



SEISMIC STABILITY EVALUATIONS (SSE2)			
Number:	SSE2 – TM – 1LN		
Subject:	Initial Review of Available Data for Lenihan Dam		
Prepared by:	Robert Kirby, PE, GE Richard Harlan, PG, CEG	Date:	11/24/2010
Reviewed by:	Yoshi Moriwaki, PE, GE Guilaine Roussel, PE	Date:	11/24/2010

1.0 INTRODUCTION

Task 3 of our scope of work for Phase A of the SSE2 Project consists of the Data Review, Work Plans, and Phase 1 Site Investigations for Steven Creek and Lenihan Dams. The purpose of Task 3 is to design a field investigation and laboratory testing program that is focused on providing the supplemental data necessary for completion of the engineering analyses for the seismic stability evaluations of the dams, and to implement the first phase of the site investigations. Fulfilling this purpose requires (a) initial review and evaluation of the completeness of the available geotechnical field and laboratory data, (b) evaluation of the nature and distribution of potential soils left in place beneath the dam embankment, (c) geologic reconnaissance, (d) completion of preliminary engineering analyses to help define the key information required to complete the detailed engineering studies, (e) identification of data gaps, (f) development of a work plan for field investigations and laboratory testing that addresses these data gaps, and (g) completion of the Phase 1 investigations. Task 3 is logically followed by Task 4 - Phase 2 Site Investigations and Laboratory Testing.

Task 5 is the Geotechnical Site Characterization. The purpose of this task is to review, evaluate and interpret, in detail, all the available site information that provides the basis for the seismic stability evaluations. Task 6 consists of the Engineering Analyses of Seismic Stability and includes a variety of detailed engineering analyses that are based on the engineering properties of the materials and other relevant site data developed in Task 5.

During the development of our contractual scope of work, the sequencing of Tasks 3 through 6 was assumed to be linear with some overlap, i.e., Task 3 is followed by Task 4, Task 4 is followed by Task 5, etc. This approach worked well for Stevens Creek Dam where our review of the available data allowed us to prepare a draft work plan for site investigations and laboratory testing at the dam. However, our work on Task 3 at Lenihan Dam suggests that this approach is not appropriate for this dam for the reasons discussed below.

Contrary to the mapping of soils left in place beneath Lenihan Dam shown by the District in their Foundation Analysis Report of SSE-2 Dams (SCVWD, 2010a), our detailed review of the data, including data from soil borings that were not reviewed by the District as part of their study, strongly indicates that no soil was left in place beneath the dam. This is a very significant and positive finding from a dam safety perspective because it means that potential liquefaction of soils left in place beneath the dam is not an issue. The finding that no soils were left in place beneath the dam is consistent with the assumption made by the Division of Safety of Dams (DSOD) during their design reviews completed in 2006 and 2007 (DSOD, 2006c and 2007a).

There is an enormous amount of geotechnical data available for Lenihan Dam that includes the approximate number of the following items from investigations located at the dam and spillway, and dam instrumentation that is currently being monitored:

- 80 borings;
- 2 test pits;
- 4 cone penetrometer probes;
- 4 sets of cross-hole seismic surveys and downhole seismic surveys at cross-hole locations;
- 32 vibrating wire piezometers (not including abandoned outlet pipe piezometers);
- 2 inclinometers with 13 in-place inclinometer sensors;
- 16 permeability tests;
- 10 unconfined compression strength tests;
- 9 Unconsolidated Undrained (UU) triaxial tests;
- 16 Isotropically Consolidated Undrained (ICU) triaxial tests;
- 29 cyclic triaxial tests;
- 4 resonant column tests;
- 5 consolidation tests;
- Compaction tests;
- Many physical and index property tests; and
- Periodic surveys on 15 survey monuments (10 on the dam and 5 on the bike path).

From our work on Task 3 we have concluded the following:

1. There appears to be sufficient information available to define the geometry of the dam and its foundation.
2. Because of the enormous amount of available geotechnical data, a detailed review and thorough evaluation of the available data on the properties of the various zones of the embankment (as indicated by the construction records, field investigations and laboratory tests completed to date) may provide most of the information necessary to support the engineering analyses.

Consequently, we recommend that our approach be adjusted so that we complete the majority of Task 5 – Geotechnical Site Characterization based on the detailed review, evaluation and interpretation of the wealth of available data; complete some of the engineering analyses based on the available data; and then decide what supplemental field and laboratory data are necessary

to reduce the uncertainties in the results of the seismic stability analyses and prepare a work plan to obtain these data.

This Technical Memorandum describes the results of the work we completed under Task 3 and provides a starting point for beginning Task 5. Section 2.0 provides a general description of the dam and of the nature of the embankment and foundation materials. Section 3.0 is a summary of the chronology and scope of the previous investigations at the site. Section 4.0 describes the review and evaluation of the existing data that led us to the conclusion that Lenihan Dam is most likely founded entirely on bedrock. Section 5.0 summarizes the key geotechnical issues to be addressed in the seismic stability evaluation of the dam and Section 6.0 discusses our review of Cone Penetrometer test (CPT) data as an example of the quality of the existing data we have available for use. Section 7.0 presents our conclusions and recommendations and Section 8.0 contains a list of references and documents that were reviewed during our Task 3 activities.

2.0 GENERAL DESCRIPTION OF DAM AND FOUNDATION CONDITIONS

2.1 Geometry and Significant Features

Figure 1A is an aerial photograph of Lenihan Dam that shows the outline of the embankment, the limits of Figures 1B and 1C, and an overview of the locations of previous explorations that are discussed in Section 3.0. Figures 1B and 1C provide larger scale location plans that include labels with the identifying name of the exploration or instrument. Borings that extended to rock are identified using the larger symbols, as shown in the legend. Figure 2 contains transverse sections through the current configuration of the dam at Stations 14+10 and 15+95 that are representative of dam zoning and conditions near the center of the valley. The locations of the two sections are shown on Figures 1A, 1B and 1C. These sections have been heavily instrumented with piezometers, and the piezometer locations are shown on the sections. In some cases (e.g. LVP-19), it was necessary to project the location of piezometers installed at nearby dam stations onto the sections shown and, in the case of LVP-19, this caused a piezometer that was actually installed in the embankment to be shown as located in rock on Section B of Figure 2. The location of previous borings are not shown on the cross-sections of Figure 2 for clarity but their plan locations are shown on Figures 1B and 1C, and the information from these borings on depth to rock is presented and discussed in later sections of this memorandum.

As shown in Figure 2, Lenihan Dam was constructed as a compacted earth dam with upstream and downstream shells, core and drainage zones. The dam is about 195 feet high as measured from the lowest point in the foundation beneath the axis to the crest, and about 207 feet high as measured from the lowest point of the downstream toe to the crest.

Following the Loma Prieta Earthquake, it was determined that the crest of the dam had settled about 2.3 feet since construction because of a combination of long-term consolidation and seismically-induced deformation from the Loma Prieta Earthquake. The crest was subsequently raised by up to 4.5 feet, and the spillway chute walls raised by up to 6 feet, during the 1996 freeboard restoration project. Thus, the crest is currently at nominal Elev. 673 feet (all elevations have been converted to NAVD88 vertical datum) and is about 40 feet wide, 830 feet long, and cambered. In general, the upstream face is inclined at 5.25 to 5.5 H :1V. The downstream slope

is inclined at 2.5 to 3H:1V. The concrete-lined, un-gated ogee crest spillway is located on the left abutment, with a nominal crest elevation of 653 feet.

Appendix C of the Second Summary Surveillance Report (SCVWD, 2007) provides a detailed discussion of the special construction methods that were allowed to handle the delayed availability of materials. During construction, delay in the design of the spillway led to delays in construction of the downstream shell of the dam that was to be built with materials from the spillway excavation. In addition, the off-site quarry had difficulty generating sufficient quantities of materials for the inclined drain zone. In order to expedite construction, engineers from DSOD allowed the contractor to finish constructing the core and the upstream shell of the dam without constructing any of the inclined drain or downstream shell, which resulted in modifications to the original design. The inclined drain does not extend the full height of the core and, on the abutment margins, does not tie directly into the blanket drain. A wedge of sandier transition material was reportedly constructed between the lower core and the inclined drain. Additional important details are provided in Appendix C of the District's report (SCVWD, 2007).

The original low level outlet pipe was extensively investigated after it experienced several partial collapses of the steel liner. The investigation showed that this occurred because of excess external pressure combined with vacuum pressures, corrosion, and out of roundness, and the outlet was subsequently repaired. The low level outlet was recently replaced by an outlet tunnel through the right abutment. The low level outlet pipe was filled with grout after completion of the outlet tunnel.

2.2 Existing Instrumentation

Twenty-three (23) vibrating wire (VW) piezometers monitor piezometric levels in the embankment, 2 VW piezometers monitor piezometric levels within the bedrock foundation beneath the dam, and 7 VW piezometers are set in the right abutment bedrock. Data from 26 embankment and foundation piezometers are included on the cross-sections shown on Figure 2. In addition, a weir measures tunnel seepage discharge at the downstream end of the new outlet tunnel. A total of 13 automated in-place inclinometers are installed in two inclinometer casings within the embankment. Fifteen (15) survey monuments are used to monitor embankment movements: 10 on the dam crest and 5 on the bike path. An accelerometer is also installed in the control building on the dam crest.

2.3 General Nature of Embankment Materials

Materials for the core and upstream shell of the dam were obtained from borrow sources upstream of the dam. The lower portion of the core up to about Elev. 590 feet was derived from very clayey alluvial material and the materials for the upstream shell and the upper core were derived from excavation of Franciscan Complex material just upstream of the upstream toe. The downstream shell was primarily derived from the spillway excavation. Previous studies, including those by Harza (1997) and R.L. Volpe & Associates (RLVA, 1999), concluded that the core materials possess very low permeability and consolidation properties and that stress-induced pore pressures from construction and/or from the 1989 Loma Prieta earthquake appear to be very slow in dissipating.

The following Table 1 provides a summary of Atterberg limits and sieve analyses data for the zones of the dam shown on Figure 2, and a description of the material classification of each zone. These data were compiled by DSOD and presented in their Memorandum of Design Review (DSOD, 2006).

**TABLE 1
SUMMARY OF INDEX PROPERTIES**

Zone	LL (%)	PI (%)	Minus #200 Sieve (%)	Gravel (%)	Material Classification
Zone 1	37	20	32	29	SC, Clayey SAND with Gravel
Zone 2					
above Elev. 590 feet	35	16	35	23	SC, Clayey SAND with Gravel
below Elev. 590 feet	65	36	95	1	CH, CLAY
Zone 4	33	16	28	41	SC-GC, Clayey SAND and GRAVEL

The following Table 2 contains a summary of dry densities on the various zones of the embankment that was also compiled by DSOD and summarized in their Memorandum of Design Review (DSOD, 2006). DSOD indicates that index property and dry density data are not available on the drain materials, Zone 3. Forty (40) field density tests were completed during construction on samples between Elev. 480 feet and Elev. 555 feet and the results of these tests are shown in brackets in Table 2 next to the results of tests made during subsequent field investigations.

**TABLE 2
SUMMARY OF DRY DENSITIES**

Zone	Dry Density, lbs/ft ³			Number of Tests
	Maximum	Minimum	Average	
Zone 1	132.3 [133.0]	95.2 [95.0]	118.5 [118.1]	14
Zone 2				
above Elev. 590 feet	131.5	113.0	120.9	12
below Elev. 590 feet	102.2 [126.7]	92.5 [84.0]	96.6 [97.4]	10
Zone 4	143.3	100.6	123.9	41

2.4 General Foundation Conditions

The dam was constructed on Franciscan Complex bedrock, without a foundation seepage cutoff or grout curtain. The foundation predominately consists of Franciscan Complex mélangé that is typically intensely fractured to crushed shale with varying amounts of harder sandstone and greenstone blocks, some of which are up to several hundred feet in length, and lesser blocks of

serpentinite and chert. An area of more massive sandstone with interbedded shale occurs at the upper end of the spillway on the left abutment. Numerous shears, localized faults and fractures of various orientations occur throughout the foundation. Several slope failures occurred during excavation, particularly along the left side, and the dam axis was shifted downstream on the left side and upstream on the right side to take advantage of better rock conditions. The rock surface on which the dam is founded was surveyed during construction after removal of surficial soils. Figure 3 shows the topographic contours from that survey and Figure 4 provides a three-dimensional perspective of the bedrock surface. Figure 5 provides transverse and longitudinal cross-sections through the dam that supplement the primary transverse sections shown on Figure 2. It is evident from Figures 2 through 5 (and particularly the 3-dimensional view of Figure 4) that the bedrock surface has a typical valley configuration on the right side of the dam but includes massive rock knobs on the upstream left side of the dam. Further discussion of dam foundation conditions is presented in Section 4.0.

3.0 CHRONOLOGY AND SCOPE OF PREVIOUS INVESTIGATIONS

Figure 1A provides an overview of the locations of previous investigations. Figures 1B and 1C are enlargements of Figure 1A that show details at the dam and spillway, respectively. These figures include the locations of previous explorations as well as those of currently active piezometers and inclinometers. Previous explorations that extended into bedrock are shown as larger symbols while those that did not are shown as smaller symbols. Piezometers and inclinometers are also shown with different symbols on these two figures.

The following is a summary of the previous investigations

1. The first significant field and laboratory investigation at Lenihan Dam was the Seismic Safety Evaluation study by Wahler Associates (Wahler, 1982). This evaluation consisted of three episodes of field investigation in 1975, 1979 and 1981, with the final report prepared in 1982. In all, Wahler completed:
 - 18 rotary borings in the dam, several of which included frequent Pitcher Barrel sampling with some Standard Penetration Tests (SPT); 12 of these borings were extended into bedrock;
 - two trenches on the downstream slope, with in-place density testing of the embankment;
 - three sets of cross-hole shear wave tests, with seismic refraction and downhole surveys at each cross-hole site;
 - installation of 5 pneumatic piezometers in the upstream shell and core (now abandoned), and three open well piezometers in the downstream shell (no longer monitored); and
 - a large amount of classification and engineering properties testing including permeability, consolidation, compaction, and triaxial tests (UU, ICU and cyclic).

Wahler concluded that the dam had high seismic resistance and that catastrophic failure due to the maximum credible earthquake (M8.5 on the San Andreas Fault) was not likely.
2. The second significant study was performed by Earth Sciences Associates (ESA, 1987) for the Outlet Modification Project. All of the explorations for this study were located outside the limits of the embankment for the purpose of designing a relocated intake structure on the

lower upstream right abutment. However, as discussed further in Section 4.0, we used some of the exploration data generated from this work to clarify the likely level of the embankment foundation excavation in this area.

3. Following the Loma Prieta Earthquake in 1989, RLVA conducted an investigation of earthquake damage at Lenihan Dam (RLVA, 1990); this investigation included mapping and trenching of earthquake-induced cracking at the dam site.
4. The next significant study was conducted by Geomatrix in 1996 for the Lexington Dam Freeboard Restoration Project (Geomatrix, 1996). Geomatrix completed:
 - 7 shallow hollow-stem auger borings along the dam crest;
 - 14 rotary, core and flight auger borings along the sides of the spillway, ranging from 3 feet to 43 feet in depth;
 - 7 test pits, mostly along the spillway; and
 - laboratory testing including index properties, compaction, and unconfined compression (UC) tests on the shallow embankment materials, and limited UU triaxial tests on weathered rock along the spillway.
5. In 1997, Harza conducted a study to simulate the Loma Prieta Earthquake deformations at the dam using the GEFDYN program for analysis (Harza, 1997). Their work included the drilling of three borings along the maximum section of the dam, with Pitcher Barrel sampling, consolidation, and triaxial testing (ICU and cyclic).
6. Another significant study was performed in 1999 and in 2001 by RLVA and the District (RLVA, 1999; SCVWD, 2001), initially for the purpose of evaluating repeated episodes of outlet conduit damage that were first noted following the Loma Prieta Earthquake and that culminated in a Level One emergency at the dam in 1998. Work completed for these studies included a detailed seepage analysis and extensive installation of instrumentation, including most of the piezometers that are now used for monitoring. This work included:
 - 4 Cone Penetrometer Tests (CPTs);
 - 22 rotary borings with installation of 29 VW piezometers and two inclinometer casings with 13 in-place inclinometers;
 - installation of 22 piezometers along the outlet conduit (all now abandoned); and
 - limited laboratory testing including index properties, consolidation, permeability and triaxial tests (UU and ICU).
7. The most recent significant investigation at Lenihan Dam was performed by Geomatrix in 2006, for the design of the new outlet tunnel (Geomatrix, 2006). Exploration completed as part of that study was concentrated on the right abutment along the alignment of the new outlet tunnel, and included:
 - 18 rotary and core borings, some with packer testing and downhole seismic velocity, optical televiewer and acoustic logging, and installation of 5 piezometers;
 - 4 test pits;
 - seismic refraction and electric resistivity lines; and
 - laboratory testing, mostly rock strength properties for tunnel design.

Data from the preceding studies have been consolidated and reviewed in general by TGP. Other data also reviewed included as-built drawings from the original construction (dated 1956), the intake structure modifications (1989), and the freeboard restoration project (1998), and other reports related to various investigations at Lenihan Dam. In addition, numerous sets of black and white stereo aerial photographs were reviewed at the District's office, and selected sets were scanned and provided by the District for our later use. A list of the documents we reviewed is presented in Section 7.0.

4.0 FOUNDATION CONDITIONS

From a seismic stability perspective, one of the most important and common sources of potential problems is that soil deposits within the valley are left in place beneath a portion of the dam. These soil deposits, particularly young alluvium, are susceptible to liquefaction during the design earthquake. The design of Lenihan Dam was based on removal of surficial soils so that the dam would be founded on rock. However, confirmation that the soils were removed is important. The District prepared a Foundation Analyses Report for SSE2 Dams in 2010 to address this issue (SCVWD, 2010a). We have made an independent and detailed assessment of foundation conditions and concluded that it is unlikely that there are any significant areas of thicker, in-place soils remaining in the foundation between the embankment and the underlying Franciscan Complex bedrock.

As part of our review of various geologic data related to foundation conditions, we closely examined a number of maps and reports that document conditions in the foundation area that existed prior to, during, and after construction of the dam. These data sources include:

1. the pre-construction and as-constructed topographic maps of the foundation area;
2. the pre-construction geologic mapping and report of the dam site area (Marliave, 1948), along with subsequent memoranda describing foundation conditions as encountered during construction by Marliave and others;
3. all pertinent exploration data from the various subsequent investigations of the dam, particularly explorations that penetrated the foundation under the dam including the 1975-1981 borings by Wahler (Wahler, 1982), the ESA exploration for design of the new intake structure (ESA, 1987), and later Phase A and B instrumentation borings drilled in 1999 and 2001 (RLVA, 1999; SCVWD, 2001); and
4. detailed geologic mapping of the dam site area by Treadwell and Rollo (2002).

We also reviewed the District's Foundation Analysis Report of SSE-2 Dams (SCVWD, 2010a). That report includes figures that depict the District's estimate of pre-construction soil thickness (colluvium, alluvium and landslide deposits) at Lenihan Dam (SCVWD 2010a, Figure 2), their estimated depths of foundation excavation (ibid, Figure 6), and their estimated distribution and thickness of in-place surficial soils remaining within the dam foundation (ibid, Figure 10). In our comparison of the pre-construction and as-built topographic surfaces (using digitized maps provided by the District), we found that the District's estimated depths of excavation as shown on Figure 6 of their report are in good agreement with the amount of excavation indicated by those maps. As noted on Figure 6 of the District's 2010 Foundation Analysis Report, the pre-

construction and as-built maps indicate areas of anomalously positive excavation values (i.e. fill, not excavation), where the as-built surface is shown to be higher than the pre-construction surface over areas of the upstream and downstream right abutment slopes. Based on their estimated depths of excavation, subtracted from their assumed thicknesses of pre-construction soils, the District identified several discrete areas of the upstream foundation area that might contain significant thicknesses of in-place soils remaining in the foundation.

Figure 6, presented herein, shows a compilation of pertinent data that we used to make our assessment of the foundation conditions under the dam. Borings that encountered bedrock are depicted along with the elevation of top of rock as encountered at each boring. Additionally, Figure 6 shows surface exposures of bedrock within, and adjacent to, the foundation as mapped by Marliave in 1948 and by Treadwell and Rollo in 2002, and includes notes that relate local geologic data to the District's estimated in-place soils for those areas. Our independent interpretation of foundation conditions at each of these areas found the following:

1. The areas of anomalously positive excavation values (and attendant estimated thicknesses of in-place soils of up to 18 feet) on the right abutment are most likely the result of a localized but pervasive as-constructed survey error over the upstream and downstream right abutment slopes. This is indicated by exploration data at borings LDP-19 and LDP-20 (Figure 6), which show the rock foundation directly under the embankment, but at elevations 18 feet to 27 feet higher than indicated on the as-built contour map. Similarly, geologic sections A-A and C-C from ESA's 1987 report indicate that their borings LO-1, LO-2 and LO-3, located on the upstream right abutment just above the original intake and the lowermost upstream embankment, were drilled into a rock slope without the overlying surficial soils that are suggested in that area on Figure 10 of the District's report (SCVWD, 2010a).
2. The central area of estimated 10-foot-thick in-place soils approximately 250 feet upstream of the axis shown on the District's Figure 10 are within an area mapped by Marliave as containing several outcrops of massive sandstone. Consequently, we conclude that this area was probably overlain by minimal thicknesses of soil rather than the approximately 20 feet of pre-construction soils estimated for this area on the District's Figure 2 (see note on Figure 6 herein).
3. The upstream left abutment areas shown on the District's Figure 10 as being underlain by 14 to 20 feet of in-place soils along the upstream toe are within an area where the embankment abuts an upstream bedrock ridge spur that is shown to be underlain by areas with surface exposures of Franciscan Complex sandstone as mapped by Treadwell and Rollo in 2002 (see Figure 6). Given the mapped rock surface exposures in this area, we conclude that this foundation area was probably blanketed by only thin colluvial soils rather than the 35 feet of pre-construction colluvium estimated for that area on the District's Figure 2.
4. A portion of the mid-left abutment area, on the left end of the upstream embankment, is shown on the District's Figure 10 as being underlain by up to 18 feet of possible in-place landslide material (the District's Figure 2 shows an estimated thickness of 45 feet of pre-construction landslide material within this area). In the various construction inspection memoranda reviewed by TGP, we did not note any reports of slide debris being left in-place

within the foundation and believe it is appropriate to think that most, if not all, slide materials were removed from the foundation area.

The results of our analysis suggest that it is unlikely that there are any significant areas of thicker, in-place soils remaining in the foundation between the embankment and the underlying Franciscan Complex bedrock. This is consistent with Marliave's concluding statement, from his 1948 geologic report, that bedrock is at, or within a few feet of, the surface within the narrow, V-shaped canyon underlying the dam site (Marliave, 1948). In summary:

1. Most of the previous exploration borings that have extended into the foundation indicate that the as-built elevation contours accurately depict the embankment foundation surface, except for areas of the right abutment where we believe a survey error mis-characterized the level of the foundation.
2. All of the borings drilled into the foundation under the dam show Franciscan Complex bedrock in direct contact with the overlying embankment.
3. The areas of estimated in-place soils depicted under the upstream portion of the embankment on the District's Figure 10 (SCVWD, 2010a) are likely the result of the above-mentioned survey error for portions of the right abutment, and an over-estimation of the pre-construction soil thickness in the central and left side areas of the upstream foundation.

As part of our Task 5 site characterization work, we will re-draw the dam foundation contours for the misrepresented right abutment areas using the available subsurface data to provide a more accurate foundation surface model for our engineering analyses.

5.0 KEY GEOTECHNICAL ISSUES IN SEISMIC STABILITY EVALUATION

As discussed in Section 4.0, Lenihan Dam has been shown to be founded on rock. As a result, there are no issues associated with the potential liquefaction of foundation soils left in place. The main issue for the seismic stability evaluation is the magnitude and pattern of permanent seismic deformations of the embankment under the design earthquake, and the implications of these seismic deformations for potential cracking of the dam and dislocation of the inclined drain. The as-built geometry, filter compatibility and permeability of the inclined drain, French drain and drainage blanket are also important issues as they relate to the potential for piping and the progression of internal erosion after damage to the dam caused by seismic deformations.

In addition to characterization of the core materials and shell materials with respect to undrained and drained modulus and shear strength parameters for monotonic loading, it is important to characterize the cyclic stress-strain properties of these materials. In particular, accumulation of cyclic strains under cyclic sub-yielding loading conditions, and accumulation of cyclic strains due to repeated cycles of loading past the yield strength of the materials, need to be considered. The cyclic triaxial tests completed in the past, combined with available data on static undrained shear strength and shear wave velocities of the various materials, are sufficient to provide a basis for making an initial assessment of these properties.

The static and dynamic engineering properties of the various materials are strongly influenced by the initial effective stresses and the degree of saturation of the materials. Consequently, analyses of steady state seepage through the dam using finite element computer programs (such as

PLAXIS or SEEPW), and comparison of estimated total heads from these finite element seepage analyses to measured total heads from piezometers, will be necessary to estimate these initial effective stresses. In addition, transient seepage analyses (i.e., consolidation analyses) may be required to better understand the measured data on the time lag of piezometric response to changing reservoir levels; PLAXIS can be used for these analyses.

The piezometer data summarized on Figure 2 show that total heads in the downstream shell (Zone 4) immediately downstream of the inclined drain (Zone 3) are at approximate Elev. 558 feet but that piezometers also located within the downstream shell, but further downstream, are dry. These measured total heads indicate that the inclined drain may not be working as effectively as would be expected. Other important observations from the piezometer data in Figure 2 are that the measured total heads in the rock foundation are relatively high beneath the core and may indicate that the Franciscan Complex bedrock is more permeable than the lower core. A preliminary estimate of the phreatic surface (top flow line) is shown on Figure 2. Finite element seepage analyses will be made in order to provide analytical estimates of the location of the phreatic surface and the distribution of total heads within the embankment. Parametric seepage analyses will be necessary to evaluate what embankment, bedrock and drain permeabilities are consistent with the measured values of total head.

6.0 PROPERTIES OF EMBANKMENT MATERIALS FROM CPT DATA

Cone Penetrometer Tests can provide very valuable data that can be used as an indication of engineering properties; and can also provide valuable information on the variability of the engineering properties from one zone of the dam to another, and from one station to another. The engineering properties inferred from CPTs should be compared to the engineering properties measured using other more direct laboratory and/or field measurements at the site to allow calibration and adjustment of the correlations used to estimate the engineering properties using CPT data. Perhaps the most important insights gained from the CPT data are those related to the similarities or differences in the inferred engineering properties from one zone of the dam to another, or spatially within a given zone of the dam.

The Lenihan Dam Outlet Investigation by RLVA (1999) included four CPTs at the locations shown on Figure 1B. As shown on Figure 1B, CPT-1, CPT-2 and CPT-4 were completed along the alignment of the outlet pipeline in the vicinity of Section B-B' on the upstream slope, dam crest, and downstream slope, respectively. CPT-3 was completed on the upstream slope in the vicinity of Section A-A', as also shown on Figure 1B. RLVA analyzed the CPT data from these four CPT probes and estimated engineering properties of the embankment soils using relationships presented by Robertson and Campanella (1990). These relationships have been refined in recent years as described by Robertson (2009). This section discusses some of the inferred properties of the soils at the site based on the results of these CPT test data using the more recent correlations by Robertson (2009).

Figure 7 shows the point resistance, q_c , and Friction Ratio data for the probes made in the vicinity of Section B-B'. CPT-4 is located at Section B-B' and CPT-1, CPT-2, and CPT-3 are projected to this section while keeping the offset from the dam centerline constant. As discussed in Section 2.2, the upstream and downstream shells of the dam and the upper portion of the core

were all constructed using materials excavated from the Franciscan Complex that were processed, placed in lifts, and compacted. As shown in Table 1 from Section 2.2, the material from the Franciscan Complex typically has 28% to 35% finer than the #200 sieve and average gravel contents that range from 23% for the upper core to 41% for the downstream shell. The Atterberg Limits of the fine grained soils indicate they are classified as clays of low plasticity (CL). The lower portion of the core was constructed using clayey alluvial soils and the data on this material shows that 95% passes the #200 sieve and there is only 1% gravel; and the Atterberg Limits of the fine grained soils indicate they are classified as clays of high plasticity (CH).

Figure 8 shows the results of the CPT test data on the soil behavior type (SBT) plot that was developed by Robertson and his colleagues 25 years ago and is now commonly used to estimate the soil types associated with CPT test data. The normalized CPT data from tests on the lower core material cluster closely together primarily in the right hand portion of zone 3 of the SBT plot indicating these soils are relatively homogeneous overconsolidated clays. These materials are known to be compacted CH clays with essentially no sand or gravel and match the soil behavior type indicated by Figure 8. The normalized CPT data from the zones of the downstream shell, upstream shell and upper core are concentrated in zone 9 of the SBT chart indicating they are very stiff fine grained soils but there is clearly more scatter in the data. This is also consistent with the fine grained matrix of the soils derived from the Franciscan Complex and the fact that there are considerable amounts of gravel in these materials. The CPT data also indicate the soils in the various embankment zones have relatively consistent properties from one CPT probe to another.

Figure 9 shows the inferred undrained shear strength from the CPT data. Because of the clay matrix found in all the embankment zones, we decided to treat all these materials as clays for purposes of estimating undrained shear strength. As recommended by Robertson (2009), the inferred strengths were calculated by subtracting the vertical total stress at each CPT test location from the measured cone tip resistance for that test location, and then dividing by 14. Inspection of the inferred undrained strength data shown in Figure 9 indicates that the undrained strengths in the lower core at CPT-1 and CPT-2 are relatively uniform and have a typical value of 3 tons/ft² (tsf). The strength data for the upper core and the upstream shell are also similar; they are somewhat stronger than the lower core, with undrained strengths that range from 2 to 6 tsf and have typical values of about 4 tsf. The downstream shell material was tested at CPT-4 and shows undrained strengths that range from 3 to 10 tsf and have typical values of about 6 tsf. The materials below the inclined drain at CPT-4 have undrained strengths that are similar to those observed for the upper core and upstream shell. This suggests that the materials beneath the inclined drainage blanket at this location are likely from the Franciscan Complex rather than from the alluvial clays used elsewhere for the lower core of the dam.

Thus, our initial review of these CPT data provides important insights into the undrained shear strength of the materials in the various zones of the embankment and the spatial variation of the undrained shear strength within a given zone. These data are a good example of the quality of the geotechnical information already available at Lenihan Dam and the value that can be derived from it.

7.0 CONCLUSIONS AND RECOMMENDATIONS

As stated in Section 1.0, we have concluded the following:

1. Our detailed review of the data strongly indicates that no soil was left in place beneath the dam. This is a very significant and positive finding from a dam safety perspective because it means that potential liquefaction of soils left in place beneath the dam is not an issue. The finding that no soils were left in place beneath the dam is consistent with the assumption made by DSOD during their design reviews completed in 2006 and 2007 (DSOD, 2006c and 2007a).
2. There appears to be sufficient information available to define the geometry of the dam and of its foundation.
3. Because of the enormous amount of available geotechnical data, a detailed review and thorough evaluation of the available data on the properties of the various zones of the embankment (as indicated by the construction records, field investigations and laboratory tests completed to date) may provide most of the information necessary to support the engineering analyses.

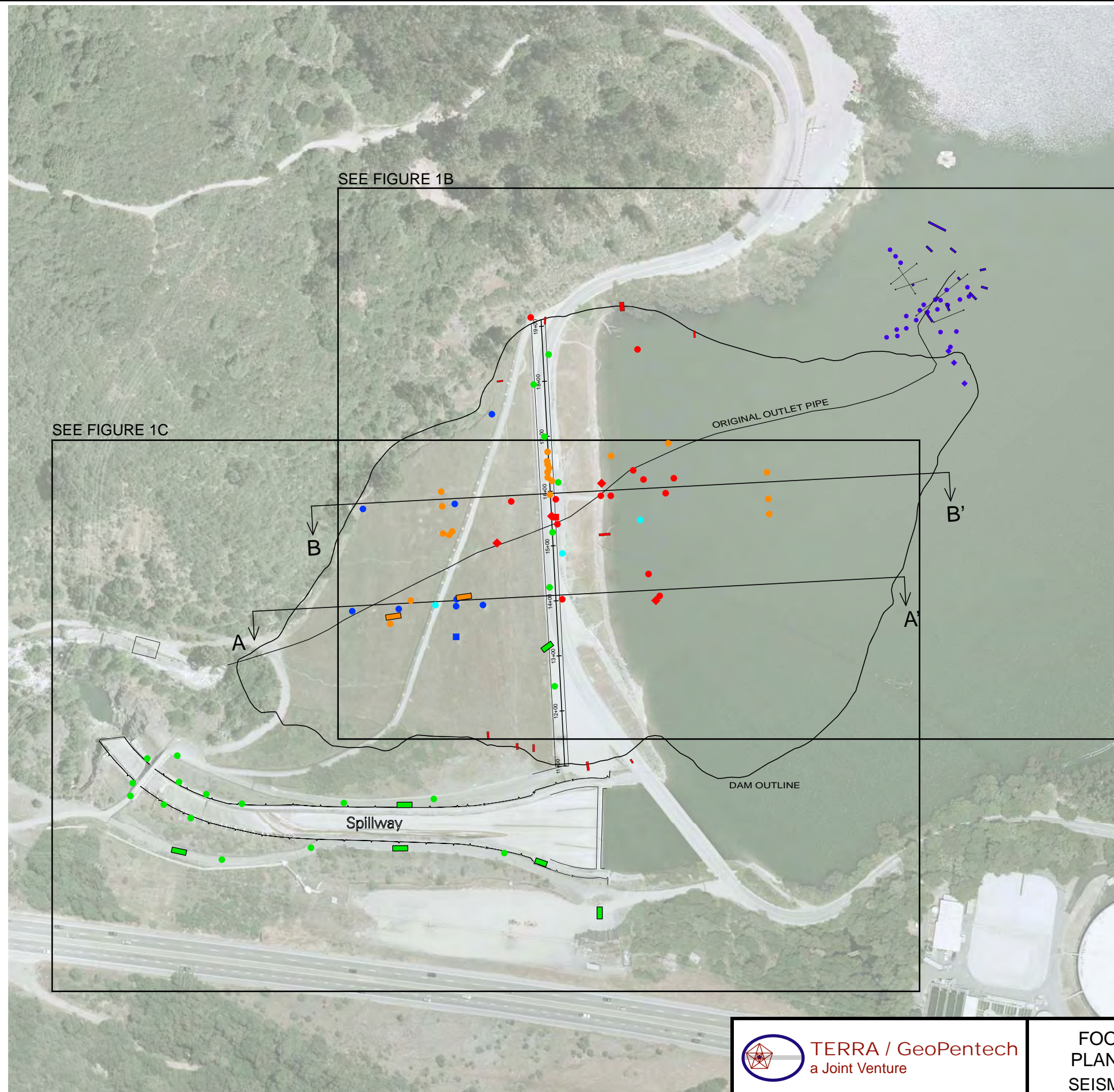
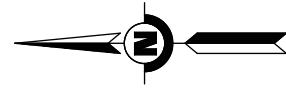
Consequently, we recommend that our approach be adjusted so that we complete the majority of Task 5 – Geotechnical Site Characterization based on the detailed review, evaluation and interpretation of the wealth of available data; complete some of the engineering analyses based on the available data; and then decide what supplemental field and laboratory data are necessary to reduce the uncertainties in the results of the seismic stability analyses and prepare a work plan to obtain these data. This work plan, if necessary, will be submitted to DSOD following review and input from the District.

8.0 REFERENCES

- AMEC, 2009 (January), Technical Memorandum No. 3, Seismotectonic and Ground Motion Study, Seismic Stability Evaluation of DIP Phase 1 Dams.
- California Department of Water Resources, 1948-1954, Geological and Engineering Construction Inspection Memoranda to Mr. H.W. Holmes, Lexington Dam, by E.C. Marliave, D.F. Dresselhaus, et al (on file at DSOD).
- California Department of Water Resources, Division of Safety of Dams, 1981 (July), Phase 1 Inspection Report for Lexington Dam.
- California Department of Water Resources, Division of Safety of Dams, 2006a (May), Memorandum of Geologic Review of Seismic Hazard, James J. Lenihan Dam, dated May 25, 2006.
- California Department of Water Resources, Division of Safety of Dams, 2006b (October), Geologic Review of Geotechnical Data Report and Lexington Fault Evaluation Memo, dated October 23, 2006.

- California Department of Water Resources, Division of Safety of Dams, 2006c (October), Memorandum of Design Review – Review of Performance During the Loma Prieta Earthquake, James J. Lenihan Dam, dated October 23, 2006.
- California Department of Water Resources, Division of Safety of Dams, 2007a (June), Memorandum of Design Review, James J. Lenihan Dam, dated June 29, 2007.
- California Department of Water Resources, Division of Safety of Dams, 2007b (November), Geologic Inspection of Lexington Fault Exposure and Review of Lexington Fault Cove Exposure Memo, dated November 26, 2007.
- California Department of Water Resources, Division of Safety of Dams, 2009 (October), Updated Ground Motion Estimates, J. Lenihan Dam, dated October 15, 2009.
- California Department of Water Resources, Division of Safety of Dams, 2010 (June), DSOD Comments on DM-2 and Interim DM-4, Guadalupe, Almaden, and Calero Dams, dated June 10, 2010.
- Cotton and Associates, 1993, Pre-Design Investigation, Lexington Dam Freeboard Restoration Project, Santa Clara Valley Water District.
- Geomatrix Consultants, 1992, Analysis of the Recorded Response of Lexington Dam During the Loma Prieta Earthquake.
- Geomatrix Consultants, 1996, Geologic/Geotechnical Study and Basis of Design Report, Lexington Dam Freeboard Restoration Project.
- Geomatrix Consultants, 2006a (July), Evaluation of Lexington Fault near Lenihan Dam, Memorandum to Mr. Nolting - Jacobs Associates, Lenihan Dam Outlet Modification Project, Memorandum, dated July 14, 2006.
- Geomatrix Consultants, 2006b, Final Geologic and Geotechnical Data Report, Lenihan Dam Outlet Modification Project.
- Geomatrix Consultants, 2007 (October), Field Review of Lexington Fault Exposure in “The Cove”, Memorandum to Mr. Nolting, Jacobs Associates, in support of Lenihan Dam Outlet Modification Project, dated October 12, 2007.
- Marliave, E.C., 1948, Geological Report on the Lexington Dam Site.
- McLaughlin et al, 2004, Geologic Map of the Loma Prieta Region, California, USGS Professional Paper 1550E.
- Robertson, P.K. and Campenella, R.G., 1990, Guidelines for Use, Interpretation and Application of CPT and CPTU, University of British Columbia, Soil Mechanics Series 105, Civil Engineering Department, Vancouver, B.C., V6T 1W5, Canada.
- Robertson, P.K., 2009, Interpretation of Cone Penetrometer Tests – a unified approach, Canadian Geotechnical Journal, Volume 46, Number 11, 1 November 2009, pp. 1337-1355.
- Santa Clara Valley Water District, 1976, Reconnaissance of Landslide Conditions at Lexington Reservoir.

- Santa Clara Valley Water District, 1996, Specifications and Contract Documents for the Construction of Lexington Dam Freeboard Restoration Project.
- Santa Clara Valley Water District, 2001 (October), Phase B Instrumentation Project Basic Data Report.
- Santa Clara Valley Water District, 2007 (July), Second Summary Surveillance Report, December 2003 through July 2007, Lenihan Dam No.72-8.
- Santa Clara Valley Water District, 2010a (January), Foundation Analysis Report on Chesbro No.72-11, Lenihan No.72-8, Stevens Creek No. 72-7, and Uvas No. 72-12 Dams (SSE-2 Dams), prepared by James L. Nelson.
- Santa Clara Valley Water District, 2010b (June), 2010 Surveillance Report, March 2009 through April 2010, Lenihan Dam No.72-8.
- Treadwell and Rollo, 2002, Geotechnical Feasibility Report for Lenihan Dam New Tunnel Option.
- Volpe, R.L. and Associates, 1990, Investigation of SCVWD Dams Affected by Loma Prieta Earthquake of October 17, 1989, Report to Santa Clara Valley Water District.
- Volpe, R.L. and Associates, 1997, Evaluation of Partial Collapse of Low Level Outlet Pipeline, James J. Lenihan Dam.
- Volpe, R.L. and Associates, 1998, Instrumentation Design and Review, Lenihan Dam.
- Volpe, R.L. and Associates, 1999a, Lenihan Dam Outlet Investigation, Vol. 1 – Final Engineering Report.
- Volpe, R.L. and Associates, 1999b, Lenihan Dam Outlet Investigation, Vol. 2 – Basic Data Report.
- Wahler Associates, 1982, Final Report on Seismic Safety Evaluation of Lexington Dam.
- Wahler Associates, 1993, Investigation of Seepage on the Downstream Face of Lexington Dam.
- Woodward-Clyde Consultants, 1993, Nonlinear Dynamic Response Analysis of Lexington Dam.



LEGEND

BORINGS / PIEZOMETERS / INCLINOMETERS AND CPTs

- WAHLER 1982 BORING
- ESA 1987 BORING
- ◆ ESA 1987 CPT
- GEOMATRIX 1996 BORING
- HARZA 1997 BORINGS
- VOLPE 1999 PIEZOMETER
- VOLPE 1999 INCLINOMETER
- ◆ VOLPE 1999 CPT
- DISTRICT 2001 PIEZOMETER
- DISTRICT 2001 INCLINOMETER

OTHER EXPLORATIONS

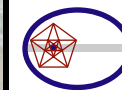
- WAHLER 1982 TEST PIT
- ESA 1987 TEST PIT
- ESA 1987 SEISMIC REFRACTION LINE
- VOLPE 1990 TRENCH
- GEOMATRIX 1996 TEST PIT

APPROXIMATE SCALE, FEET



NOTES:

- 1) SEE FIGURES 1B AND 1C FOR ADDITIONAL DETAILS.
- 2) INVESTIGATIONS FOR NEW OUTLET TUNNEL IN RIGHT ABUTMENT NOT SHOWN.





LEGEND

BORINGS / PIEZOMETERS / INCLINOMETERS AND CPTs

- WAHLER 1982 BORINGS
- ESA 1987 BORINGS
- ◆ ESA 1987 CPT
- GEOMATRIX 1996 BORING
- HARZA 1997 BORING
- VOLPE 1999 PIEZOMETERS
- VOLPE 1999 INCLINOMETER
- ◆ VOLPE 1999 CPT
- DISTRICT 2001 PIEZOMETERS
- DISTRICT 2001 INCLINOMETER

LARGE SYMBOLS ARE FOR BORINGS INTO ROCK

OTHER EXPLORATIONS

- ▭ WAHLER 1982 TEST PIT
- ▭ ESA 1987 TEST PIT
- ESA 1987 SEISMIC REFRACTION LINE
- ▭ VOLPE 1990 TRENCH
- ▭ GEOMATRIX 1996 TEST PIT

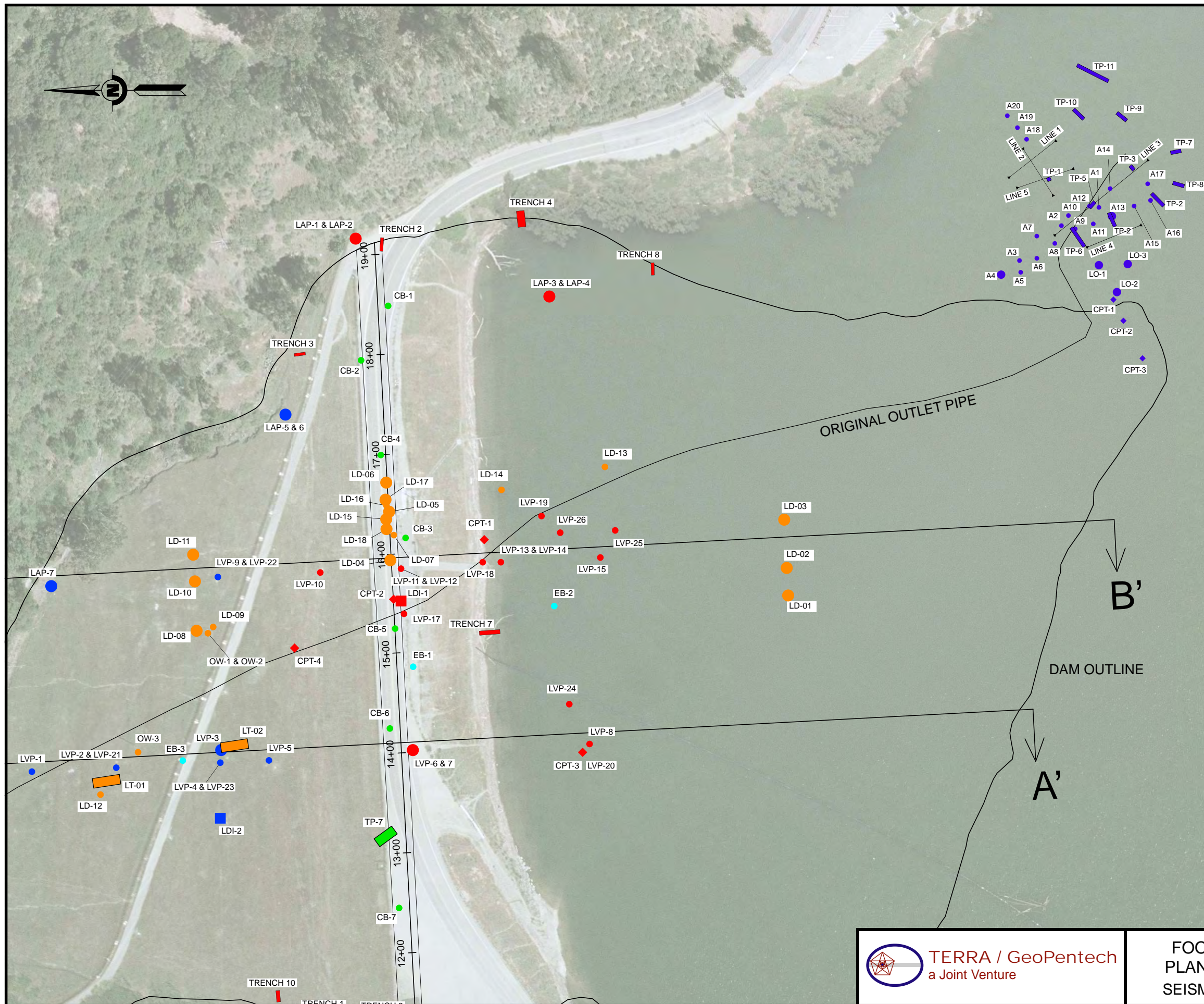
APPROXIMATE SCALE, FEET



NOTES:

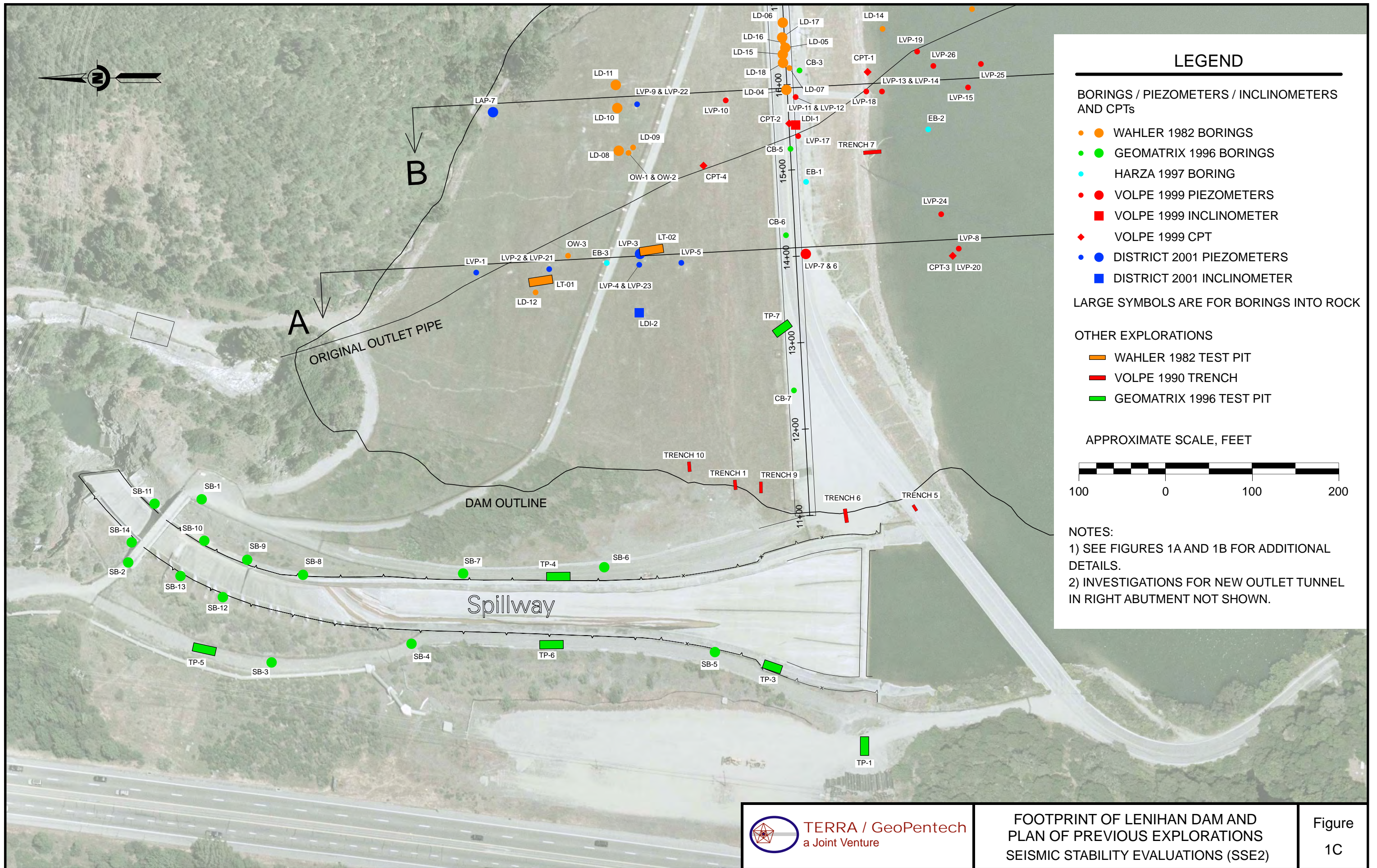
- 1) SEE FIGURES 1A AND 1C FOR ADDITIONAL DETAILS.
- 2) INVESTIGATIONS FOR NEW OUTLET TUNNEL IN RIGHT ABUTMENT NOT SHOWN.

Rev. 1 09/09/2010 SSE2-TM-1LN



FOOTPRINT OF LENIHAN DAM AND PLAN OF PREVIOUS EXPLORATIONS SEISMIC STABILITY EVALUATIONS (SSE2)

Figure 1B



LEGEND

BORINGS / PIEZOMETERS / INCLINOMETERS AND CPTs

- WAHLER 1982 BORINGS
- GEOMATRIX 1996 BORINGS
- HARZA 1997 BORING
- VOLPE 1999 PIEZOMETERS
- VOLPE 1999 INCLINOMETER
- ◆ VOLPE 1999 CPT
- DISTRICT 2001 PIEZOMETERS
- DISTRICT 2001 INCLINOMETER

LARGE SYMBOLS ARE FOR BORINGS INTO ROCK

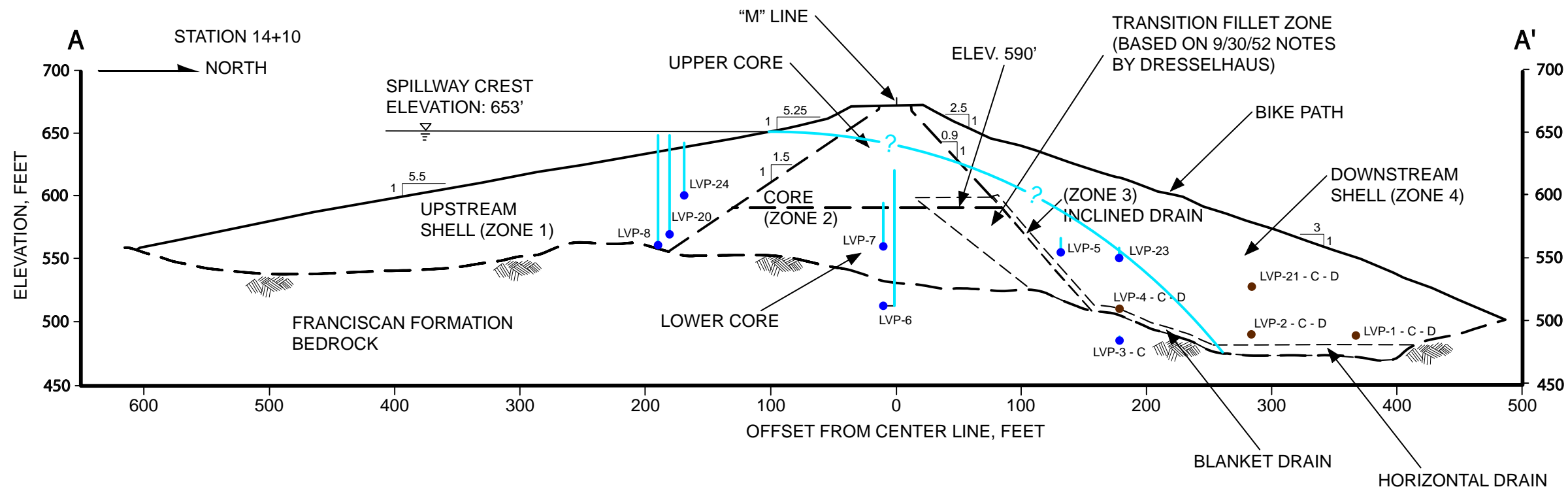
OTHER EXPLORATIONS

- ▭ WAHLER 1982 TEST PIT
- ▭ VOLPE 1990 TRENCH
- ▭ GEOMATRIX 1996 TEST PIT

APPROXIMATE SCALE, FEET



NOTES:
 1) SEE FIGURES 1A AND 1B FOR ADDITIONAL DETAILS.
 2) INVESTIGATIONS FOR NEW OUTLET TUNNEL IN RIGHT ABUTMENT NOT SHOWN.



LEGEND

LOCATION AND NUMBER OF PIEZOMETER TIP

NOTES

ELEVATION OF THE TOP OF BLUE BAR REPRESENTS MEASURED TOTAL HEAD AT FULL RESERVOIR LEVEL.

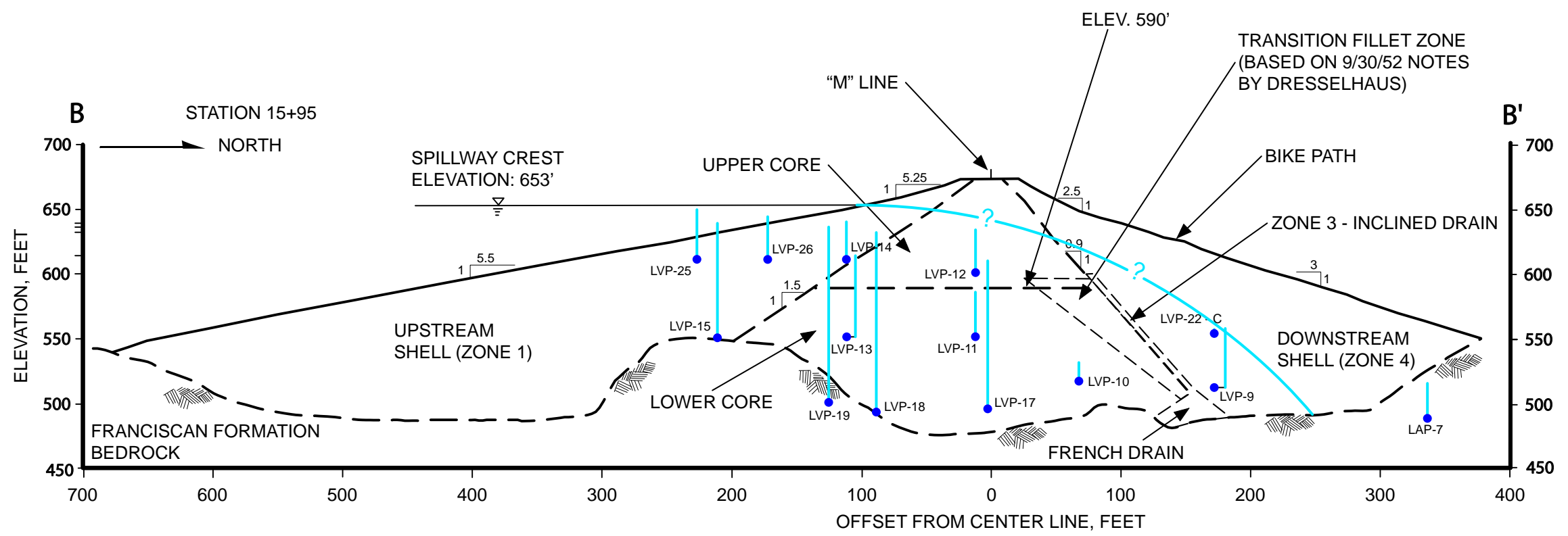
HEIGHT OF BLUE BAR REPRESENTS MEASURED PRESSURE HEAD.

C INDICATES TOTAL HEAD IS INDEPENDENT OF RESERVOIR LEVEL.

D INDICATES THE PIEZOMETER TIP IS DRY. A BROWN TIP IS SHOWN AT THESE LOCATIONS.

TOP FLOW LINE SHOWN IS APPROXIMATE, PARTICULARLY IN VICINITY OF CHIMNEY DRAIN, AND IS AT THE SAME LOCATION ON BOTH SECTIONS.

LVP-19 PROJECTED 30 FT WEST AND IS SCREENED IN EMBANKMENT



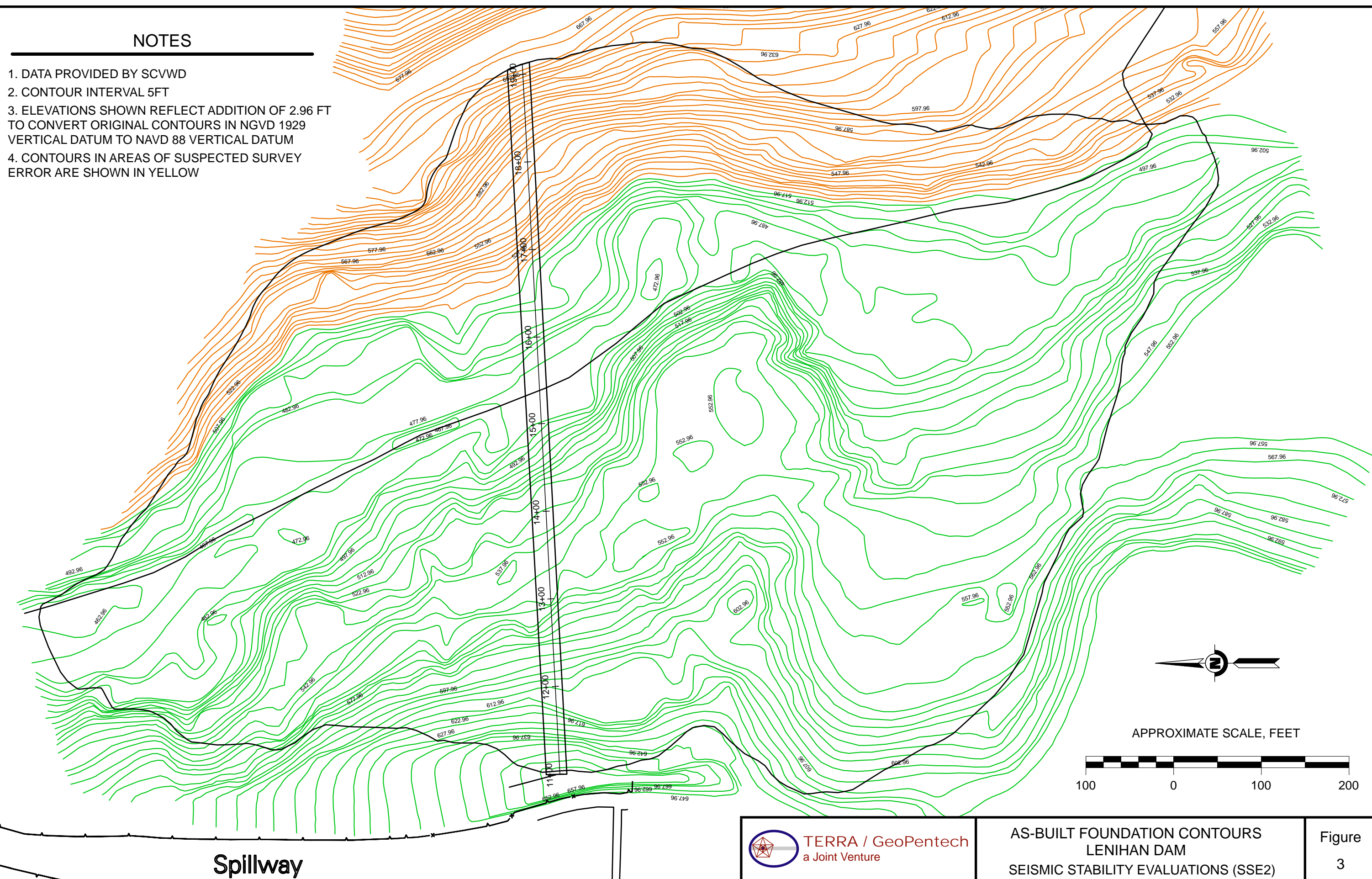
Rev. 1 09/09/2010 SSE2-TM-1LN



CROSS SECTIONS AND MEASURED PIEZOMETRIC LEVELS - LENIHAN DAM SEISMIC STABILITY EVALUATIONS (SSE2)

NOTES

- 1. DATA PROVIDED BY SCVWD
- 2. CONTOUR INTERVAL 5FT
- 3. ELEVATIONS SHOWN REFLECT ADDITION OF 2.96 FT TO CONVERT ORIGINAL CONTOURS IN NGVD 1929 VERTICAL DATUM TO NAVD 88 VERTICAL DATUM
- 4. CONTOURS IN AREAS OF SUSPECTED SURVEY ERROR ARE SHOWN IN YELLOW



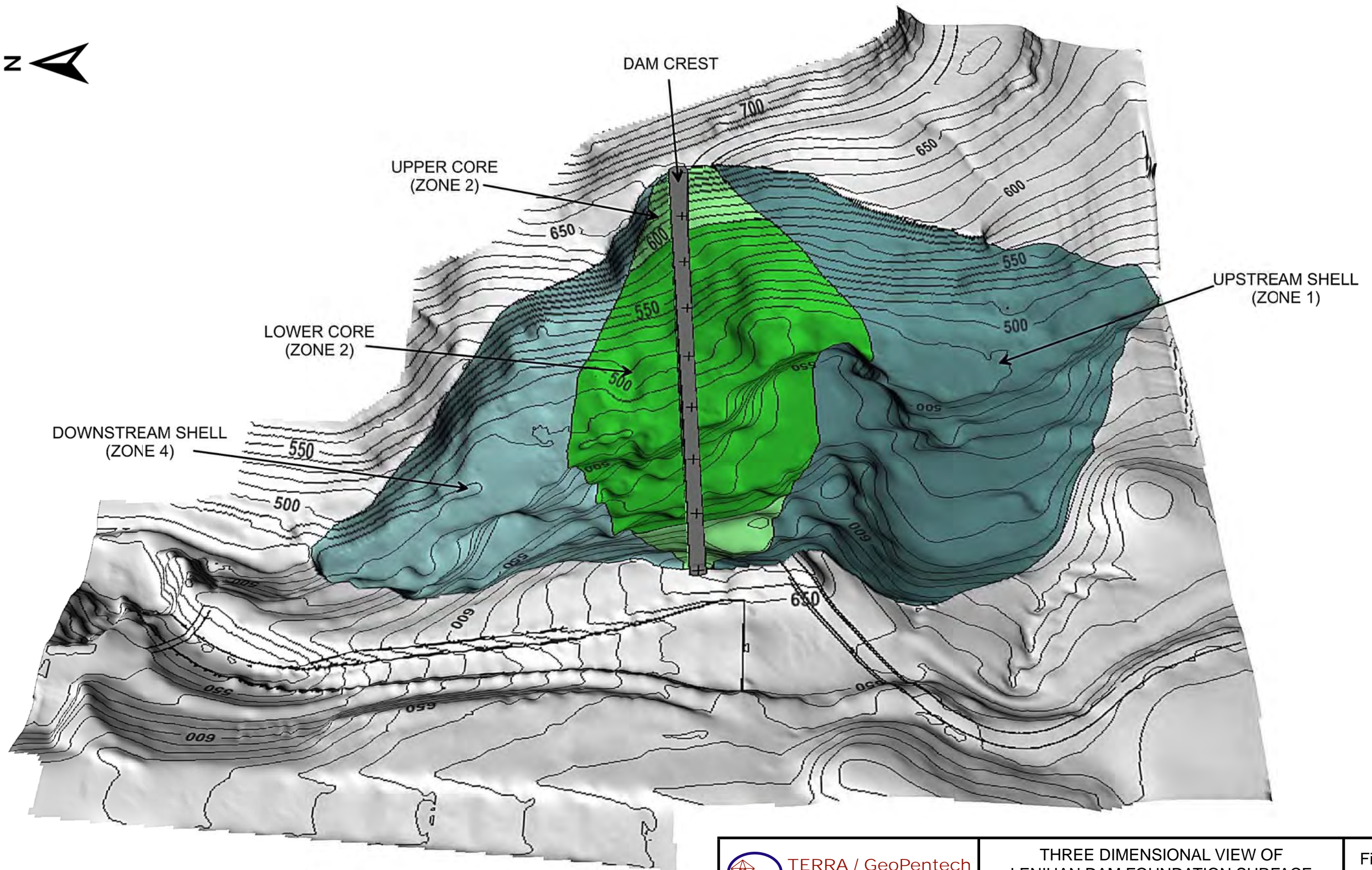
Rev. 0 08/16/2010 SSE2-TM-1LN



AS-BUILT FOUNDATION CONTOURS
LENIHAN DAM
SEISMIC STABILITY EVALUATIONS (SSE2)

Figure
3

0' 100' 200' 300' 400'



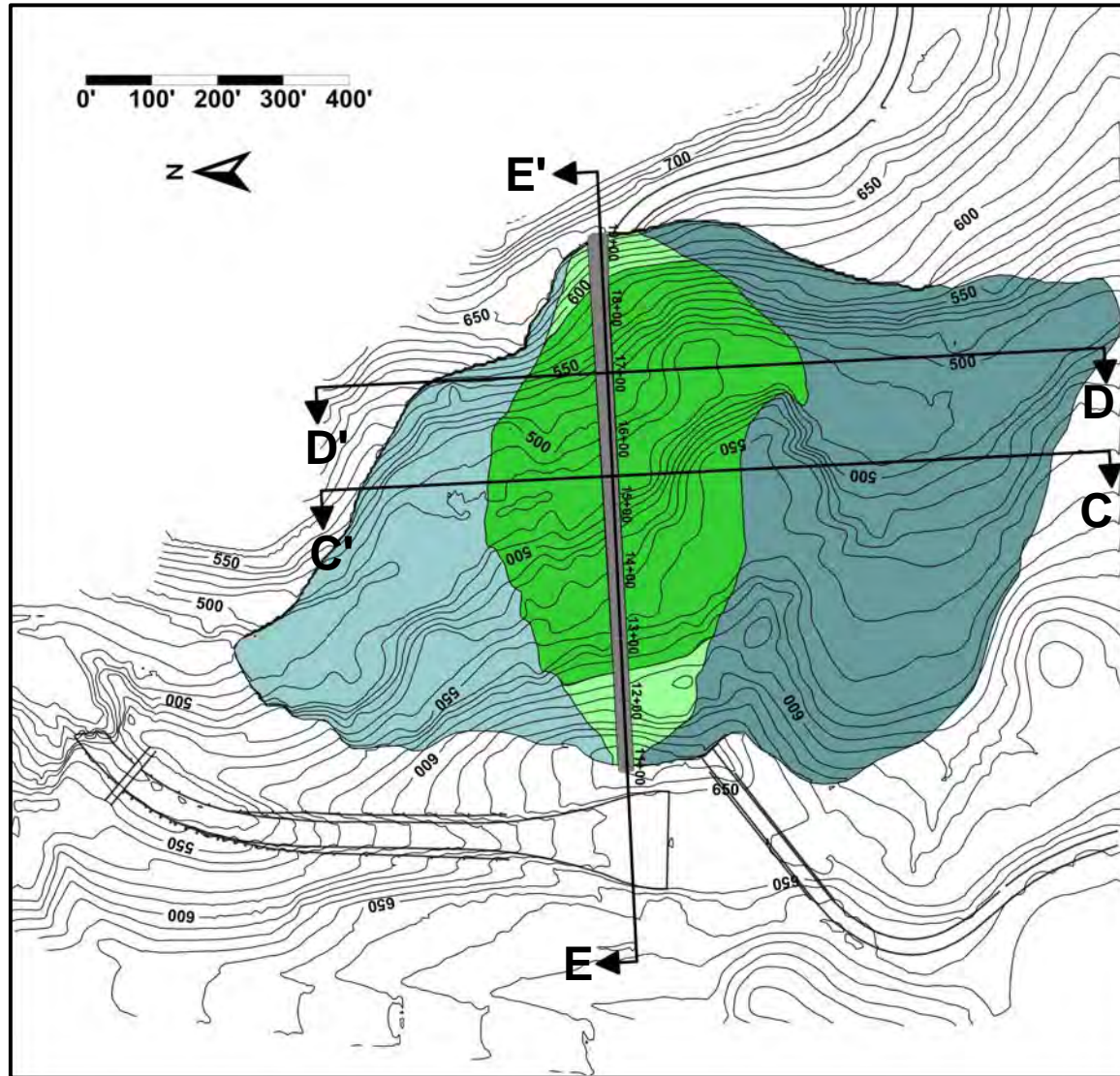
Note:
1. Contacts of various dam zones with foundation surface are shown.



THREE DIMENSIONAL VIEW OF
LENIHAN DAM FOUNDATION SURFACE
SEISMIC STABILITY EVALUATIONS (SSE2)

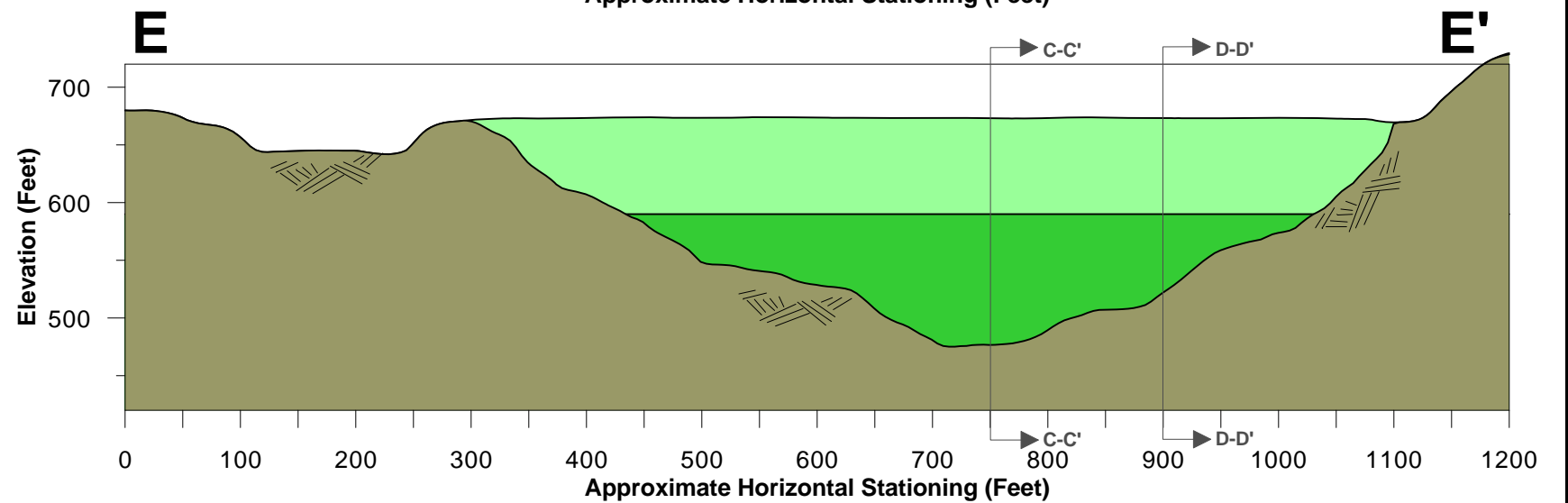
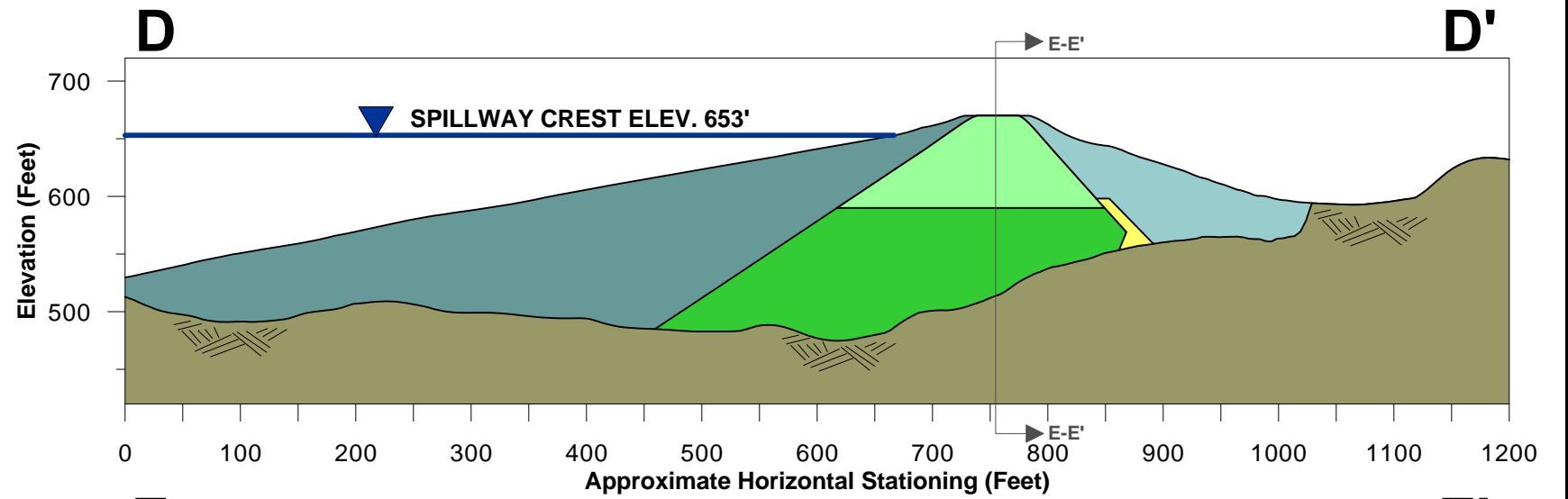
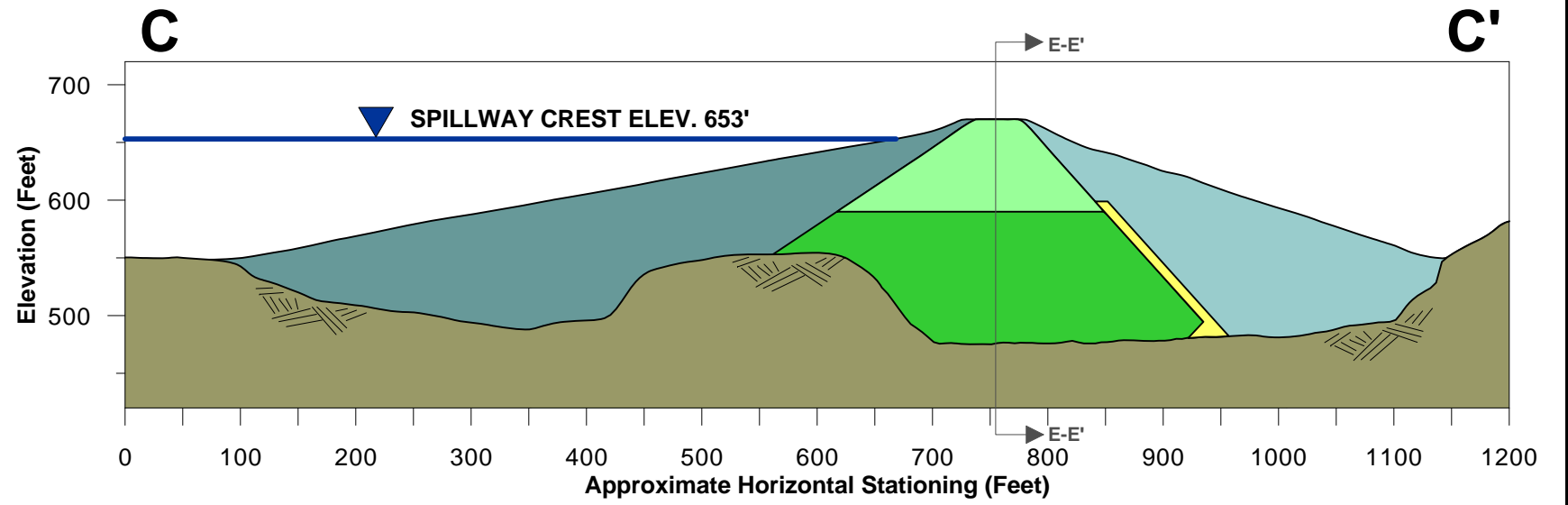
Figure
4

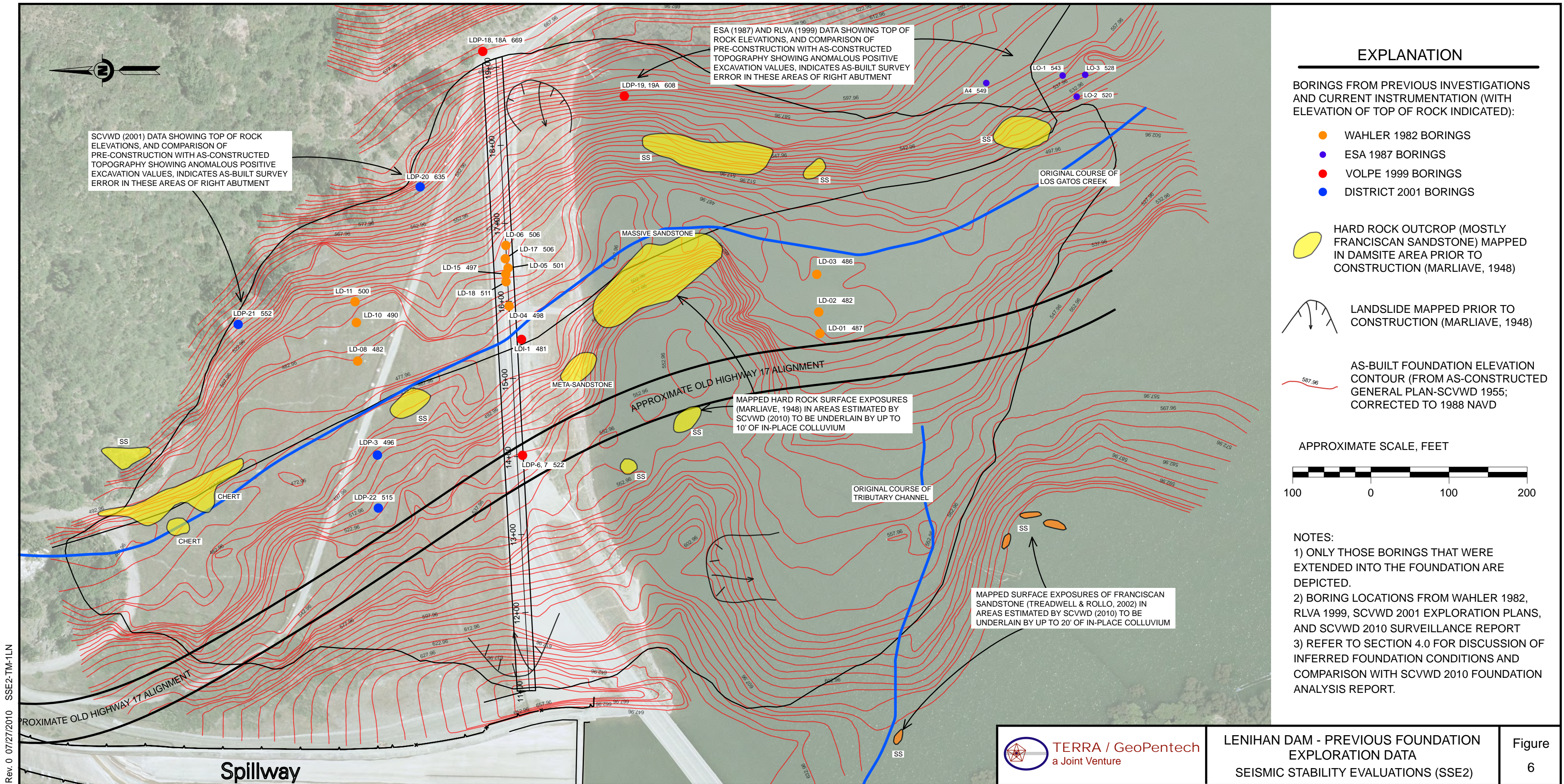
CROSS SECTION LOCATION MAP



LEGEND

- Upstream Shell (Zone 1)
- Lower Core (Zone 2, Below El. 590)
- Upper Core (Zone 2, Above El. 590)
- Drain Zone (Zone 3)
- Downstream Shell (Zone 4)
- Franciscan Complex Bedrock





EXPLANATION

BORINGS FROM PREVIOUS INVESTIGATIONS AND CURRENT INSTRUMENTATION (WITH ELEVATION OF TOP OF ROCK INDICATED):

- WAHLER 1982 BORINGS
- ESA 1987 BORINGS
- VOLPE 1999 BORINGS
- DISTRICT 2001 BORINGS

● HARD ROCK OUTCROP (MOSTLY FRANCISCAN SANDSTONE) MAPPED IN DAMSITE AREA PRIOR TO CONSTRUCTION (MARLIAVE, 1948)

⤴ LANDSLIDE MAPPED PRIOR TO CONSTRUCTION (MARLIAVE, 1948)

— AS-BUILT FOUNDATION ELEVATION CONTOUR (FROM AS-CONSTRUCTED GENERAL PLAN-SCVWD 1955; CORRECTED TO 1988 NAVD)

APPROXIMATE SCALE, FEET



- NOTES:
- 1) ONLY THOSE BORINGS THAT WERE EXTENDED INTO THE FOUNDATION ARE DEPICTED.
 - 2) BORING LOCATIONS FROM WAHLER 1982, RLVA 1999, SCVWD 2001 EXPLORATION PLANS, AND SCVWD 2010 SURVEILLANCE REPORT
 - 3) REFER TO SECTION 4.0 FOR DISCUSSION OF INFERRED FOUNDATION CONDITIONS AND COMPARISON WITH SCVWD 2010 FOUNDATION ANALYSIS REPORT.

SCVWD (2001) DATA SHOWING TOP OF ROCK ELEVATIONS, AND COMPARISON OF PRE-CONSTRUCTION WITH AS-CONSTRUCTED TOPOGRAPHY SHOWING ANOMALOUS POSITIVE EXCAVATION VALUES, INDICATES AS-BUILT SURVEY ERROR IN THESE AREAS OF RIGHT ABUTMENT

ESA (1987) AND RLVA (1999) DATA SHOWING TOP OF ROCK ELEVATIONS, AND COMPARISON OF PRE-CONSTRUCTION WITH AS-CONSTRUCTED TOPOGRAPHY SHOWING ANOMALOUS POSITIVE EXCAVATION VALUES, INDICATES AS-BUILT SURVEY ERROR IN THESE AREAS OF RIGHT ABUTMENT

MAPPED HARD ROCK SURFACE EXPOSURES (MARLIAVE, 1948) IN AREAS ESTIMATED BY SCVWD (2010) TO BE UNDERLAIN BY UP TO 10' OF IN-PLACE COLLUVIUM

MAPPED SURFACE EXPOSURES OF FRANCISCAN SANDSTONE (TREADWELL & ROLLO, 2002) IN AREAS ESTIMATED BY SCVWD (2010) TO BE UNDERLAIN BY UP TO 20' OF IN-PLACE COLLUVIUM

Rev. 0 07/27/2010 SSE2-TM-1LN

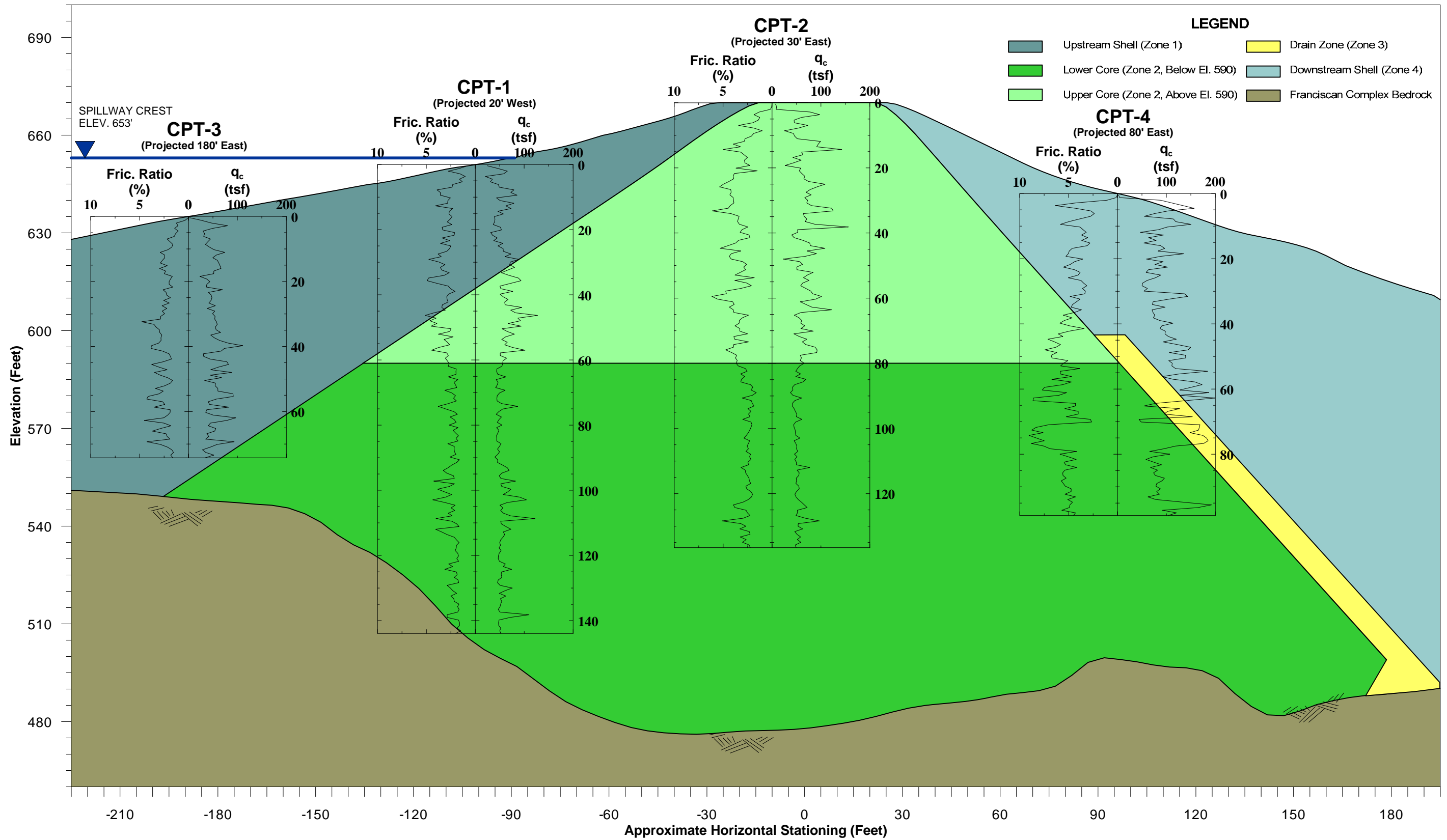


LENIHAN DAM - PREVIOUS FOUNDATION EXPLORATION DATA
SEISMIC STABILITY EVALUATIONS (SSE2)


Figure 6

B

B'

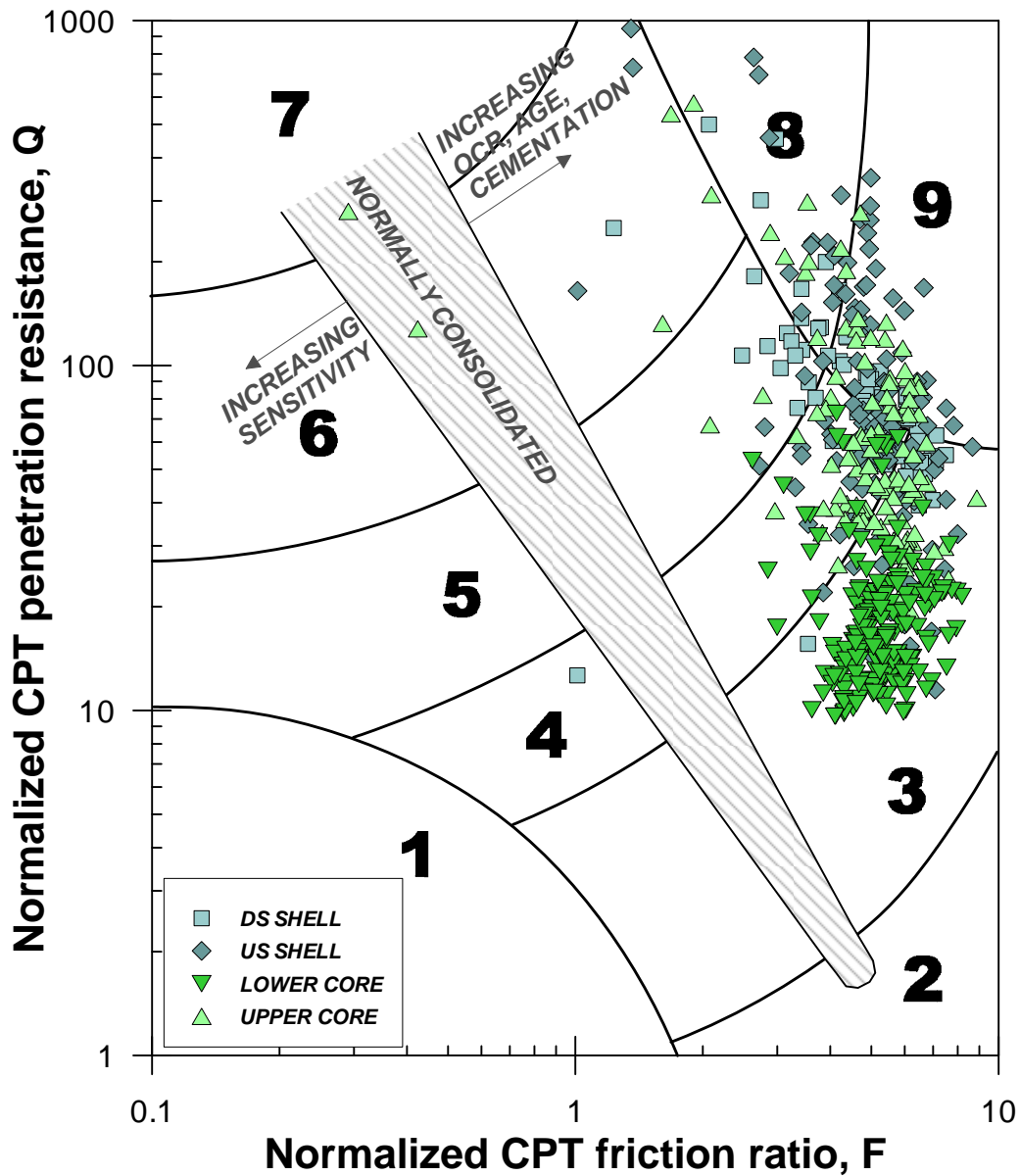


Note: See Figure 2 for complete zoning, piezometric data and location of section B-B'

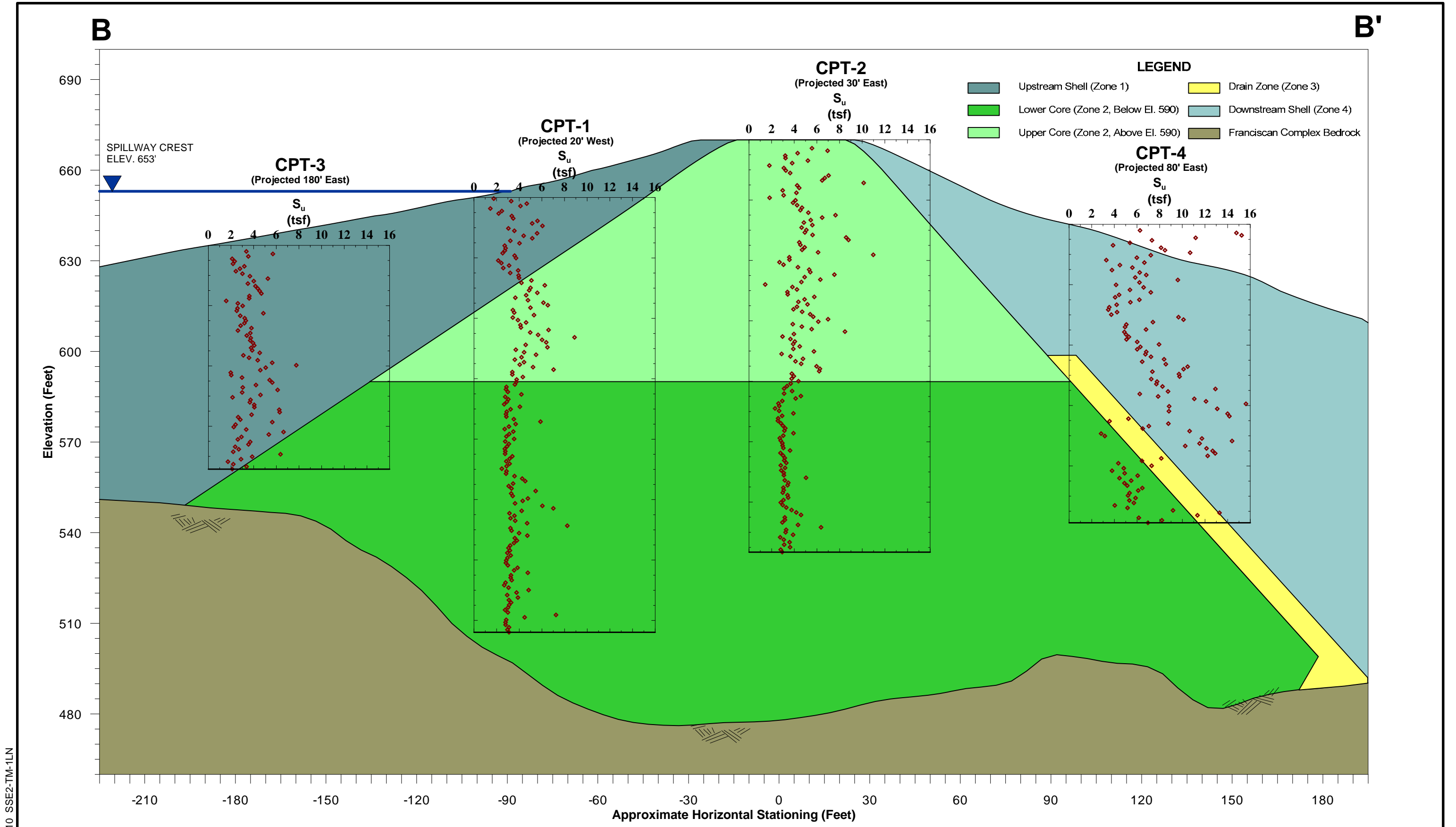
	<p>CROSS SECTION B-B' WITH CPT DATA PLOTS SEISMIC STABILITY EVALUATIONS (SSE2)</p>	<p>Figure 7</p>
---	--	---------------------

Zone	Soil Behavior Type	I_c
1	Sensitive, fine grained	N/A
2	Organic soils – peats	> 3.6
3	Clays – silty clay to clay	2.95 – 3.6
4	Silt mixtures – clayey silt to silty clay	2.60 – 2.95
5	Sand mixtures – silty sand to sandy silt	2.05 – 2.6
6	Sands – clean sand to silty sand	1.31 – 2.05
7	Gravelly sand to dense sand	< 1.31
8	Very stiff sand to clayey sand*	N/A
9	Very stiff, fine grained*	N/A

* Heavily overconsolidated or cemented



Notes: Soil Behavior Type Index Chart based on Robertson (2009)



Notes: 1. See Figure 2 for complete zoning, piezometric data and location of section B-B'
 2. Inferred Undrained Shear Strength, S_u based on Robertson (2009)



CROSS SECTION B-B'
 WITH INFERED UNDRAINED STRNGTH
 SEISMIC STABILITY EVALUATIONS (SSE2)

Figure
 9