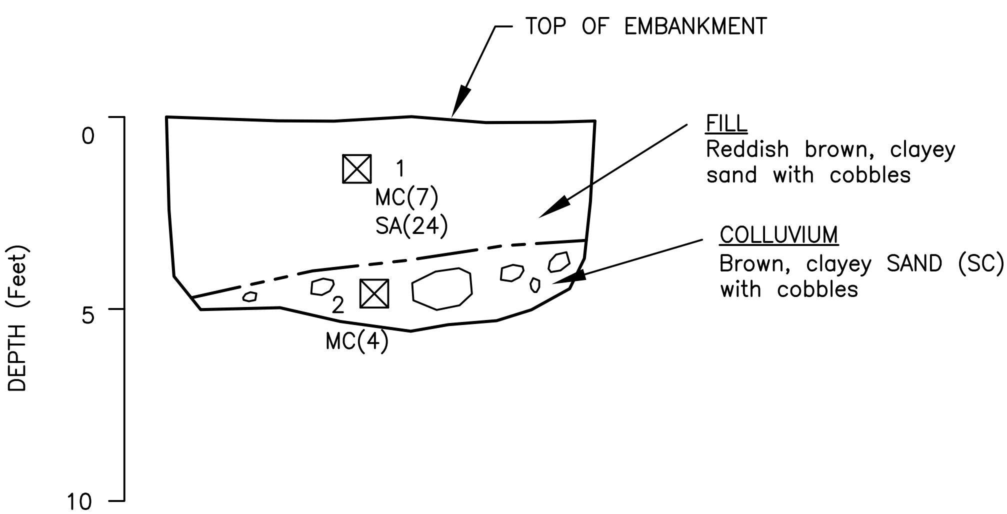
TP-07



TP-08



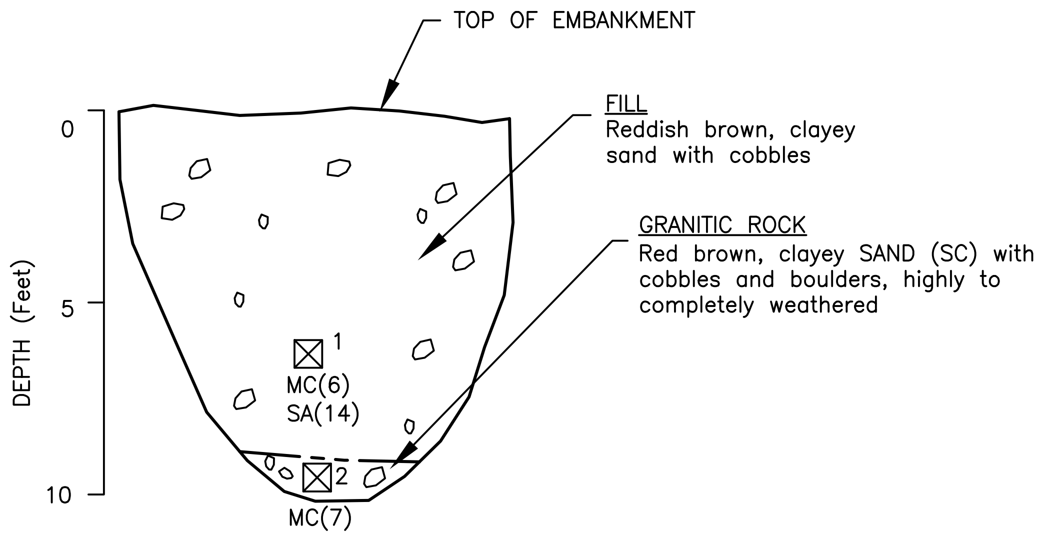
☒ BAGGIE OR SACK SAMPLE

FOR LABORATORY TESTING ABBREVIATIONS SEE APPENDIX A-1

LOG OF TEST PITS TP-07 AND TP-08
SHADOW RUN RANCH
PAUMA VALLEY, CALIFORNIA

URS	2.5 0 2.5 5 feet SCALE: 1"= 5' (H&V)	CHECKED BY: DLS	DATE: 11-25-13	FIG. NO:
		PM: DLS	PROJ. NO: 27661027.10000	**A-11**

TP-09



⊠ BAGGIE OR SACK SAMPLE

FOR LABORATORY TESTING ABBREVIATIONS SEE APPENDIX A–1

LOG OF TEST PIT TP-09
SHADOW RUN RANCH
PAUMA VALLEY, CALIFORNIA

URS	2.5 0 2.5 5 feet SCALE: 1"= 5' (H&V)	CHECKED BY: DLS	DATE: 11–25–13	FIG. NO:
		PM: DLS	PROJ. NO: 27661027.10000	**A-12**

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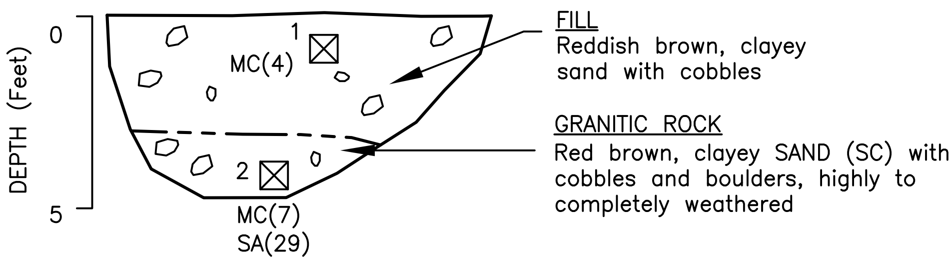
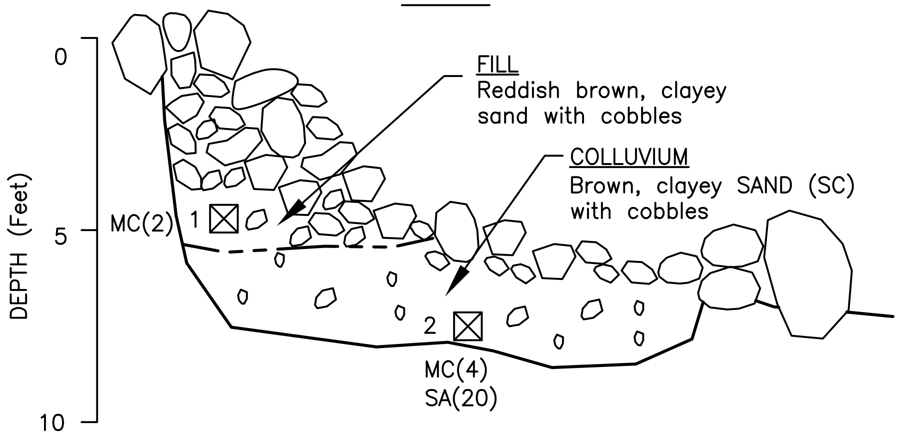
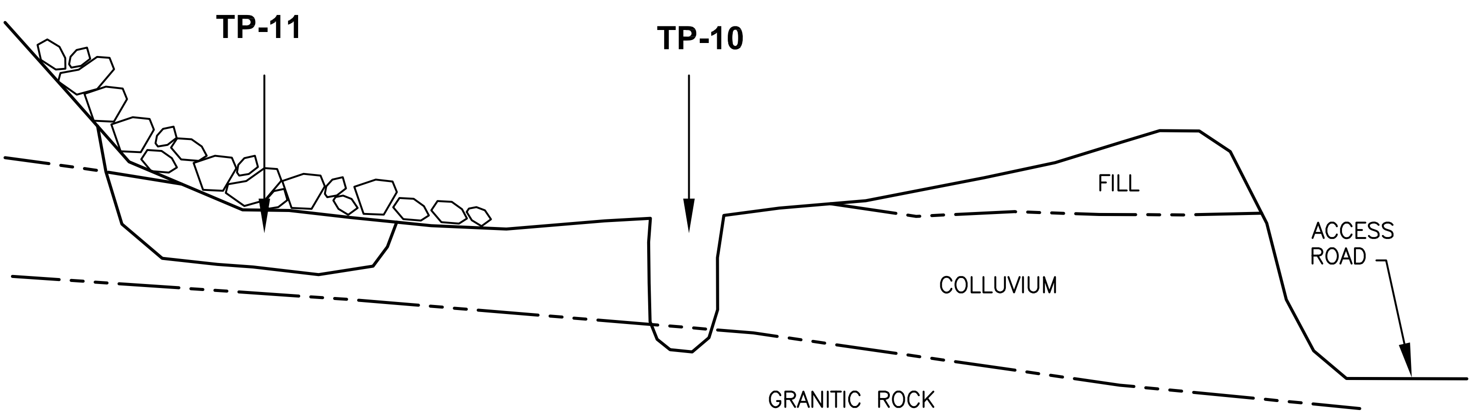
TP-10



TP-11





CROSS SECTION (VEW SOUTH) OF TP-10 AND TP-11

☒ BAGGIE OR SACK SAMPLE

FOR LABORATORY TESTING ABBREVIATIONS SEE APPENDIX A–1

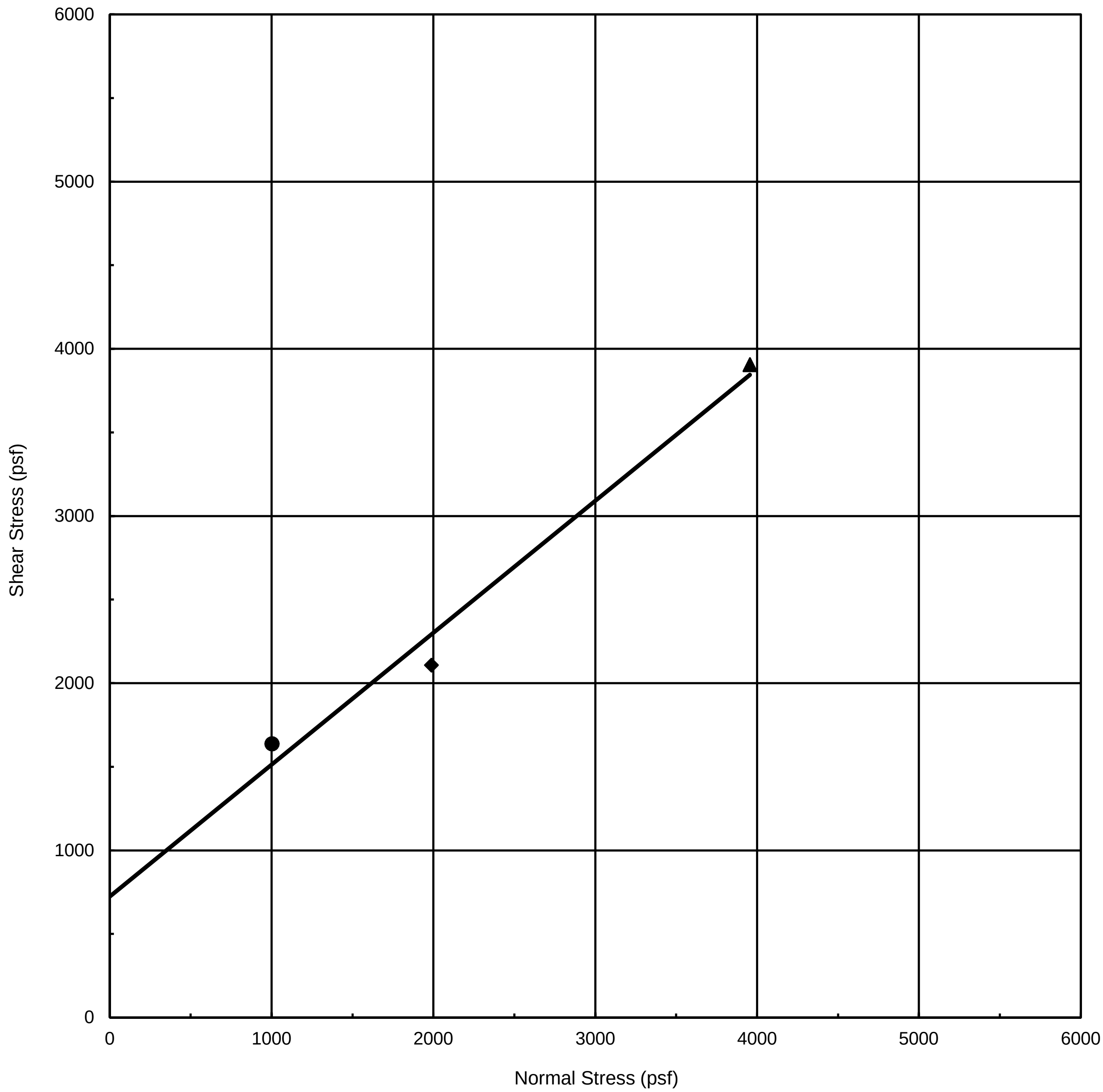
LOG OF TEST PITS TP-10 AND TP-11
SHADOW RUN RANCH
PAUMA VALLEY, CALIFORNIA

URS	2.5 0 2.5 5 feet SCALE: 1"= 5' (H&V)	CHECKED BY: DLS	DATE: 11–25–13	FIG. NO:
		PM: DLS	PROJ. NO: 27661027.10000	A-13

The materials observed in the borings were visually classified and evaluated with respect to grain size, dry density and moisture content. These classifications were substantiated by performing laboratory tests on representative samples of the soils.

The laboratory tests were performed in general accordance with test methods of the ASTM International (ASTM) and included the following:

- Moisture content and dry density (ASTM D2216 and D2937); the test results are presented on the boring logs at the corresponding sample depth;
- The percentage of fines passing a No. 200 sieve (ASTM D1140; denoted by WA) are presented on the boring logs at the corresponding sample depth;
- Direct shear tests (ASTM D3080); the normal stress versus shear stress curves are presented on Figures B-1 to B-4, and the resulting friction angle and cohesion are also given on the figure.
- Compaction tests (ASTM D698 and D1557); the compaction curves and particle size distribution curves are presented on Figures B-5 through B-7;
- Atterberg limit tests (ASTM D4318) were performed on representative samples of the fill soil. Test results are shown on Figures B-8 through B-11. Test results are also shown on the boring logs at the corresponding sample depth;
- Pinhole tests (ASTM 4647) were performed to evaluate the dispersive nature of the fill soils. Test results are presented on Figures B- 12 through B-14. Test results are also shown on the boring logs at the corresponding sample depth.



Peak Values are : ● ,solid trend line   Ultimate Values are: ❍ ,dashed trend line

Boring No.:	B02	Strength Intercept (C) :	724.3	psf	Peak	XXXXX	psf	Ultimate
Sample No.:	3		34.7	kPa		XXXXX	kPa	
Depth (ft | m)	9.0 | 2.7	Friction Angle (φ) :	38	degree		XXXXX	degree	
Description:	**Very dark grayish brown clayey Sand (SC)**		Shear rate :	0.0030 (in/min) ,	0.0076 (cm/min)			

SYMBOL	% Water Content	Total Unit Weight (pcf)	Total Unit Weight (kN/m³)	Dry Unit Weight (pcf)	Dry Unit Weight (kN/m³)	Normal Stress (psf)	Normal Stress (kPa)	Peak Stress (psf)	Peak Stress (kPa)	Ultimate Stress (psf)	Ultimate Stress (kPa)
Initial / Set up	11.1	135.1	21.2	121.6	19.1	XXXXX	XXXXX	XXXXX	XXXXX	XXXXX	XXXXX
pre-shear ● spec. 1	15.7	141.6	22.3	122.4	19.2	1003	48	1638	78	XXXXX	XXXXX
pre-shear ◆ spec. 2	18.6	137.2	21.6	115.7	18.2	1988	95	2108	101	XXXXX	XXXXX
pre-shear ▲ spec. 3	14.0	144.5	22.7	126.8	19.9	3956	189	3906	187	XXXXX	XXXXX

URS

Proposed Shadow Run Ranch

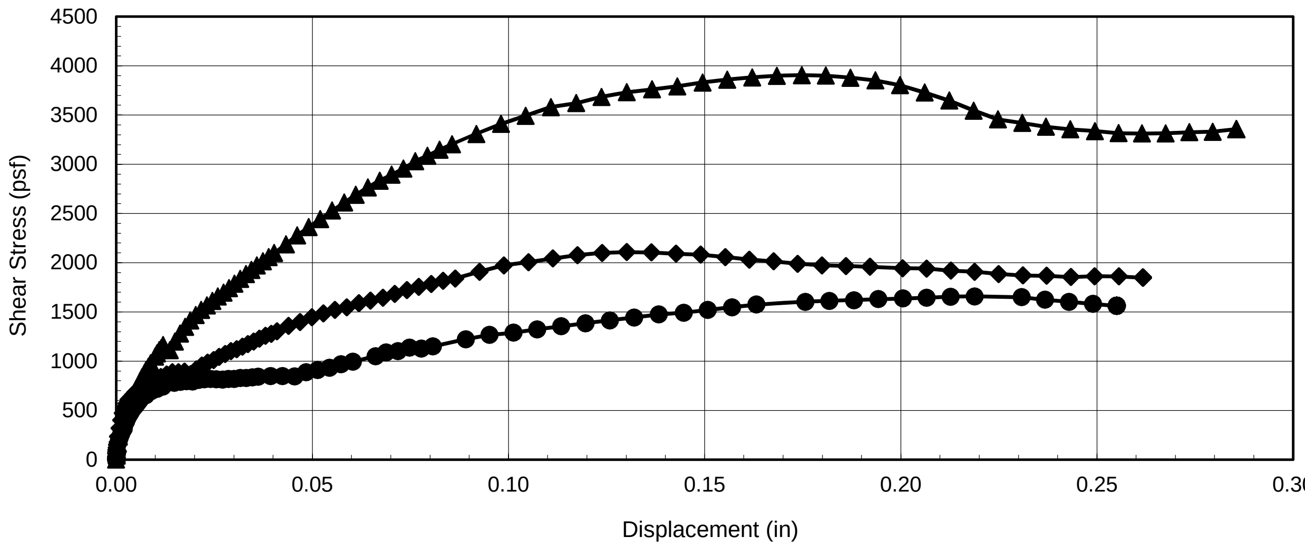
Project Number: 27661027
Test Date: 10/8/2013

DIRECT SHEAR TEST

Figure B-1

DIRECT SHEAR TEST
ASTM D 3080

Project Name :	Shadow Run Ranch	Boring No.:	B02
Project Number :	27661027	Sample No.:	3
		Sample Depth (ft.):	9
Specimen Description :	Very dark grayish brown clayey Sand (SC)		
		Normal Stress (psf):	1003 ●
Apparatus No.:	DS-2	Normal Stress (psf):	1988 ◆
Shear rate (in/min):	0.003	Normal Stress (psf):	3956 ▲



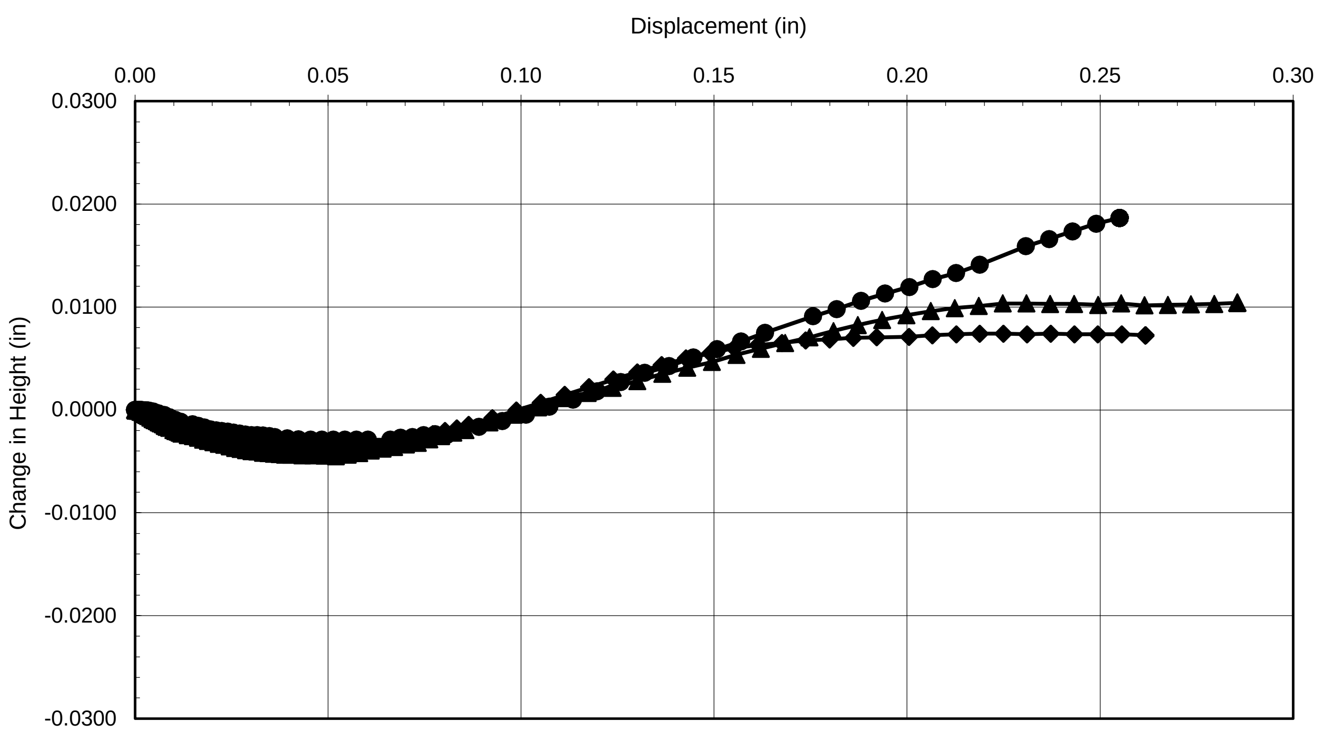
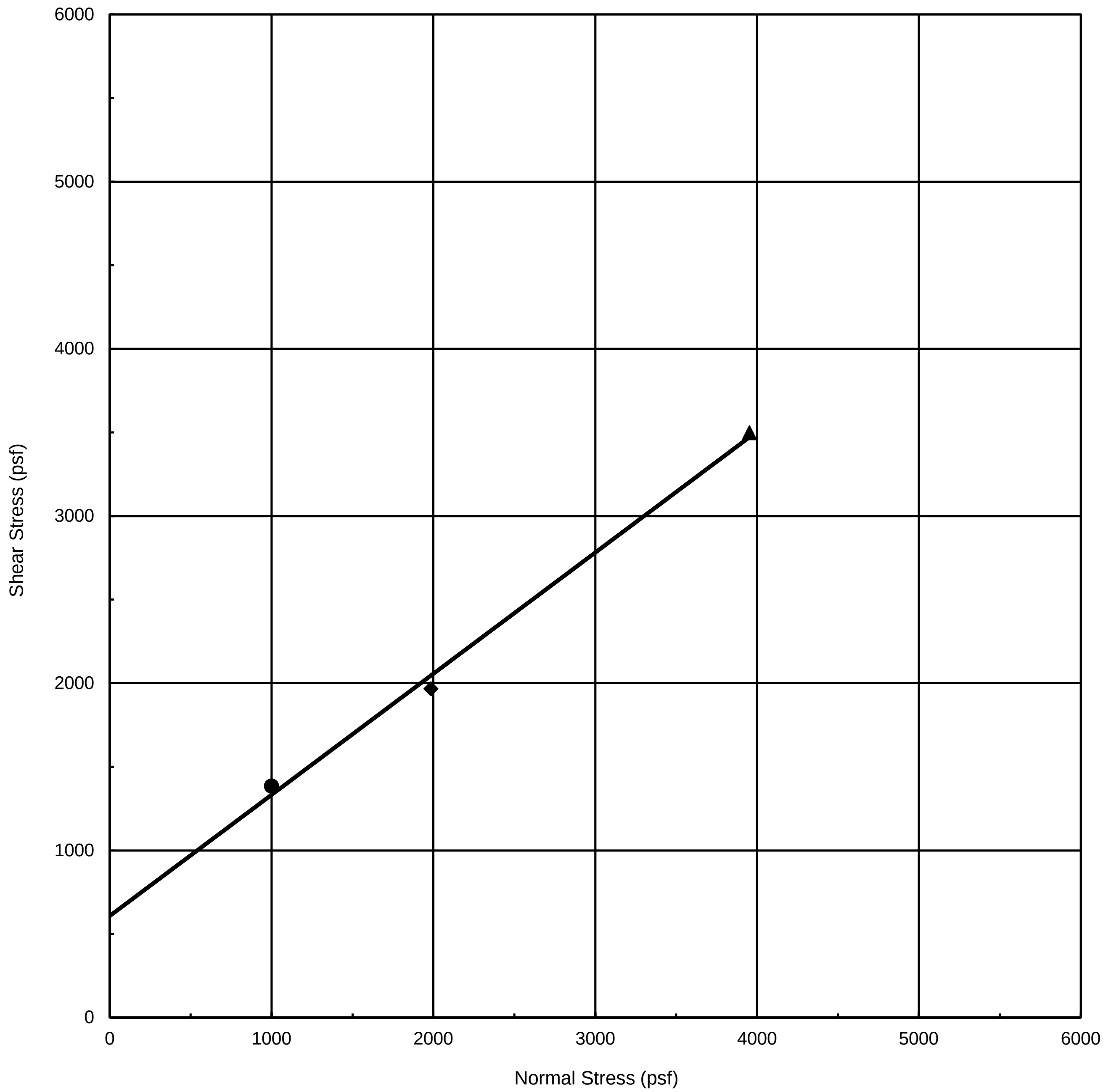


Figure B-2



Peak Values are : ● ,solid trend line   Ultimate Values are: ❍ ,dashed trend line

Boring No.:	B02		Strength Intercept (C) :		607.8	psf	Peak	XXXXX	psf	Ultimate	
Sample No.:	7				29.1	kPa		XXXXX	kPa		
Depth (ft | m)	19.0	5.8	Friction Angle (ϕ) :		36	degree		XXXXX	degree		
Description:	**Very dark gray clayey Sand (SC)**				Shear rate :		0.0030	(in/min) ,	0.0076	(cm/min)	
SYMBOL	% Water Content	Total Unit Weight (pcf)	Total Unit Weight (kN/m^3)	Dry Unit Weight (pcf)	Dry Unit Weight (kN/m^3)	Normal Stress (psf)	Normal Stress (kPa)	Peak Stress (psf)	Peak Stress (kPa)	Ultimate Stress (psf)	Ultimate Stress (kPa)
Initial / Set up	8.9	119.9	18.8	110.1	17.3	XXXXX	XXXXX	XXXXX	XXXXX	XXXXX	XXXXX
pre-shear ● spec. 1	15.5	136.5	21.4	118.1	18.6	999	48	1385	66	XXXXX	XXXXX
pre-shear ◆ spec. 2	15.8	136.8	21.5	118.1	18.6	1985	95	1966	94	XXXXX	XXXXX
pre-shear ▲ spec. 3	15.8	133.9	21.0	115.6	18.2	3952	189	3498	167	XXXXX	XXXXX

URS

Proposed Shadow Run Ranch

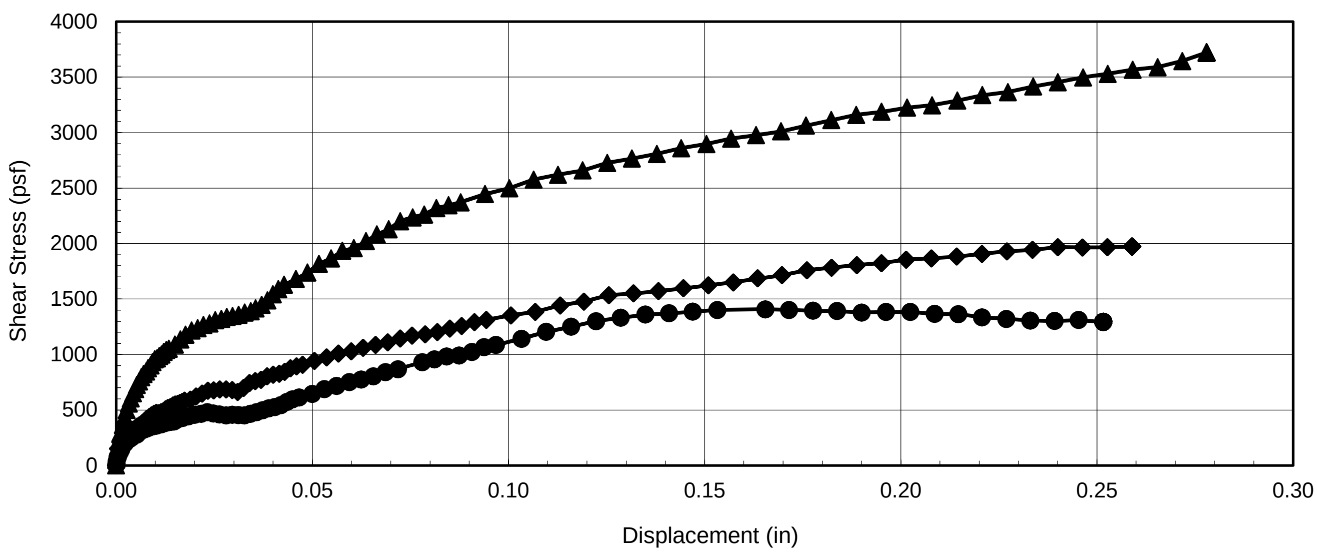
Project Number: 27661027
Test Date: 10/8/2013

DIRECT SHEAR TEST

Figure B-3

DIRECT SHEAR TEST
ASTM D 3080

Project Name :	Shadow Run Ranch	Boring No.:	B02
Project Number :	27661027	Sample No.:	7
		Sample Depth (ft.):	19
Specimen Description :	Very dark gray clayey Sand (SC)		
		Normal Stress (psf):	999 ●
Apparatus No.:	DS-2	Normal Stress (psf):	1985 ◆
Shear rate (in/min):	0.003	Normal Stress (psf):	3952 ▲



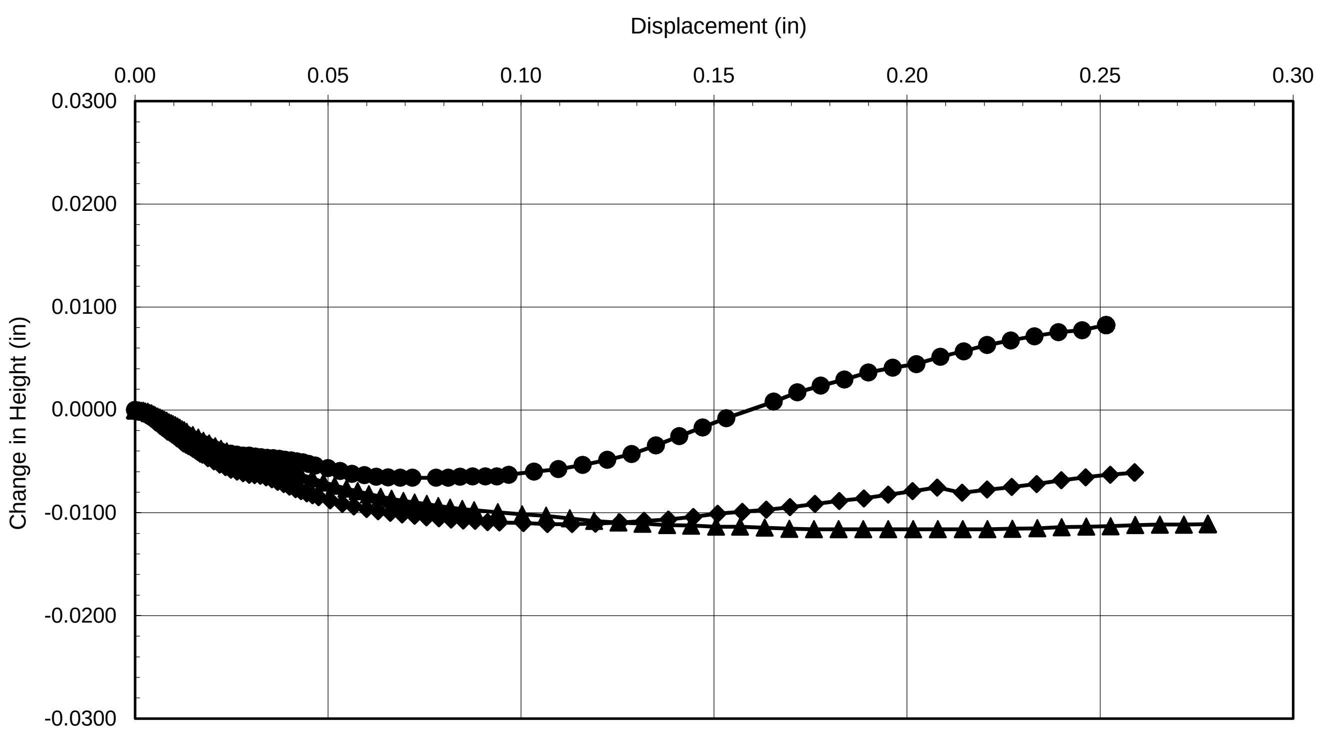
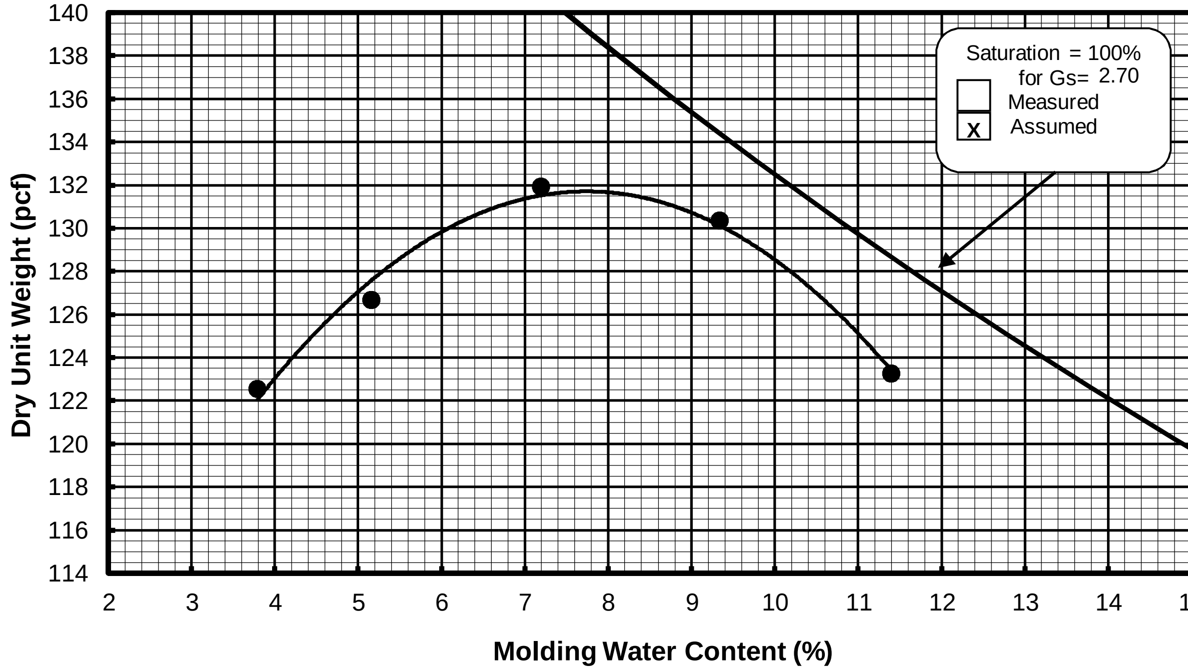


Figure B-4

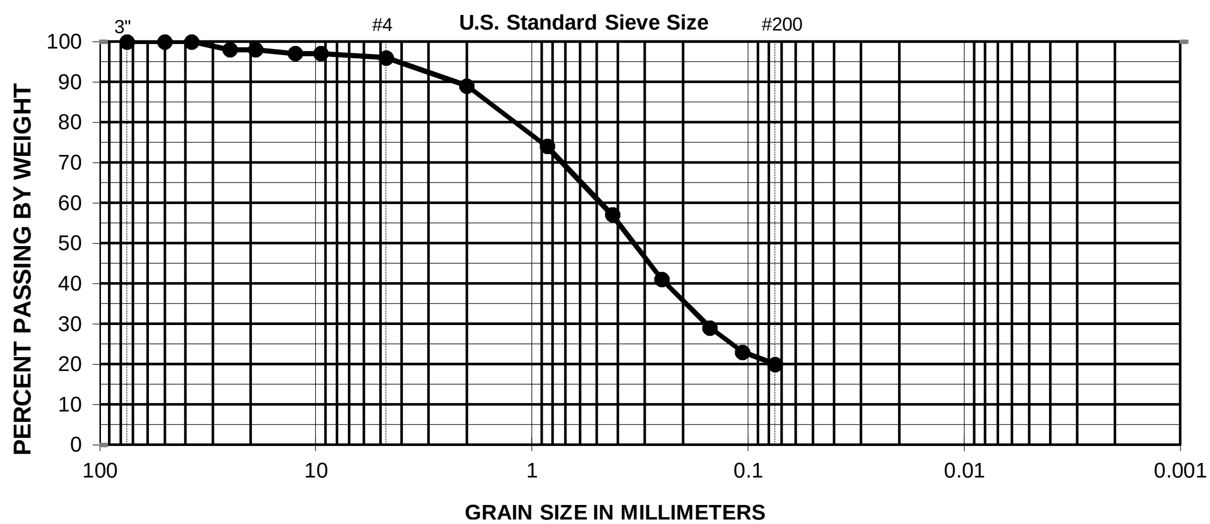
COMPACTION CURVE

Test Method: ● ASTM D 1557 ■ ASTM D 698 ◆ CA-DWR: S-10 ❍ Other Effort

Compaction Procedure: **B** Specimen Preparation Method: **Moist**



PARTICLE-SIZE DISTRIBUTION CURVE



NOTATION: ● Representative of entire sample ◆ Representative of compacted specimen ■ Representative of compacted specimen and entire sample

Boring Number	Sample Number	Depth (ft.)	Optimum WC (%)	Maximum DUW (pcf)	Description and/or Classification
					Dark grayish brown silty Sand (SM)
TP-2	4	13.0	8.0	131.5	

PROJECT NAME: **Proposed Shadow Run Ranch**
PROJECT NUMBER: **27661027**

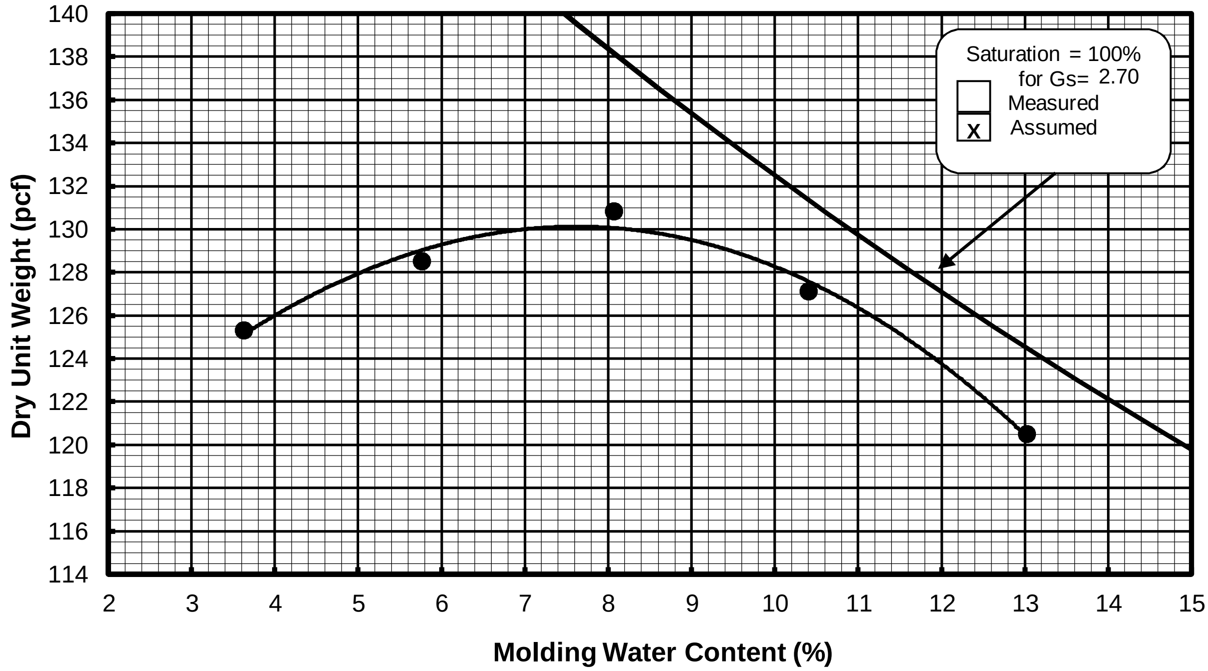
Figure B-5

SUBMITTED BY: Thomas J. O'Meara

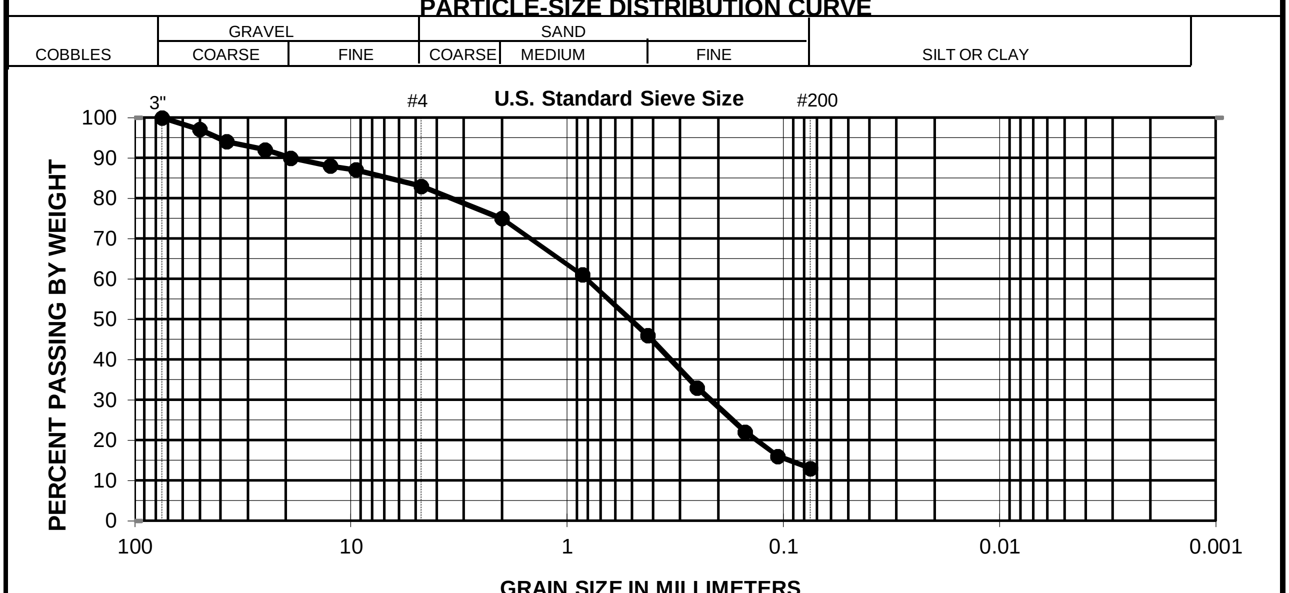
COMPACTION CURVE

Test Method: ● ASTM D 1557 ■ ASTM D 698 ◆ CA-DWR: S-10 ❍ Other Effort

Compaction Procedure: **B** Specimen Preparation Method: **Moist**



PARTICLE-SIZE DISTRIBUTION CURVE



NOTATION: ● Representative of entire sample ◆ Representative of compacted specimen ■ Representative of compacted specimen and entire sample

Boring Number	Sample Number	Depth (ft.)	Optimum WC (%)	Maximum DUW (pcf)	Description and/or Classification
TP-3	4	11.0	8.0	130.0	Grayish brown silty Sand with gravel (SM)

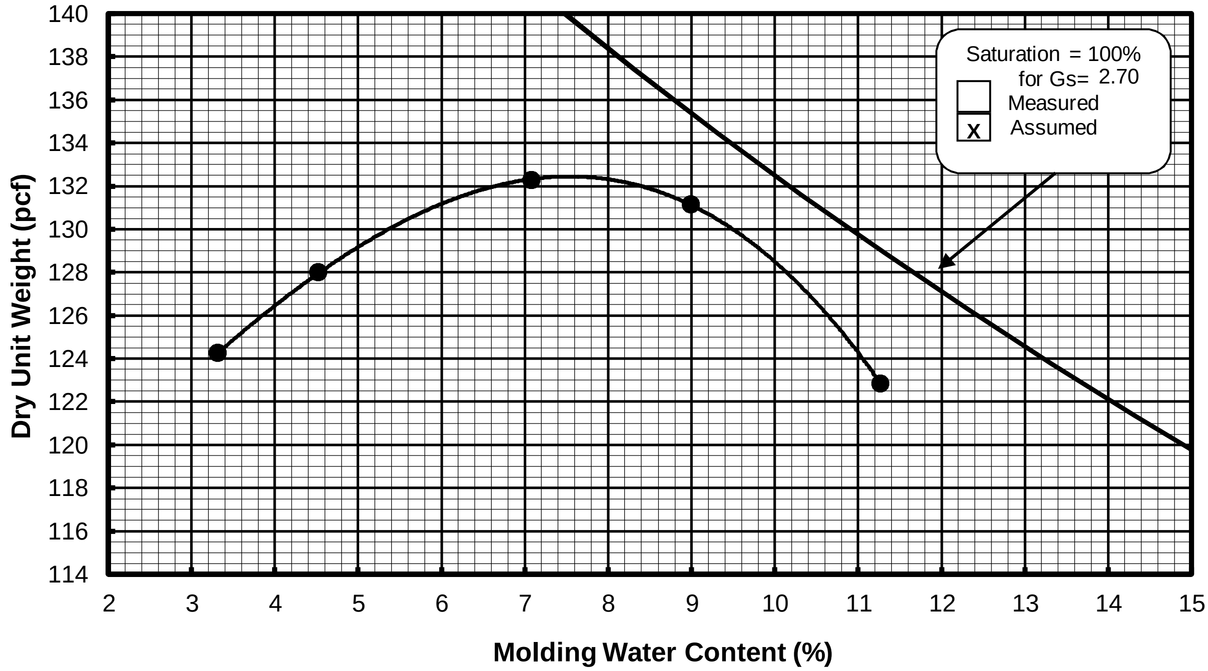
PROJECT NAME: Proposed Shadow Run Ranch **PROJECT NUMBER: 27661027**	**Figure B-6**

SUBMITTED BY: Thomas J. O'Mear

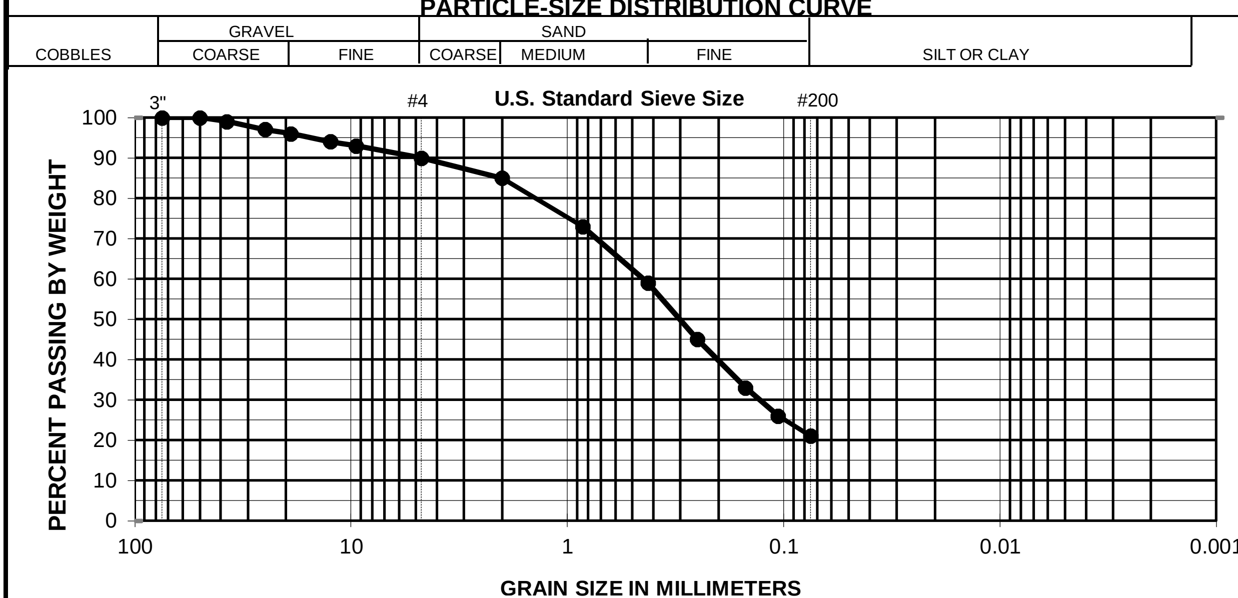
COMPACTION CURVE

Test Method: ● ASTM D 1557 ■ ASTM D 698 ◆ CA-DWR: S-10 ❍ Other Effort

Compaction Procedure: **B** Specimen Preparation Method: **Moist**



PARTICLE-SIZE DISTRIBUTION CURVE



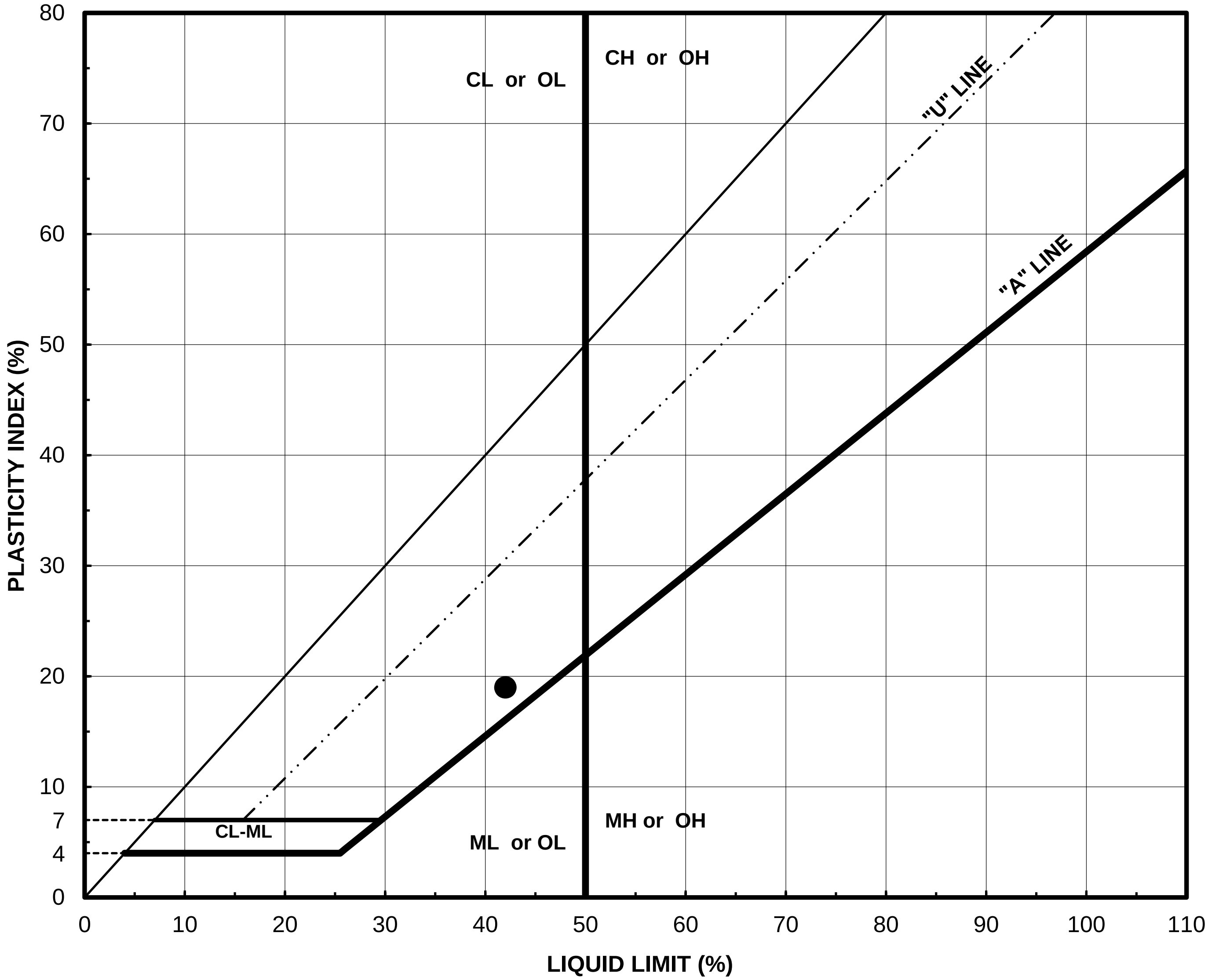
NOTATION: ● Representative of entire sample ◆ Representative of compacted specimen ■ Representative of compacted specimen and entire sample

Boring Number	Sample Number	Depth (ft.)	Optimum WC (%)	Maximum DUW (pcf)	Description and/or Classification
TP-4	2	5.0	7.5	132.5	Brown silty Sand (SM)

PROJECT NAME: Proposed Shadow Run Ranch
PROJECT NUMBER: 27661027

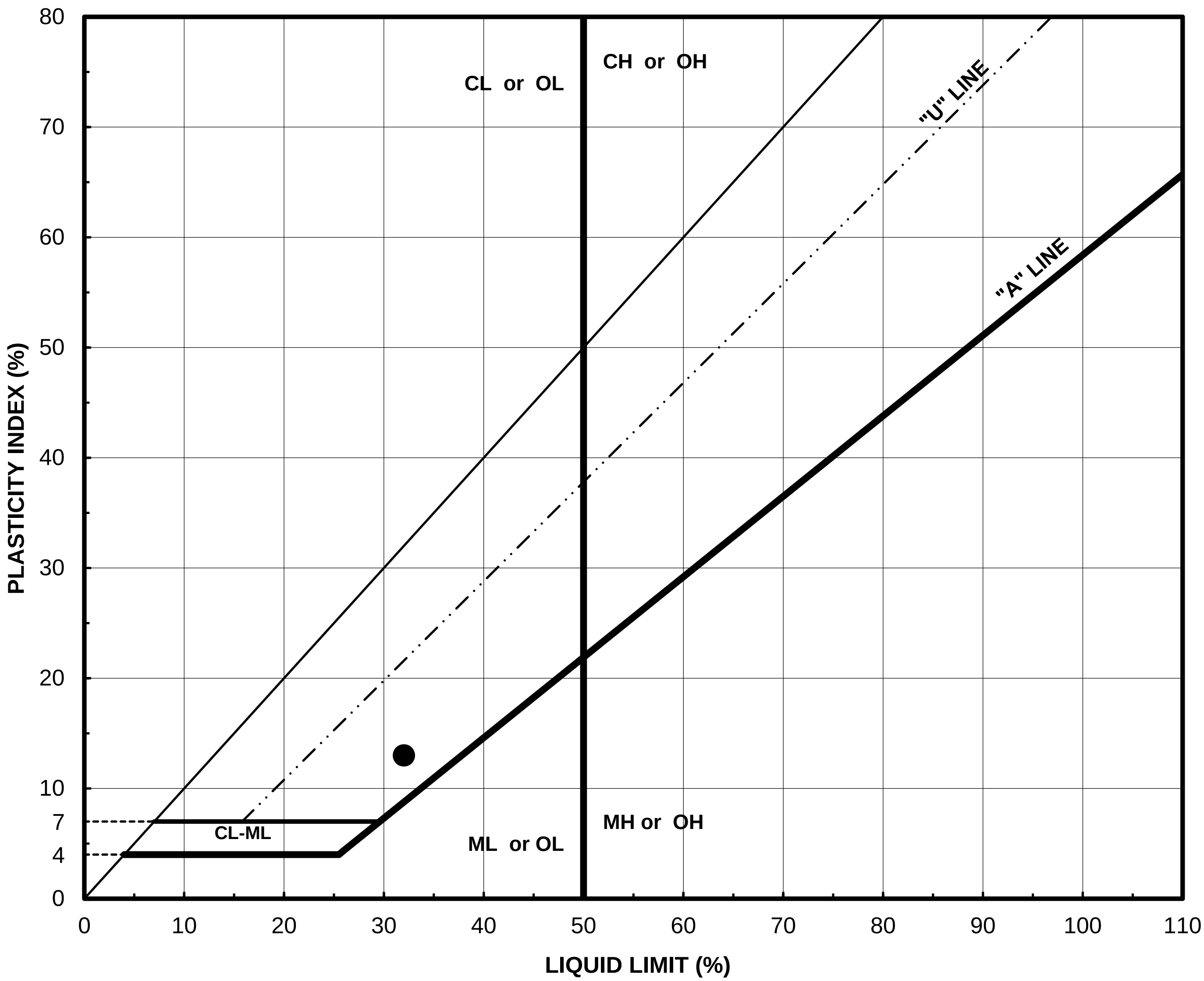
Figure B-7

SUBMITTED BY: Thomas J. O'Meara



Boring Number	Sample Number	Depth (ft)	Water Content (%)	LL	PI	DESCRIPTION / CLASSIFICATION
B-01	6	15.0	NA	42	19	Olive gray clayey Sand (SC)

Project Name: Shadow Run Ranch	**PLASTICITY CHART**
Project Number: 27661027	**Figure B-8**



Boring Number	Sample Number	Depth (ft)	Water Content (%)	LL	PI	DESCRIPTION / CLASSIFICATION
B-01	10	25.0	NA	32	13	Dark gray clayey Sand (SC)

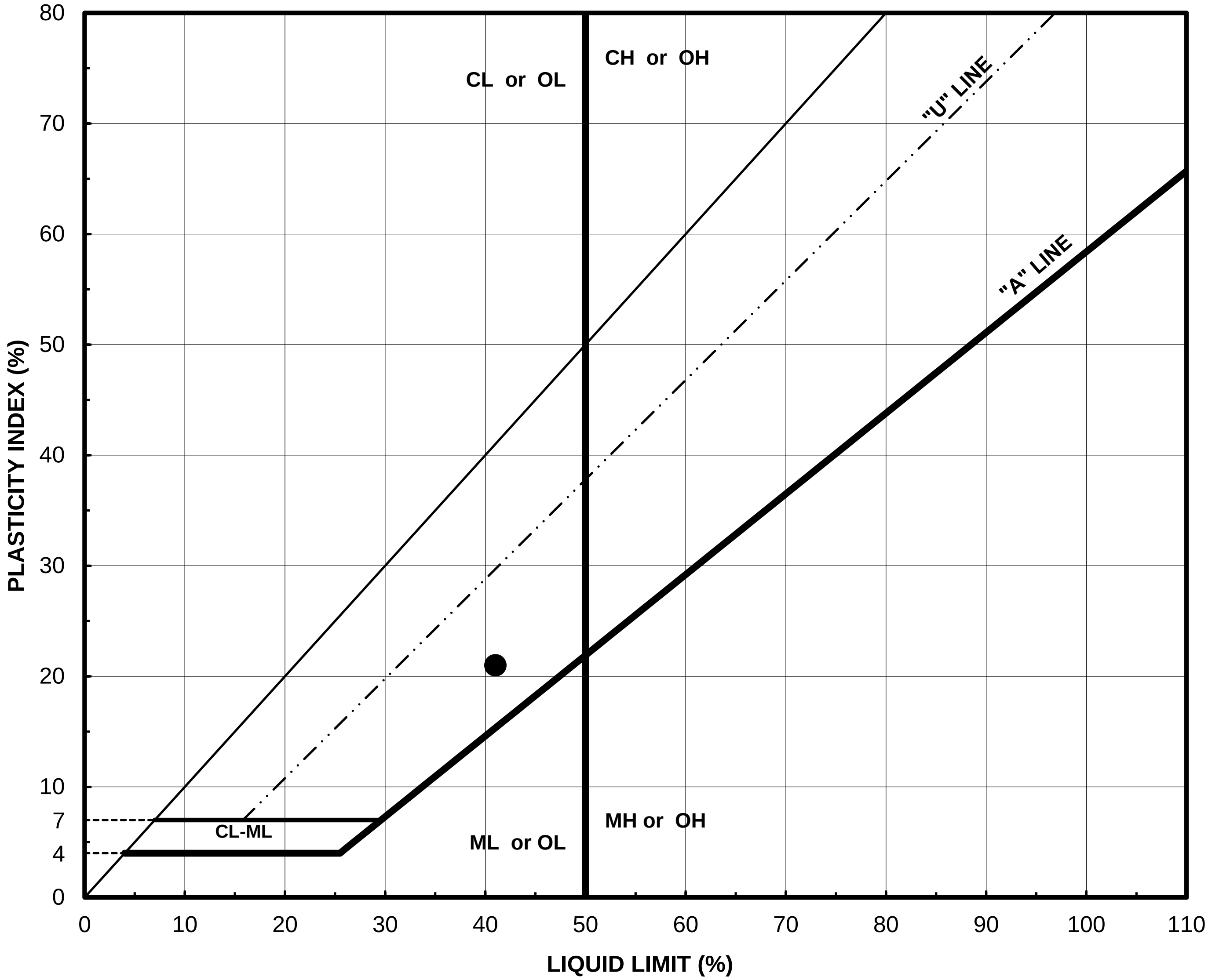
Project Name: Shadow Run Ranch

Project Number: 27661027

PLASTICITY CHART

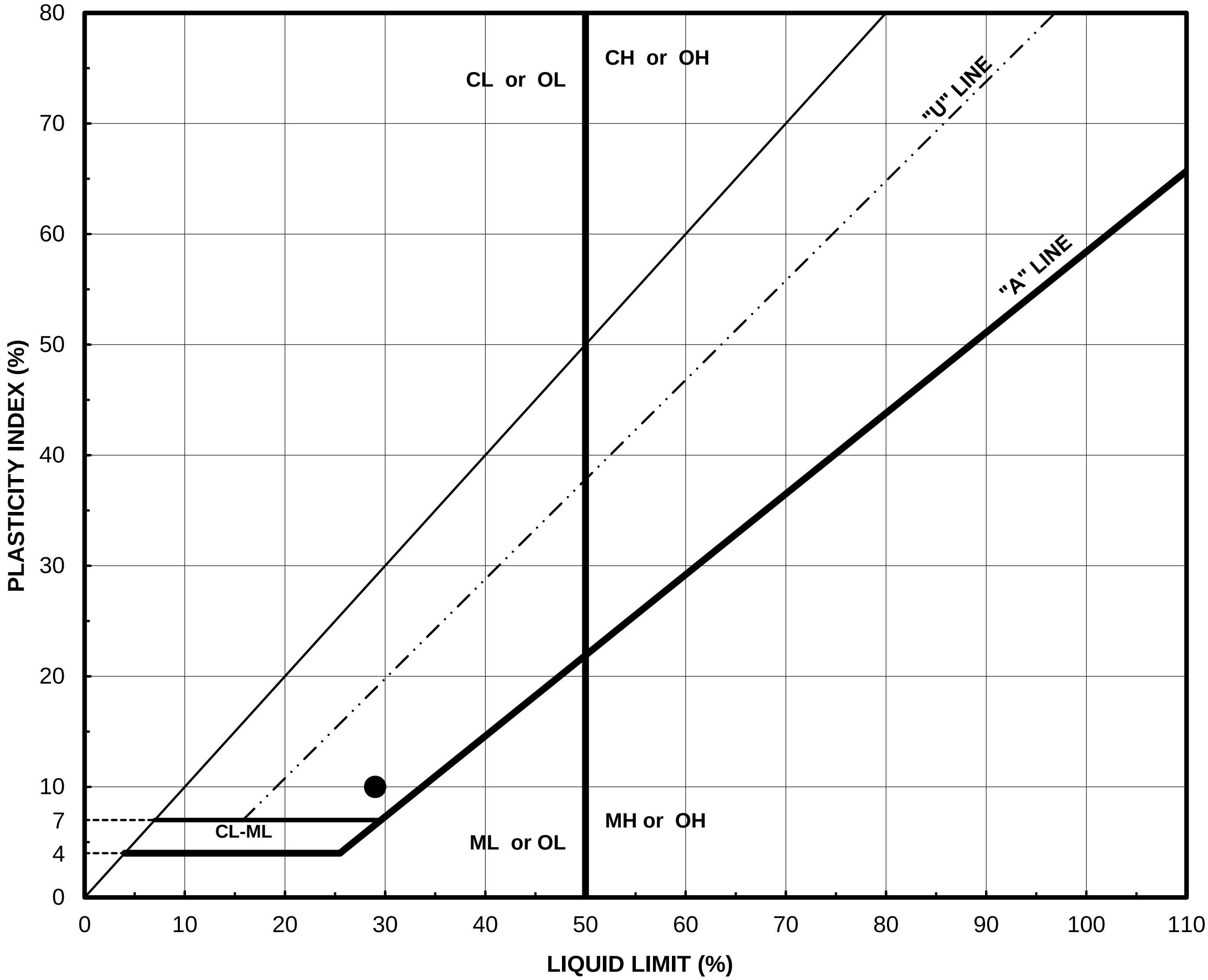
Figure B-9

URS



Boring Number	Sample Number	Depth (ft)	Water Content (%)	LL	PI	DESCRIPTION / CLASSIFICATION
B-02	3	3.0	NA	41	21	Brown clayey Sand (SC)

Project Name:	**Shadow Run Ranch**	**PLASTICITY CHART**
Project Number:	**27661027**	**Figure B-10**



Boring Number	Sample Number	Depth (ft)	Water Content (%)	LL	PI	DESCRIPTION / CLASSIFICATION
B-02	5	15.0	NA	29	10	Dark grayish olive clayey Sand (SC)

Project Name: Shadow Run Ranch	**PLASTICITY CHART**
Project Number: 27661027	**Figure B-11**

SAMPLE PROCESSING: REMOLDING SPECIMENS

Project Number: 27661027 Task Number: 10000 Boring No.: B-01

Project Name: Shadow Run Ranch Sample No.: 12

Project Engineer: DS Test: Pinhole Depth (ft): 29

Specimen Description and/or Classification: Dark olive brown clayey Sand (SC)

WATER CONTENT (OVEN DRIED)	Specimen Condition: INITIAL	REMOLD	AFTER TEST
Container No.		s43	s21
Mass of Container & Wet Specimen, M1 (g)		274.51	196.60
Mass of Container & Dry Specimen, M2 (g)		255.60	186.50
Mass of Container, M3 (g)		81.01	104.86
WATER CONTENT, wn (%)		**10.8**	**12.4**
AVERAGE OR SELECTED VALUE, wn (%)			XXXXXXXX
Circle Appr. Max. Grain Size in "Sample" : 1 1/2"	3/4"	#4	#40

MOLDING TUBE DIMENSIONS

	Length (in)	Dia. (in)
1	1.500	1.366
2		
3		
4	xxxxx	xxxxx
Ave.	1.500	1.366

Sample Compaction Data

Compactive Effort:

Max. Dry Unit Weight, (pcf):

Optimum Water Content (%):

Reconstituted Specimen Compaction Data

Percent Compaction:

Est. Water Content (%):

Molding Wet Wgt, MWW (g):

Number of Compaction Lifts: 1

Wet Weight per Lift (g)

% Under Comp. (1st. lift): 0

UNIT WEIGHT	Molding Tube	Comp. Specimen
Container No.		
Mass of Container and Wet Specimen, M4 (g)		572.35
Mass of Container, M5 (g)		477.92
Mass of Wet Specimen, M6 (g)		94.43
Nominal [ ] Measured [x] Specimen Dia., D (in) or ( )	1.500	1.500
Nominal [ ] Measured [x] Spec. Length, L (in) or ( )	1.500	1.500
Specimen Area, A (in^2) or ( )	1.767	1.767
Specimen Volume, V (cm^3) or ( )	43.44	43.44
Assumed [x] Measured [ ] Specific Gravity, Gs	2.70	2.70
WET UNIT WEIGHT, WUW (pcf) or ( )		135.7
DRY UNIT WEIGHT, DUW (pcf) or ( )		122.4
DEGREE OF SATURATION, S (%)		77.7
Relative Compaction, RC (%)	**XXXXX**	

SPECIMEN DIMENSIONS

	Length (in)	Dia. (in)
1	1.5	1.5
2		
3		
4		
5		
6	XXXX	
Ave.	1.500	1.500

REMARKS:

w or wn = ((M1 - M2) / (M2 - M3)) * 100

WUW = (M6,g) / (V,cm^3) * (62.428, lb/ft^3) MWW = (Max. DUW * Percent Compaction) * (1 + (Molding WC / 100)) * Molding Tube Volume / 62.428

DUW = WUW / (1 + (w / 100))

S = (DUW * w * Gs) / ((Gs * Unit Weight or Density of Water) - DUW)

Unit Weight or Density of Water (20 ^{o}C) = 62.316 pcf or 0.99821 g/cm^3

RC % = (Test DUW / Max. DUW) * 100

COMPACTED BY: LV DATE: 12/13/13

CALCULATED BY: TJO DATE: 12/14/13

CHECKED BY: TJO DATE: 12/14/13

REVIEWED and SUBMITTED BY: Thomas J O'Meara

Figure B-12

PINHOLE TEST DATA

ASTM D 4647

Project Name: Shadow Run Ranch

Boring No.: B-01

Project Number 27661027

Task Number: 10000

Sample No.: 12

Project Engineer: DS

Depth (ft): 29.0

Tested by: LV

Date: 12/13/2013

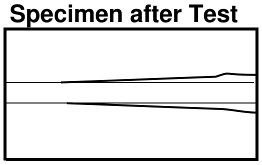
Checked by: TJO

Date: 12/14/2013

Visual Description: Dark olive brown clayey Sand (SC)

Specimen condition: [ ] Intact or [x] Remolded

Specimen after Test



Final Diameter (Df), mm ~ Di * 1.5

Total Unit Weight: 135.7 pcf

Water Content: 10.8 %

Fines Content ____ %

Pinhole Classification: D1

by Method: C

C = Clear
MD = Moderately Dark
BV = Barely Visible
D = Dark
SD = Slightly Dark
VD = Very Dark

Head (inch)	Time (hh:mm)	Elapsed Time (sec)	Incr Time (sec)	Cumm Flow (ml)	Incr Flow (ml)	Flow Rate (ml/s)	Turbidty from side of cylinder	Remarks
2	11:42	0	0	0	0	0.00		
		27	27	25	25	0.93	D	
		48	21	50	50	2.38	D	
		74	26	75	75	2.88	D	
		100	26	100	100	3.85	VD	
		123	23	125	25	1.09	VD	
		148	25	150	25	1.00	VD	
		197	49	200	50	1.02	VD	
		222	25	225	25	1.00	VD	
		246	24	250	50	2.08	VD	
		300	54	305	55	1.02	VD	Flow rate > 1.0 ml/s, dark, D1 by Method C

Figure B-12 continued

SAMPLE PROCESSING: REMOLDING SPECIMENS

Project Number: 27661027 | Task Number: 10000 | Boring No.: B-02

Project Name: Shadow Run Ranch | Sample No.: 3

Project Engineer: DS | Test: Pinhole | Depth (ft): 9

Specimen Description and/or Classification: Brown silty Sand (SM)

WATER CONTENT (OVEN DRIED)	Specimen Condition INITIAL	REMOLD	AFTER TEST
Container No.		s8	s47
Mass of Container & Wet Specimen, M1 (g)		395.77	170.34
Mass of Container & Dry Specimen, M2 (g)		368.99	161.62
Mass of Container, M3 (g)		115.21	82.61
WATER CONTENT, wn (%)		**10.6**	**11.0**
AVERAGE OR SELECTED VALUE, wn (%)			XXXXXXXX
Circle Appr. Max. Grain Size in "Sample" : 1 1/2"	3/4"	#4	#40

MOLDING TUBE DIMENSIONS

	Length (in)	Dia. (in)
1	1.500	1.500
2		
3		
4	xxxxx	xxxxx
Ave.	1.500	1.500

Sample Compaction Data | **Reconstituted Specimen Compaction Data**

Compactive Effort:

Max. Dry Unit Weight, (pcf): | Percent Compaction:

Optimum Water Content (%): | Est. Water Content (%):

Molding Wet Wgt, MWW (g):

Number of Compaction Lifts: 1

Wet Weight per Lift (g)

% Under Comp. (1st. lift): 0

UNIT WEIGHT	Molding Tube	Comp. Specimen
Container No.		
Mass of Container and Wet Specimen, M4 (g)		568.52
Mass of Container, M5 (g)		478.20
Mass of Wet Specimen, M6 (g)		90.32
Nominal [ ] Measured [x] Specimen Dia., D (in) or ( )	1.500	1.500
Nominal [ ] Measured [x] Spec. Length, L (in) or ( )	1.500	1.500
Specimen Area, A (in^2) or ( )	1.767	1.767
Specimen Volume, V (cm^3) or ( )	43.44	43.44
Assumed [x] Measured [ ] Specific Gravity, Gs	2.70	2.70
WET UNIT WEIGHT, WUW (pcf) or ( )		129.8
DRY UNIT WEIGHT, DUW (pcf) or ( )		117.4
DEGREE OF SATURATION, S (%)		65.4
Relative Compaction, RC (%)	**XXXXX**	

SPECIMEN DIMENSIONS

	Length (in)	Dia. (in)
1	1.5	1.5
2		
3		
4		
5		
6	XXXX	
Ave.	1.500	1.500

REMARKS:

w or wn = ((M1 - M2) / (M2 - M3)) * 100

WUW = (M6,g) / (V,cm^3) * (62.428, lb/ft^3) MWW = (Max. DUW * Percent Compaction) * (1 + (Molding WC / 100)) * Molding Tube Volume / 62.428

DUW = WUW / (1 + (w / 100))

S = (DUW * w * Gs) / ((Gs * Unit Weight or Density of Water) - DUW)

Unit Weight or Density of Water (20 ^oC) = 62.316 pcf or 0.99821 g/cm^3

RC % = (Test DUW / Max. DUW) * 100

COMPACTED BY: LV DATE: 12/13/13

CALCULATED BY: TJO DATE: 12/14/13

CHECKED BY: TJO DATE: 12/14/13

REVIEWED and SUBMITTED BY: Thomas J. O'Mear [signature]

Figure B-13

PINHOLE TEST DATA

ASTM D 4647

Project Name: Shadow Run Ranch — Boring No.: B-02

Project Number 27661027 — Task Number: 10000 — Sample No.: 3

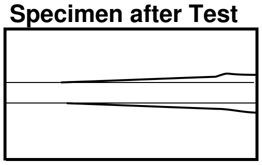
Project Engineer: DS — Depth (ft): 9.0

Tested by: LV — Date: 12/13/2013 — Checked by: TJO — Date: 12/14/2013

Visual Description: Brown silty Sand (SM)

Specimen condition: [ ] Intact or [x] Remolded

Specimen after Test



Final Diameter (Df), mm ~ Di * 1.5

Total Unit Weight: 129.8 pcf

Water Content: 10.6 %

Fines Content ____ %

Pinhole Classification: **D2**

by Method: C

C = Clear — MD = Moderately Dark

BV = Barely Visible — D = Dark

SD = Slightly Dark — VD = Very Dark

Head (inch)	Time (hh:mm)	Elapsed Time (sec)	Incr Time (sec)	Cumm Flow (ml)	Incr Flow (ml)	Flow Rate (ml/s)	Turbidty from side of cylinder	Remarks
2	11:50	0	0	0	0	0.00		
		28	28	25	25	0.89	D	
		68	40	50	50	1.25	D	
		109	41	75	75	1.83	D	
		144	35	100	100	2.86	D	
		177	33	125	25	0.76	D	
		211	34	150	25	0.74	D	
		246	35	175	25	0.71	D	
		279	33	200	25	0.76	D	
		300	21	220	20	0.95	D	Flow rate < 1.0 ml/s, dark, continue with 2 inch head
2	12:05	0	0	0	0	0.00		
		20	20	25	25	1.25	MD	
		42	42	50	50	1.19	MD	
		65	23	75	25	1.09	MD	
		90	25	100	25	1.00	D	
		114	24	125	25	1.04	D	
		138	24	150	25	1.04	D	
		164	26	175	25	0.96	D	
		187	23	200	25	1.09	D	
		211	24	225	25	1.04	D	
		234	23	250	25	1.09	D	

PINHOLE TEST DATA

ASTM D 4647

Project Name: Shadow Run Ranch

Boring No.: B-02

Project Number 27661027

Task Number: 10000

Sample No.: 3

Project Engineer: DS

Depth (ft): 9.0

Tested by: LV

Date: 12/13/2013

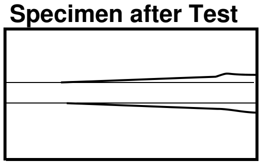
Checked by: TJO

Date: 12/14/2013

Visual Description: Brown silty Sand (SM)

Specimen condition: [ ] Intact or [x] Remolded

Specimen after Test



Final Diameter (Df), mm ~ Di * 1.5

Total Unit Weight: 129.8 pcf

Water Content: 10.6 %

Fines Content ____ %

Pinhole Classification: **D2**

by Method: C

C = Clear

MD = Moderately Dark

BV = Barely Visible

D = Dark

SD = Slightly Dark

VD = Very Dark

Head (inch)	Time (hh:mm)	Elapsed Time (sec)	Incr Time (sec)	Cumm Flow (ml)	Incr Flow (ml)	Flow Rate (ml/s)	Turbidty from side of cylinder	Remarks
		261	27	275	25	0.93	D	Flow rate > 1.0 ml/s, dark, D2 by Method C
		300	39	315	40	1.03	D	

SAMPLE PROCESSING: REMOLDING SPECIMENS

Project Number: 27661027 — Task Number: 10000 — Boring No.: B-03

Project Name: Shadow Run Ranch — Sample No.: 3

Project Engineer: DS — Test: Pinhole — Depth (ft): 9.00

Specimen Description and/or Classification: Dark brown clayey Sand (SC)

WATER CONTENT (OVEN DRIED)	Specimen Condition: INITIAL	REMOLD	AFTER TEST
Container No.		s25	c7
Mass of Container & Wet Specimen, M1 (g)		336.24	161.49
Mass of Container & Dry Specimen, M2 (g)		314.60	152.33
Mass of Container, M3 (g)		98.99	71.58
WATER CONTENT, wn (%)		**10.0**	**11.3**
AVERAGE OR SELECTED VALUE, wn (%)			XXXXXXXX
Circle Appr. Max. Grain Size in "Sample" : 1 1/2"	3/4"	#4	#40

MOLDING TUBE DIMENSIONS

	Length (in)	Dia. (in)
1	1.500	1.500
2		
3		
4	xxxxx	xxxxx
Ave.	1.500	1.500

Sample Compaction Data

Compactive Effort:

Max. Dry Unit Weight, (pcf):

Optimum Water Content (%):

Reconstituted Specimen Compaction Data

Percent Compaction:

Est. Water Content (%):

Molding Wet Wgt, MWW (g):

Number of Compaction Lifts: 1

Wet Weight per Lift (g)

% Under Comp. (1st. lift): 0

UNIT WEIGHT	Molding Tube	Comp. Specimen
Container No.		
Mass of Container and Wet Specimen, M4 (g)		568.52
Mass of Container, M5 (g)		478.20
Mass of Wet Specimen, M6 (g)		90.32
Nominal [ ] Measured [x] Specimen Dia., D (in) or ( )	1.500	1.500
Nominal [ ] Measured [x] Spec. Length, L (in) or ( )	1.500	1.500
Specimen Area, A (in^2) or ( )	1.767	1.767
Specimen Volume, V (cm^3) or ( )	43.44	43.44
Assumed [x] Measured [ ] Specific Gravity, Gs	2.70	2.70
WET UNIT WEIGHT, WUW (pcf) or ( )		129.8
DRY UNIT WEIGHT, DUW (pcf) or ( )		118.0
DEGREE OF SATURATION, S (%)		63.2
Relative Compaction, RC (%)	**XXXXX**	

SPECIMEN DIMENSIONS

	Length (in)	Dia. (in)
1	1.5	1.5
2		
3		
4		
5		
6	XXXX	
Ave.	1.500	1.500

REMARKS:

w or wn = ((M1 - M2) / (M2 - M3)) * 100

WUW = (M6,g) / (V,cm^3) * (62.428, lb/ft^3)

MWW = (Max. DUW * Percent Compaction) * (1 + (Molding WC / 100)) * Molding Tube Volume / 62.428

DUW = WUW / (1 + (w / 100))

S = (DUW * w * Gs) / ((Gs * Unit Weight or Density of Water) - DUW)

Unit Weight or Density of Water (20 °C) = 62.316 pcf or 0.99821 g/cm^3

RC % = (Test DUW / Max. DUW) * 100

COMPACTED BY: LV — DATE: 12/13/13

CALCULATED BY: TJO — DATE: 12/14/13

CHECKED BY: TJO — DATE: 12/14/13

REVIEWED and SUBMITTED BY: Thomas J. O'Mara (signature)

Figure B-14

PINHOLE TEST DATA

ASTM D 4647

Project Name: Shadow Run Ranch　　Boring No.: B-03

Project Number 27661027　　Task Number: 10000　　Sample No.: 3

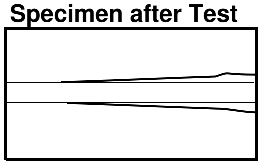
Project Engineer: DS　　Depth (ft): 9.0

Tested by: LV　　Date: 12/13/2013　　Checked by: TJO　　Date: 12/14/2013

Visual Description: Dark brown clayey Sand (SC)

Specimen condition: [ ] Intact or [x] Remolded

Specimen after Test



Final Diameter (Df), mm ~ Di * 1.5

Total Unit Weight: 129.8 pcf

Water Content: 10.0 %

Fines Content ____ %

Pinhole Classification: ND4

by Method: C

C = Clear　　MD = Moderately Dark

BV = Barely Visible　　D = Dark

SD = Slightly Dark　　VD = Very Dark

Head (inch)	Time (hh:mm)	Elapsed Time (sec)	Incr Time (sec)	Cumm Flow (ml)	Incr Flow (ml)	Flow Rate (ml/s)	Turbidty from side of cylinder	Remarks
2	13:40	0	0	0	0	0.00		
		46	46	25	25	0.54	VD	
		100	54	50	25	0.46	VD	
		152	52	75	25	0.48	VD	
		210	58	100	25	0.43	VD	
		266	56	125	25	0.45	VD	
		300	34	145	20	0.59	VD	Flow rate < 1.0 ml/s, dark, continue with 2 inch head
2	13:54	0	0	0	0	0.00		
		23	23	25	25	1.09	VD	
		49	49	50	50	1.02	VD	
		75	52	75	50	0.96	VD	
		126	77	125	75	0.97	VD	
		151	76	150	75	0.99	VD	
		177	51	175	50	0.98	VD	
		199	48	200	50	1.04	VD	
		226	49	225	50	1.02	VD	
		250	51	250	50	0.98	VD	
		275	49	275	50	1.02	VD	Flow rate < 1.0 ml/s, dark, ND4 by Method C
		300	50	295	45	0.90	VD	

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F. 760.741.1786

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MASSON
& ASSOCIATES, INC.
PLANNING ▲ ENGINEERING ▲ SURVEYING

November 26, 2013

Sherrill Schoepe
Shadow Run Ranch, LLC
Post Office Box 1249
Pauma Valley, CA 92061

RE: Other County Requirements (Addendum to Geotechnical Plan)

Dear Ms. Schoepe:

This letter is to address GEI and County Comments “Other County Requirements” in the response letter from GEI dated August 12, 2013 it requests additional information including:

> In addition to the geotechnical assessment of the Reservoir embankment stability, the county also required that the applicant provide the following:
>
> 1. Relocate reservoir spillway
> 2. Reservoir drainage (1000 yr flood calcs)
> 3. Reservoir overtopping (seiche overtopping - short term concentrated flows)
> 4. Failure analyses including path and limits of inundation (in the event of a reservoir embankment failure).
> 5. Detailed maintenance plan to control deterioration due to erosion, vegetation, and small animals.
> 6. Emergency drawdown calculations

Page 6-7, Other Hazards. The geotechnical report states that the project drainage system should be checked for its ability to handle short-term, concentrated flows if significant reservoir overtopping were to occur during an earthquake. Please include an addendum to the geotechnical report which provides a detailed evaluation of the project drainage system and whether it can handle short-term concentrated flows if significant reservoir overtopping were to occur. The addendum should consider the worst case scenario of failure of the existing reservoir embankment in its evaluation. The addendum should include specific design measures as necessary to dissipate and/or divert flows to levels that ensure the safety of all proposed house pads to be placed below the dam. The addendum shall include the following concluding statement and must be signed and stamped by a California Certified Engineering Geologist and if necessary a California Licensed Civil Engineer: “Based on the available information described in this addendum, it is the opinion of the undersigned, that the measures described herein are sufficient to assure the house pads would be safe from the potential effects of dam inundation at the site.” 12/14/2012 2nd Request. This comment was not addressed.

Solved.

Item 1: See revised preliminary grading plan moved spillway (Attachment 1)

Item 2: See additional drainage 1,000 yr. calculations (Attachment 2)

Item 3: Previously addressed in May 31, 2013 letter submitted by Masson & Associates Inc. (Attachment 3)

Item 4: With regard to the worst case scenario of failure, by piping, we reference the URS letter dated August 22, 2013 "Work plan for Geotechnical Assessment" which indicates failure and inundation analysis is not required at this time.

Item 5: See Operation & Maintenance Plan (Attachment 5)

Item 6: See Emergency drawdown calculations (Attachment 6)

Sincerely,

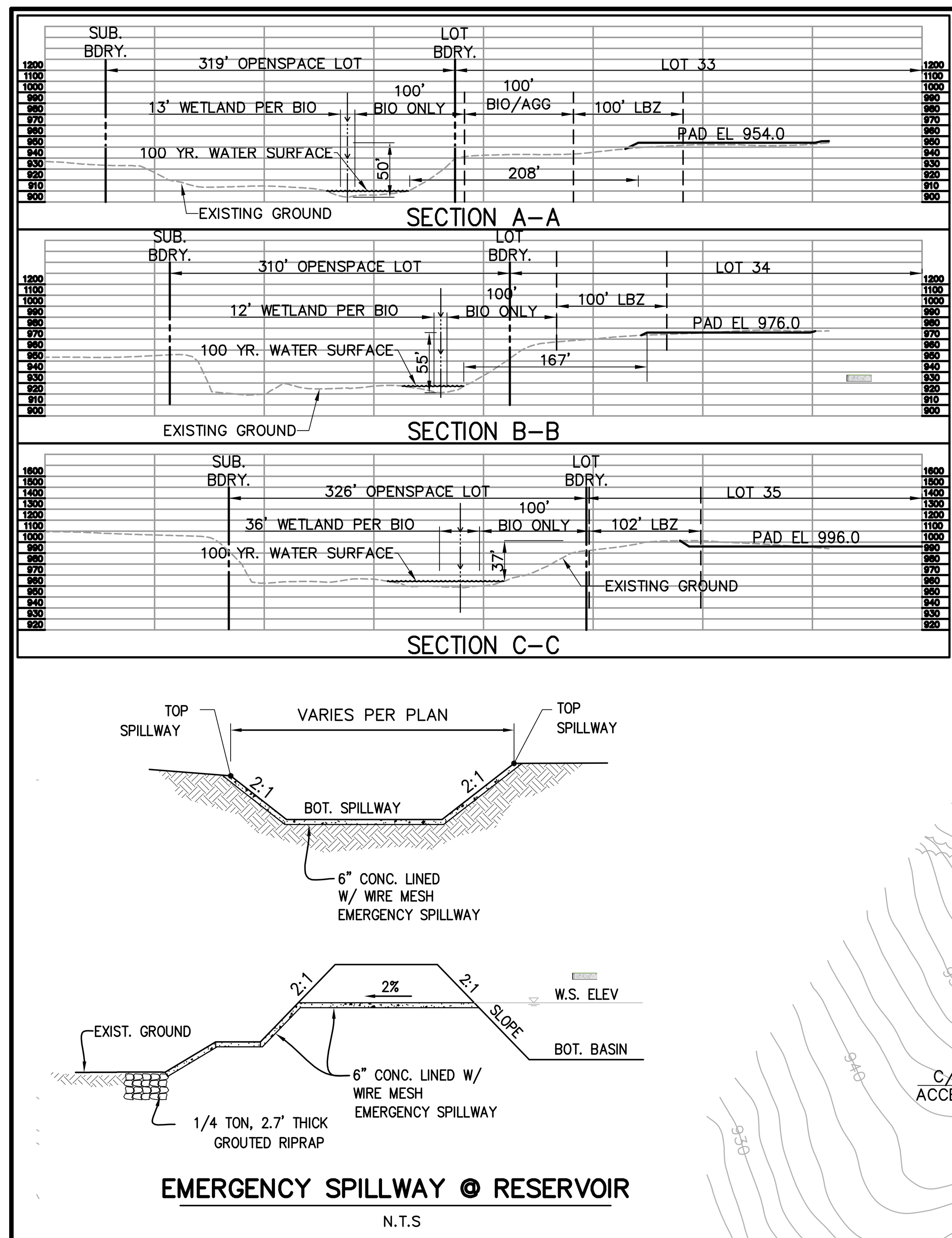
Bruce A. Tait, QSP/QSD
Director of Engineering

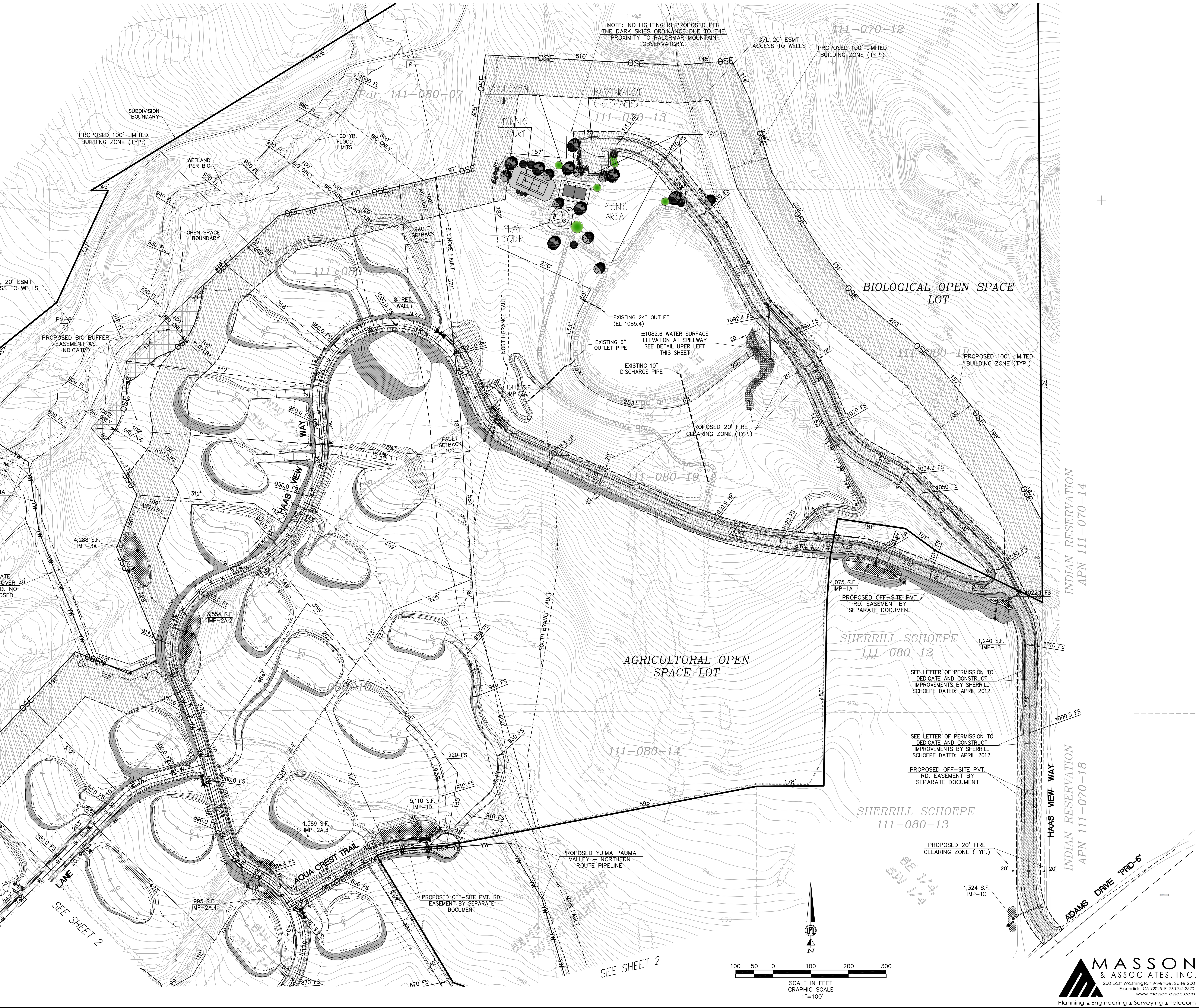
Cc: David Schug
Cc: Robert Hingtgen

COUNTY OF SAN DIEGO TRACT NO. TM 5223 RPL-3

SHADOW RUN RANCH, PAUMA VALLEY

PRELIMINARY GRADING PLAN





1,000 YR. RESERVOIR CALCULATIONS

For
SHADOW RUN RANCH

TM 5223

Preparation/Revision Date:

November 25, 2013

Prepared for:

Sherrill Ann Schoepe, General Partner

Shadow Run Ranch, LLC

P.O. Box 1249

Pauma Valley, CA 92061

Telephone: (760) 742-1893

Prepared by:

Masson & Associates, Inc.

200 East Washington Avenue, Suite 200

Escondido, CA 92025

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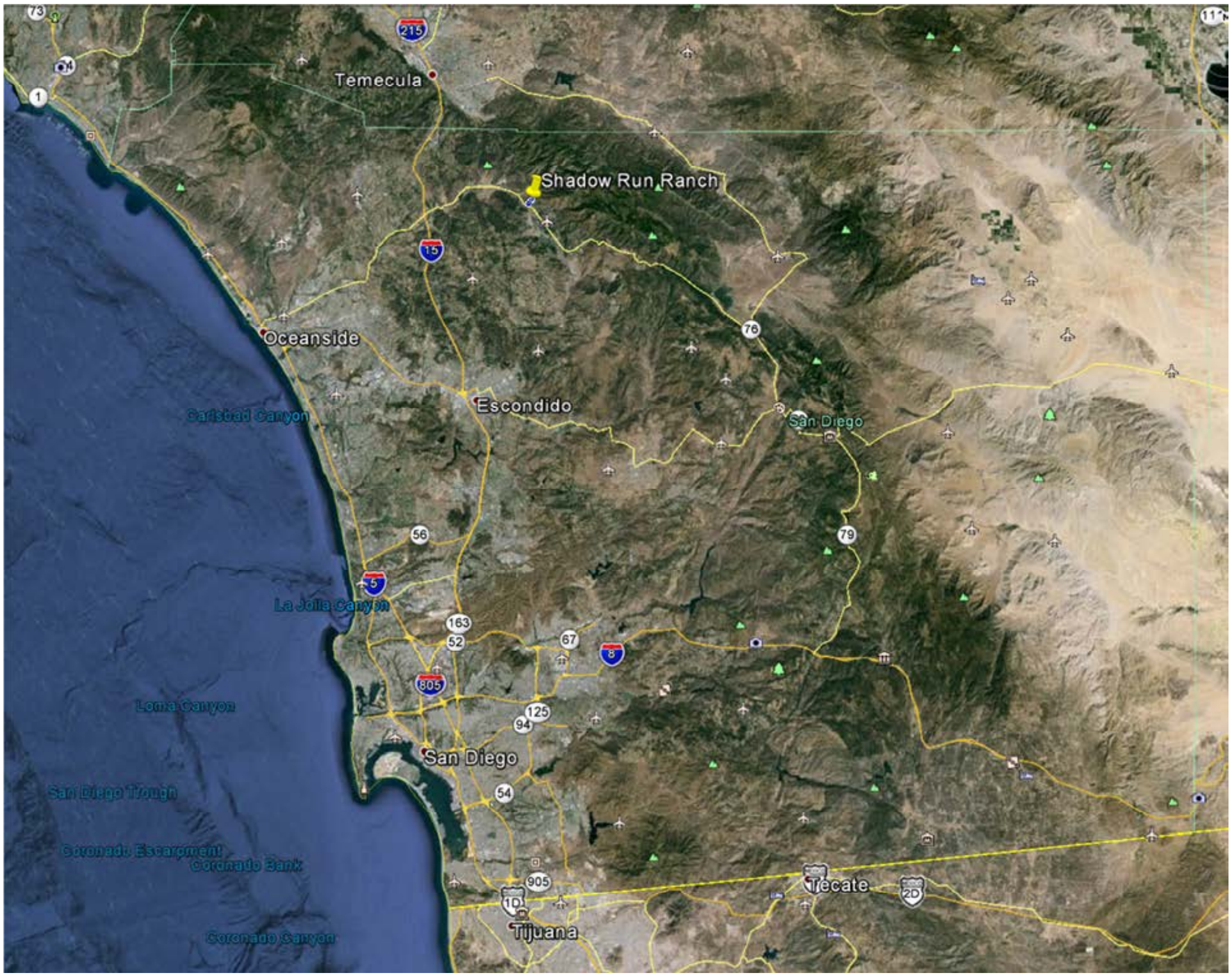
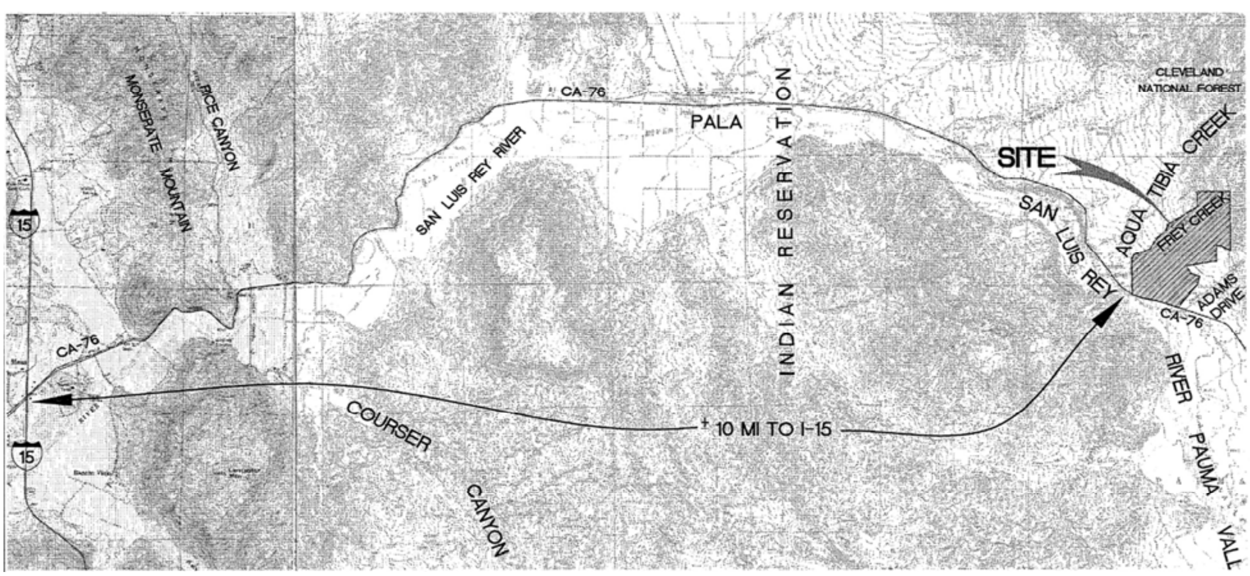
Tuesday, November 26, 2013

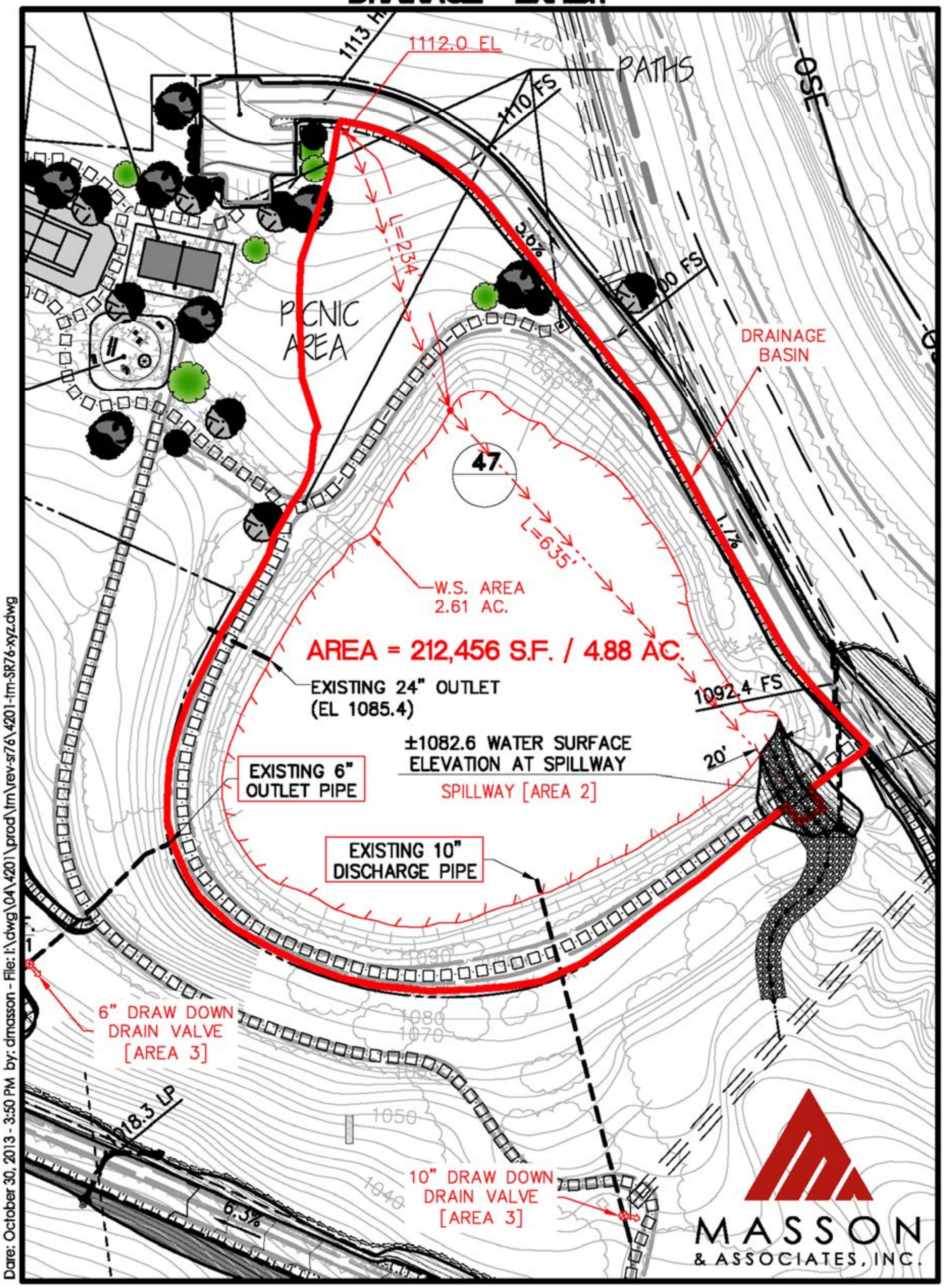
Regional map:



Project location Map:



DRAINAGE - EXHIBIT



Page (1)

PROJECT NO.:
DESCRIPTION:
CALCULATED BY: DATE:
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SHEET OF
SCALE:

MASSON & ASSOCIATES, INC.

Calculating Q for 1000 years

soil Type D ⇒ C = 0.35 From Table 3.1 , A = 4.88 Ac

$T_c = T_i + T_t$

$T_i = 6.9$ min from Table 3.2 (for 10% slope, slope on site is around 12%)

$T_t = 0$ ∴ $T_c = 6.9$ min

By calculation Q for years 2, 5, 10, 25, 50, 100, we will find the fraction for Q between $Q_{2,5,10,25,100}$ to Estimate the Q_{1000}

year 2

$P_6 = 1.68$ in $P_{24} = 2.7$ in $\frac{P_6}{P_{24}} = \frac{1.68}{2.7} \times 100 = 62\%$ ✓ ∴ $P_6 = 1.68$ in

∴ I = 3.7 in/hr (From figure 3.1)

$Q_2 = CIA$ $Q_2 = 0.35 \times 3.7 \times 4.88 = 6.3$ cfs

year 5

$P_6 = 2.2$ in $P_{24} = 4.1$ in $\frac{P_6}{P_{24}} = \frac{2.2}{4.1} \times 100 = 53.7\%$ ✓ ∴ $P_6 = 2.2$ in

∴ I = 4.4 in/hr (figure 3.1)

$Q_5 = CIA$ $Q_5 = 0.35 \times 4.4 \times 4.88 = 7.5$ cfs

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Page (2)

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Year 10

$P_6 = 2.43$ in $P_{24} = 5.2$ in $\frac{P_6}{P_{24}} = \frac{2.43}{5.2} \times 100 = 46.7$ ✓ ∴ $P_6 = 2.43$ in

$I = 4.8$ in/hr (figure 3.1)

$Q_{10} = CIA$ $Q_{10} = 0.35 \times 4.8 \times 4.88 = 8.2$ cfs

Year 25

$P_6 = 2.9$ in $P_{24} = 5.6$ in $\frac{P_6}{P_{24}} = \frac{2.9}{5.6} \times 100 = 51.8\%$ ✓ ∴ $P_6 = 2.9$

$I = 5.8$ in/hr (figure 3-1)

$Q_{25} = CIA$ $Q_{25} = 0.35 \times 5.8 \times 4.88 = 9.9$ cfs

Year 50

$P_6 = 3.3$ in $P_{24} = 6.7$ in $\frac{P_6}{P_{24}} = \frac{3.3}{6.7} \times 100 = 49.3\%$ ✓ ∴ $P_6 = 3.3$ in

∴ $I = 6.8$ in/hr (figure 3-1)

$Q_{50} = CIA$ $Q = 0.35 \times 6.8 \times 4.88 = 11.6$ cfs

Year 100

$P_6 = 3.7$ in $P_{24} = 7.5$ in $\frac{P_6}{P_{24}} = \frac{3.7}{7.5} \times 100 = 49.3\%$ ✓ ∴ $P_6 = 3.7$ in

$I = 7.7$ in/hr (figure 3-1)

$Q_{100} = CIA$ $Q_{100} = 0.35 \times 7.7 \times 4.88 = 13.2$ cfs

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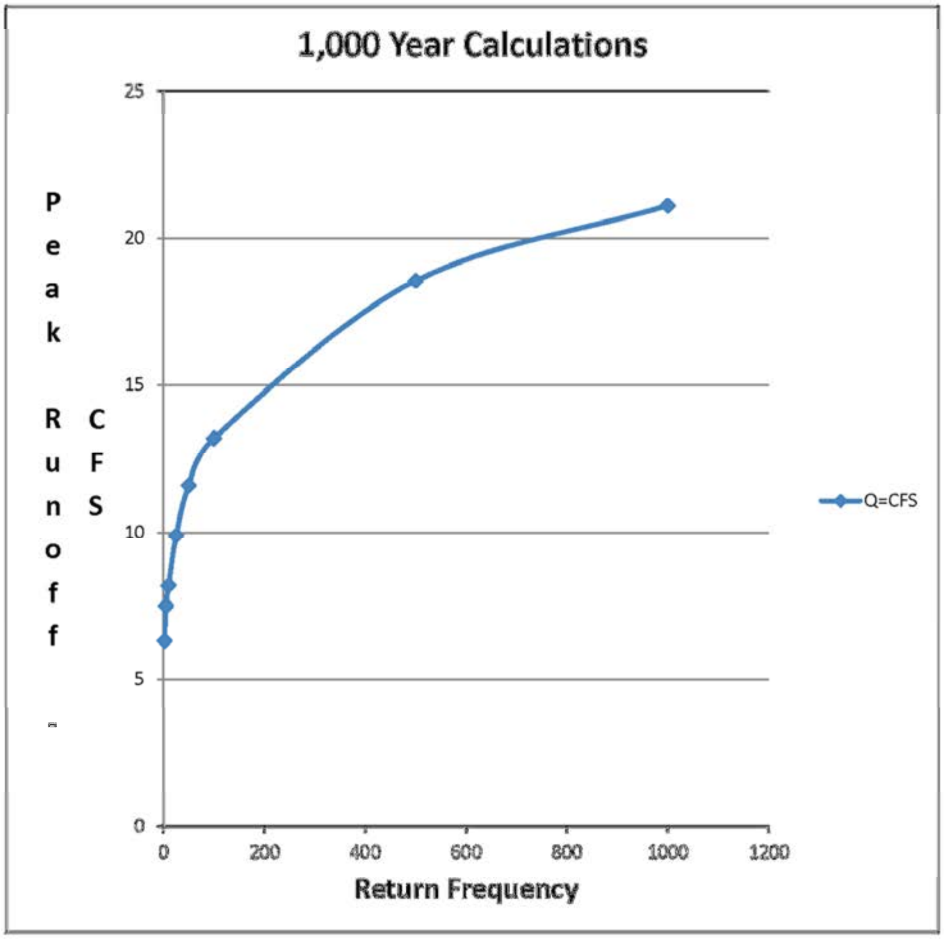
Yr	Q=CFS
2	6.3
5	7.5
10	8.2
25	9.9
50	11.6
100	13.2
500	18.56
1000	21.1



Methodology:

The peak runoff for the 2, 5, 10, 25, 50 and 100 year frequency storms have been calculated on the previous page and summarize on this page, in accordance with the requirements of the County of San Diego Hydrology Manual. The relationship between the values of the storms remains fairly consistent. The relationship between runoff intensity / flow is 1.6:1 for storm frequencies that have a return frequency that is 10:1. For example:

The flow for a 50 year storm is 1.6 times the flow rate for a 5 year storm and the flow rate for a 100 year storm is 1.6 times the flow rate for a 10 year storm.

Using this relationship we can extrapolate and calculate the peak flow rate for a 1000 year storm, which would be 1.6 times the 100 year storm (or 21.1 cfs).

page (4)

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PROJECT NO.:
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SCALE:

We calculation Q fraction between 50/5, 100/10 to find out fraction between 1000/100 :

$\frac{50}{5} = \frac{11.6}{7.5} = 1.55$ $\frac{100}{10} = \frac{13.2}{8.2} = 1.6$

$\Rightarrow Q_{1000} = 13.2 \times 1.6 = 21.1$ cfs

We use weirs equation for trapezoidal cross section to find H in feet:

$Q = CLH^{3/2}$ $H = \left(\frac{Q}{CL}\right)^{2/3} = \left(\frac{21.1}{3.1 \times 20}\right)^{2/3} = 0.5$ ft

for C value please refer to table 5.9 from Hand Book of Hydraulics (Brater and King)

Therefore, we have shown that the spillway is more than sufficient to convey the 1000 year storm.

MASSON
& ASSOCIATES, INC.
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May 31, 2013

Sherrill Schoepe
Shadow Run Ranch, LLC
Post Office Box 1249
Pauma Valley, CA 92061

RE: Scoping letter comment DPLU 20-5

Dear Ms. Schoepe:

This letter is to address DPLU Comment #20-5, in the County of San Diego EIR Scoping Letter dated December 14, 2012, it states:

> Page 6-7, Other Hazards. The geotechnical report states that the project drainage system should be checked for its ability to handle short-term, concentrated flows if significant reservoir overtopping were to occur during an earthquake. Please include an addendum to the geotechnical report which provides a detailed evaluation of the project drainage system and whether it can handle short-term concentrated flows if significant reservoir overtopping were to occur. The addendum should consider the worst case scenario of failure of the existing reservoir embankment in its evaluation. The addendum should include specific design measures as necessary to dissipate and/or divert flows to levels that ensure the safety of all proposed house pads to be placed below the dam. The addendum shall include the following concluding statement and must be signed and stamped by a California Certified Engineering Geologist and if necessary a California Licensed Civil Engineer: "Based on the available information described in this addendum, it is the opinion of the undersigned, that the measures described herein are sufficient to assure the house pads would be safe from the potential effects of dam inundation at the site." 12/14/2012 2nd Request. This comment was not addressed.

Per this request we have looked into the proposed downstream drainage system and have determined that the proposed downstream drainage system is sized appropriately to convey the water outlined in the seiche overtopping scenario italicized below for the short term concentrated flow.

> *From URS - We modeled the reservoir water level like a seiche, with a 1-ft wave running up the inner slope of the reservoir. Based on this we estimate the reservoir could overflow (or overtop) at a rate of about 0.2 ft3/ft-sec.*

The area of most concern is the southwesterly portion of the reservoir facing the proposed project. This area has a total overtopping rate of 62 cfs. This area is upstream from a proposed 1,415 s.f. hydro-modification basin 2A.1. HMP basin 2A.1 has been designed to accommodate water storage of 1,179 cubic feet, which is several times the amount of the overtopping volume. This basin has a 36" stand pipe and an outlet flow capacity of 24 cfs. Therefore, the overtopping flows would not exceed the original design capacity of the proposed drainage system.

Solved.

With regard to the worst case scenario of failure, by piping, we reference the URS letter dated May 22, 2013 which includes a proposal for a synthetic liner system. Preliminary recommendations for the liner, based on input from URS and liner manufacturers, are attached. In our opinion, a properly designed and installed impervious liner system would be sufficient to mitigate the hazard of piping related seepage from the reservoir.

Sincerely,

Bruce A. Tait, QSP/QSD
Director of Engineering

Cc: David Schug
Cc: Robert Hingtgen

OPERATION & MAINTENANCE PLAN

For
SHADOW RUN RANCH

TM 5223

Preparation/Revision Date:

November 21, 2013

Prepared for:

Sherrill Ann Schoepe, General Partner

Shadow Run Ranch, LLC

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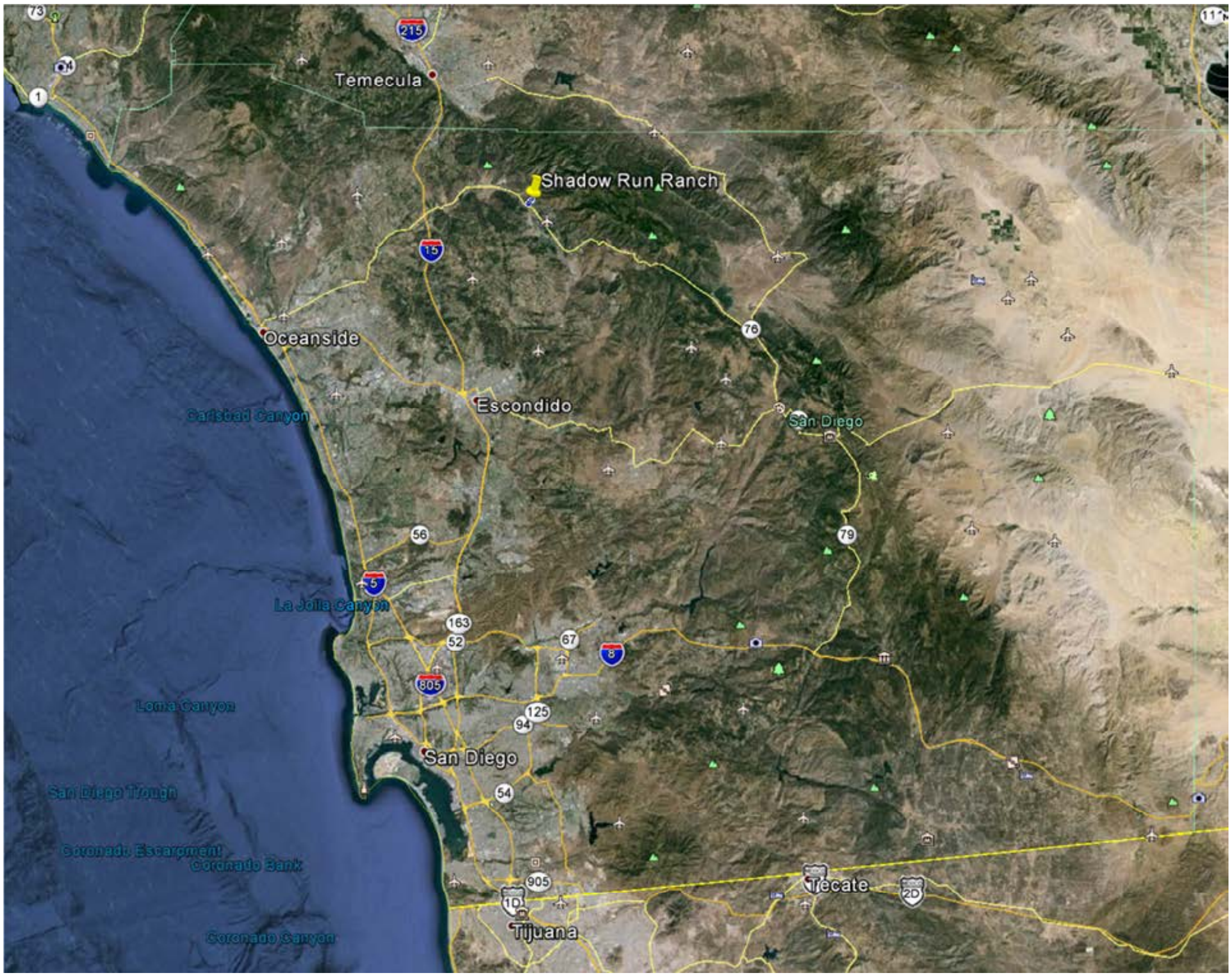
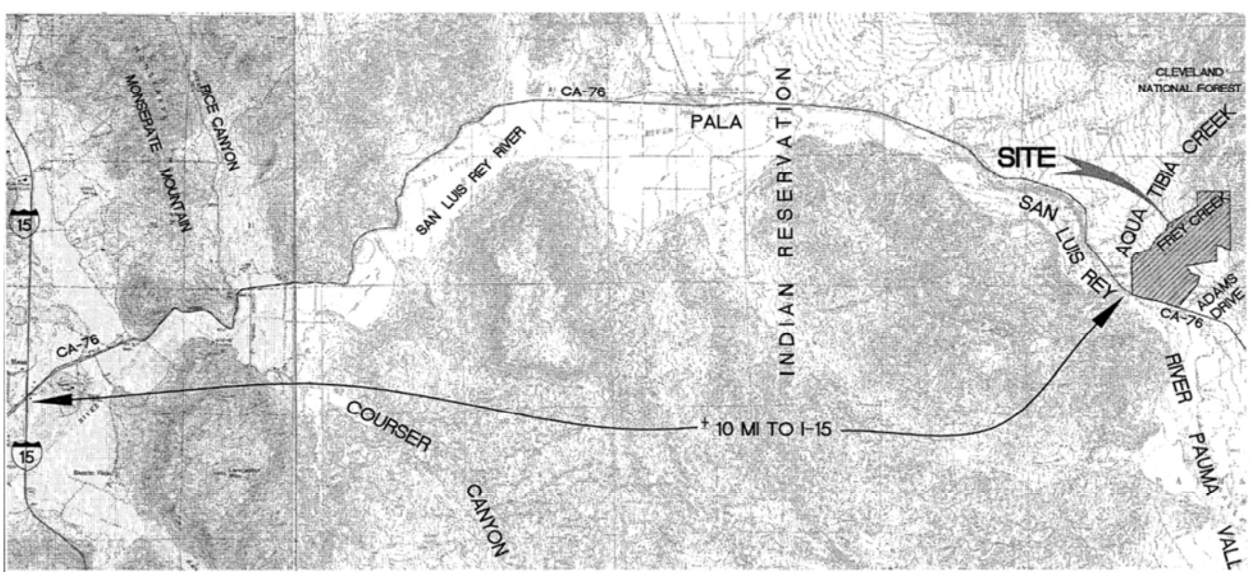
Tuesday, November 26, 2013

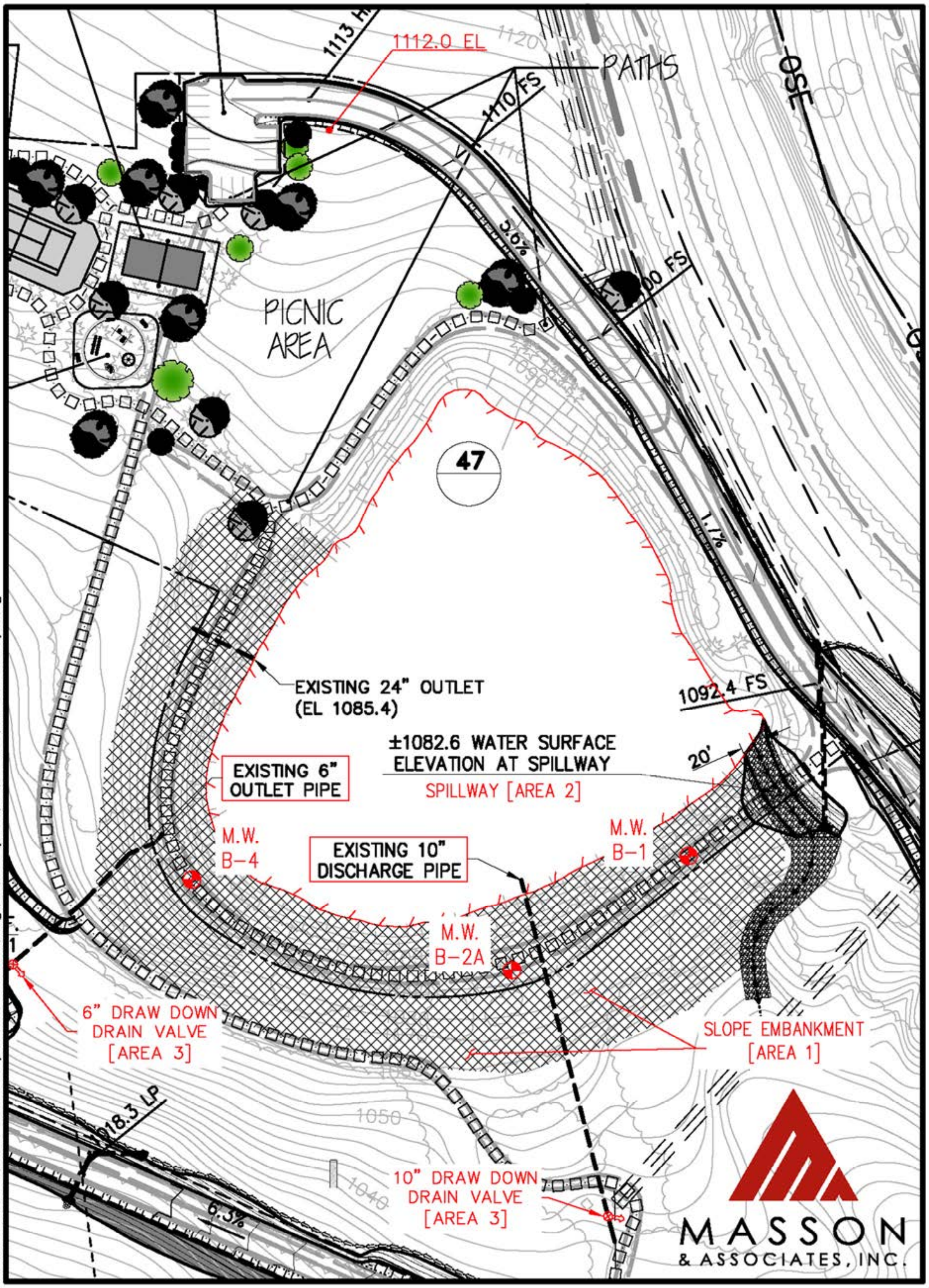
Regional map:



Project location Map:



Reservoir area:



1.0 Purpose of Document

The proposed project's HOA, County of San Diego as well as the ongoing agricultural operations and future home owners of the project are stakeholders in this Operations and Maintenance Plan (O&M). The following is an outline of the system and elements affected by this O&M. The operation and maintenance of the existing reservoir are the responsibility of the project proponent. They will enter into a MANAGEMENT AND DISCHARGE CONTROL MAINTENANCE AGREEMENT with the County of San Diego to implement this O&M. The operation and maintenance associated with the existing reservoir are discussed below. The discussion includes a routine action, maintenance indicator, field observation methods, frequency, and maintenance activity. Costs associated with each activity are included. The scope and purpose of this O&M is to ensure the operational items associated with the existing reservoir are working properly and the safety and stability of the reservoir are maintained at optimum working levels.

The three primary maintenance areas for the reservoir are as follows:

1. Reservoir embankment [AREA 1]
    a. Stability
    b. Landscaping
    c. Irrigation
    d. Burrowing animals
2. Reservoir spillway [AREA 2]
    a. Stability
    b. Energy dissipaters
    c. Scour
3. Reservoir drain lines [AREA-3]
    a. Pipeline condition
    b. Shutoff valves
    c. Drain valves
4. Monitoring wells [AREA-4]
    a. Depth to ground water

The landscape architect should choose plant coverage for slope protection and erosion control along the outer edge of the reservoir embankment that will be high in erosion control value with shallow root systems and which will deter small burrowing animals. The reservoir slope embankment shall be watered sparingly to maintain landscape coverage for erosion control.

2.0 Facilities and Resources

The facilities and resources identified to be managed and inspected are shown the graphic on page 3, "Reservoir areas." The management and inspection of the reservoir will be the responsibility of the ownership of the recreational open space lot 47 of TM 5223 (the homeowners association (HOA)). The property manager(s) of the HOA shall at all times have a qualified grove manager(s) that will be employed by said HOA and have a set number of hours dedicated monthly to inspect and fill out inspection reports in conformance with this Operations and Maintenance Plan. General qualifications shall be on-site training for all systems and grounds. The HOA shall dedicate $____ monthly and have a reserve fund to anticipate any startup and ongoing maintenance of the reservoir systems.

3.0 Operations

The goal of this O&M is to ensure safety and operational conditions of all reservoir systems on a monthly basis. Testing the valves on the two (2) 6” irrigation/down drain lines and the one (1) 10” down drain line shall be maintained and operational to be opened in case of an emergency to drawdown ½ the reservoir capacity within 7 days, and completely drain the reservoir within 20 days.

The report shall contain, at a minimum, the following items:

Inspection Protocol: Inspections will include:

- Date of inspection
- Reservoir level
- Water use in previous month
- Note any unusual signs of changed water levels
- Condition of the spillway
- Check scour and erosion
- Condition of the 6” drain line
- Condition of the 10” drain line
- Overall embankment stability
- Any signs of slope movement
- Any signs of seepage around or below reservoir
- Any rock falls nearby
- Vegetation control
- Control of burrowing animals
- Irrigation control
- Recommendations for repairs
- Three existing monitoring wells
- Record depth to ground water

Quarterly inspections:

The grove manager shall visually inspect on a quarterly basis, the entire slope embankment [Area 1] of the reservoir including the spillway [Area 2] looking for any settlement, surface cracking, burrowing animals, overwatering and seepage. In addition the (2) 6” drain line pipes and (1) 10” drain line pipe [Area 3] shall be tested monthly, to ensure the valves and drain capacities are working properly.

On a quarterly basis, or if an earthquake is felt at or near the reservoir (as outlined below), measure and record the depth to groundwater in the three existing monitoring wells at the top of the reservoir embankment [Area 4]. The HOA shall be notified immediately if any substantially changed groundwater levels are indicated. The reports shall be submitted to the HOA and COSD within 10 working days of the date of the inspection and will be filed in the HOA manager’s office and shall be stored for 5 years.

Special inspections:

If an earthquake occurs at or near the reservoir, or has been reported to occur, within the following criteria, immediate inspection shall be required:

- M $\geq$ 4.0 w/in 25 miles,
- M $\geq$ 5.0 w/in 50 miles,
- M $\geq$ 6.0 w/in 75 miles,
- M $\geq$ 7.0 w/in 125 miles,
- M $\geq$ 8.0 w/in 200 miles,

If such an earthquake occurs, the following items shall be inspected and reported upon:

- Date of inspection
- Reservoir level
- Note any unusual signs of changed water levels
- Condition of the spillway
- Condition of the 6" drain line
- Condition of the 10" drain line
- Overall embankment stability
- Any signs of slope movement
- Any signs of seepage around or below reservoir
- Any rock falls nearby
- Recommendations for repairs

Repairs recommended in the inspection reports shall be accomplished within: 10 working days, or immediately for repairs that are mandated by reservoir stability issues.

4.0 Maintenance / Repair

IMPLEMENTATION AND MAINTENANCE REQUIREMENTS

Reservoir Embankment

The primary maintenance requirements for the reservoir embankment are as follows:

- Weed, prune, and water, especially during plant establishment
- Keep landscape healthy and clean
- The grounds shall be free of large deep rooted trees and bushes
- Maintain control of small burrowing animals
- When encountered burrowing animals shall be removed and any holes filled in

Aesthetic and Functional Maintenance:

Aesthetic maintenance is important for public acceptance of facilities. Functional maintenance is important for performance and safety reasons.

Both forms of maintenance will be combined into overall system maintenance.

Aesthetic Maintenance

The following activities will be included in the aesthetics maintenance program:

- Replace dead or dying plants.
- Weed Control.
- Weeds will be removed through mechanical means.
- Herbicide will not be used because these chemicals impact the water quality.
- Prune overgrown plants.

Functional Maintenance

Components of a Functional Maintenance program include Preventive Maintenance and Corrective Maintenance.

a. **Preventive Maintenance -** Preventive maintenance activities to be instituted are:

- Trash and Debris. During each inspection, debris and trash removal will be conducted.
- Down drain outlet piping: Visual inspection of (2) 6” drain line pipes and (1) 10” drain line pipe shall be inspected and checked for leaking and or corrosive condition.
- Test down drain system. During each inspection, each down drain pipe shall be tested. Open valves check piping for any leaking.
- Sediment Removal. Sediment accumulation, as part of the operation and maintenance program at the spillway, will be monitored quarterly during the dry season, and after every large storm (0.50 inch), and monthly during the wet season. If accumulation of debris or sediment is determined to cause of decline in design performance, prompt action (i.e., within ten working days) will be taken to restore to design performance standards. Actions will include removal of sediment. Characterization and appropriate disposal of sediment will comply with applicable local, county, state, or federal requirements.
- Removal of Standing Water - Standing water must be removed if it contributes to the development of aquatic plant communities or mosquito breeding areas. Water standing for more than 96 hours will be removed at outflow.
- Fertilization – Any vegetation seed mix will be designed so that fertilization and irrigation (after establishment of the planting) is not necessary. Fertilizers will not be used to maintain the vegetation.
- On a quarterly basis, and if an earthquake is felt at or near the reservoir (as outlined above) measure and record the depth to groundwater in the three existing monitoring wells at the top of the reservoir embankment. Notify the HOA immediately if any substantially changed groundwater levels are indicated.

b. **Corrective Maintenance -** Corrective maintenance is required on an emergency or non-routine basis to correct problems and to restore the intended operation and safe function.

Corrective maintenance activities include:

- Removal of Debris and Sediment - Sediment, debris and trash, which impede the hydraulic functioning of reservoir spillway and vegetative growth, will be removed and properly disposed.
- Down drain outlet piping – two (2) 6" drain line pipes and one (1) 10" drain line pipe. Paint exposed piping, poly-wrap pipe protection if necessary, replace damaged sections.
- Test down drain system. Replace valves if necessary.
- Embankment and Slope Repairs – Damaged to slopes and embankments will be evidenced by erosion or collapsed surface areas. Once deemed necessary, damage to the slopes of the reservoir embankment will be repaired (within 10 working days).
- Erosion Repair – Erosion will be evident by rills or small gullies in the surfaces of the reservoir embankment slope. Corrective steps will be taken to prevent loss of soil and any subsequent danger to the performance of the reservoir embankment. There are a number of corrective actions that can be taken. These include temporary measures such as erosion control blankets or reducing flow through the area. Designers or contractors will be consulted to address erosion problems if the solution is not evident.
- Elimination of Animal Burrows - Animal burrows (evidenced by holes & mounds) will be filled and steps taken to remove the animals if burrowing problems continue to occur (filling and compacting). If the problem persists, vector control specialists will be consulted regarding removal steps. This consulting is necessary as the threat of rabies in some areas may necessitate the animals being destroyed rather than relocated. If the reservoir embankment performance is affected, abatement will begin. Otherwise, abatement will be performed annually in September.
- General Facility Maintenance - In addition to the above elements of corrective maintenance, general corrective maintenance will address the overall facility and its associated components. If corrective maintenance is being done to one component, other components will be inspected to see if maintenance is needed.
- Replace dead or dying plant material.

Regulatory Assurance

Maintenance is assured by the Major Use Permit # ______ and conditions of approval, as well as a MANAGEMENT AND DISCHARGE CONTROL MAINTENANCE AGREEMENT with the County of San Diego, which will be recorded against the property and run with the land.

Maintenance Costs

A detailed cost breakdown for the operation & maintenance of each area / system are attached and made part of this document. Total estimated annual costs for each are:

Reservoir embankment = $_______
Down drain piping = $_______
Water valves = $_______
Landscaping = $_______
Irrigation = $_______
Burrowing animals = $_______

Inspection Frequency

- All items above will be monitored quarterly during the dry season, and after every large storm (0.50 inch), and monthly during the rainy season.
- Condition of vegetation: Monthly
- After each seismic event as listed above.

Each inspection will be fully documented and made available upon request. Records will be kept for a minimum of 5 years.

Appendix

Sherrill Schoepe Shadow Run Ranch, LLC Post Office Box 1249 Pauma Valley, CA 92061

RESERVOIR DRAWDOWN CALCULATIONS

For
SHADOW RUN RANCH

TM 5223

Preparation/Revision Date:

November 25, 2013

Prepared for:

Sherrill Ann Schoepe, General Partner

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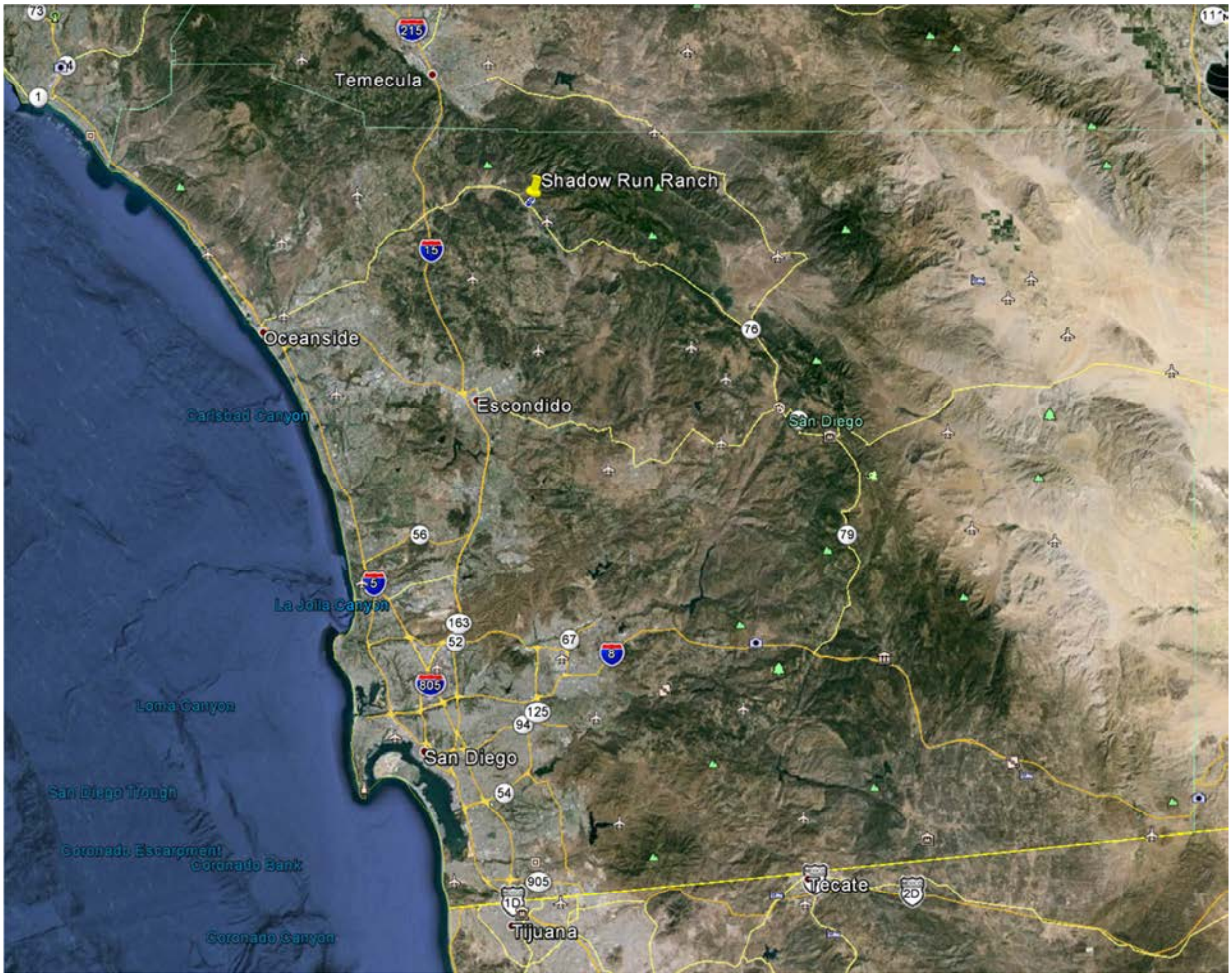

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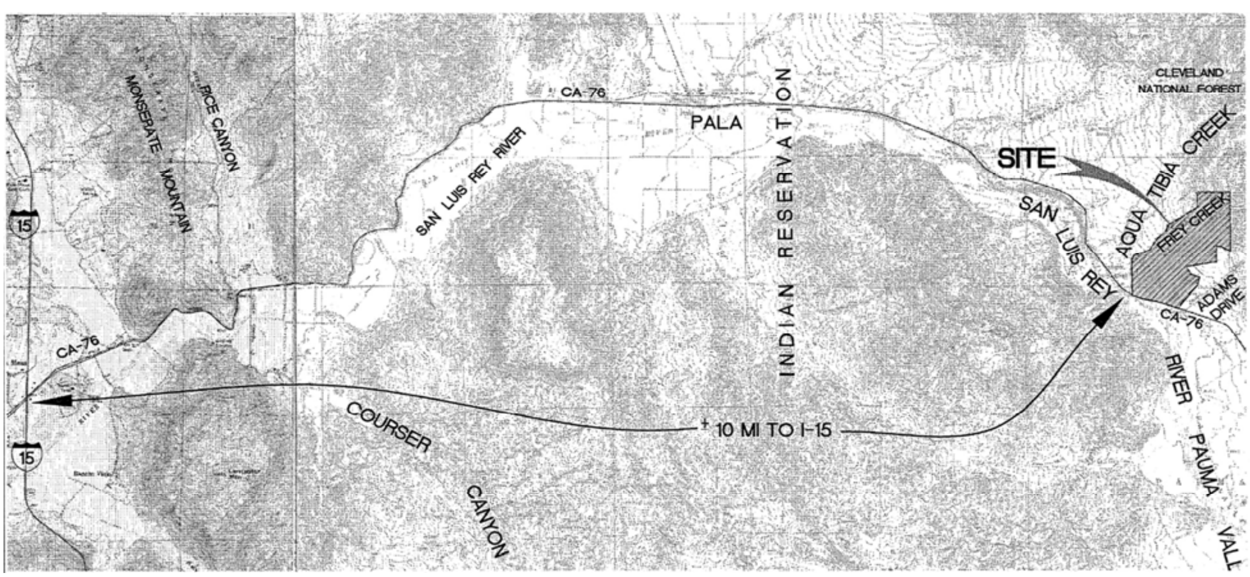
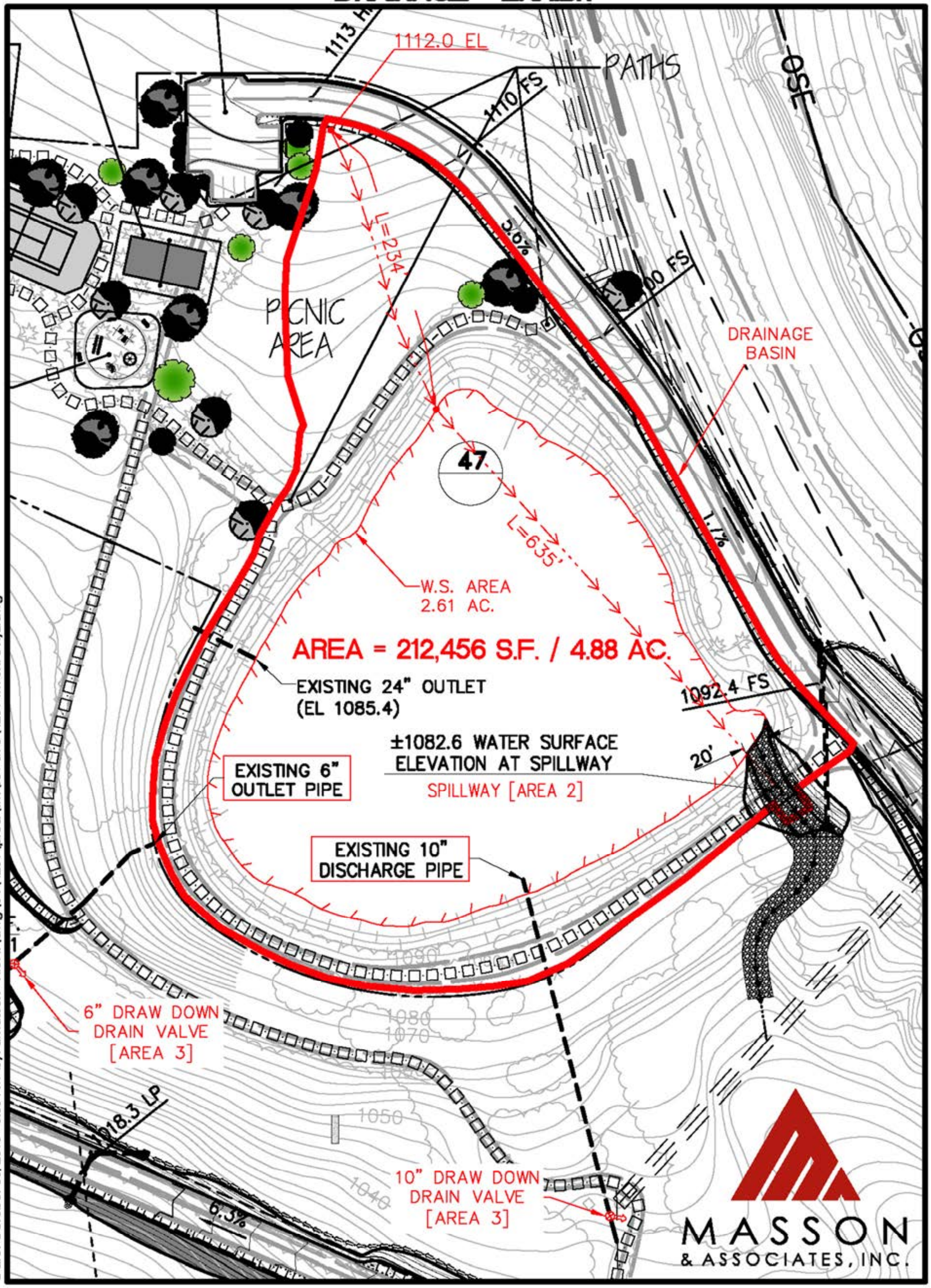


DRAINAGE - EXHIBIT



Per the exhibit above there are two existing drawdown pipes 6” and 10” that can be used to drain the entire reservoir within 3 days per the calculations provided below.

MASSON & ASSOCIATES, INC.

PROJECT NO.:

DESCRIPTION:

CALCULATED BY: DATE:

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Calculation for Two pipes 6", 10" that will drain the the total volume of the reservior in x days:

$n = 0.017$ steel $Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$

Pipe, $D = 6''$ $r = 3'' \Rightarrow r = 0.25$ ft

$S = \frac{1062 - 1050}{120} = 0.1 \times 100 = 10\%$

$R = \frac{A}{P} = \frac{\pi r^2}{2\pi r} = \frac{r}{2} = \frac{0.25}{2}$

$R = 0.125$ ft

$Q_1 = \frac{1.49}{0.017} \times 3.14 (0.25)^2 \times (0.125)^{2/3} \times (0.1)^{1/2} = 1.36$ cfs

Pipe: $D = 10'' \Rightarrow r = 5'' \Rightarrow r = 0.42$ ft, $S = \frac{1069 - 1048}{295} = 0.064 \times 100 = 6.4\%$

$R = \frac{A}{P} = \frac{\pi r^2}{2\pi r} = \frac{r}{2} = \frac{0.42}{2} = 0.21$ ft

$Q_2 = \frac{1.49}{0.017} \times 3.14 (0.42)^2 \times (0.21)^{2/3} \times (0.064)^{1/2} = 4.36$ cfs

$Q_T = Q_1 + Q_2 = 1.36 + 4.36 = 5.72$ cfs, $V = 34.5$ Ac-feet

$V = 1,502,820$ cubic feet

$\frac{V}{Q_1 + Q_2} = \frac{1,502,820}{5.72} = 262730$ s

$= 4378$ min

$= 72.98$ hr

$= 3.0$ days

∴ These Two pipes 6" and 10" will drain the reservior, volume = 34.5 Ac-feet in 3 days.

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