APPENDIX 3-A

Existing Studies and Data

FINAL REPORT

INVESTIGATION AND ANALYSIS OF SANTA ANITA DEBRIS DAM LOS ANGELES COUNTY, CALIFORNIA

Prepared for
Department of Public Works
County of Los Angeles
900 South Fremont Avenue
Alhambra, California 91803-1331

August 1994 (Rev. 1)

Woodward-Clyde

Woodward-Clyde Consultants
2020 East First Street, Suite 400
Santa Ana, California 92705

Project No. 904E151A

Woodward-Clyde

Engineering & sciences applied to the earth & its environment

August 3, 1994
Project No. 904E151A

Mr. Harry W. Stone, Acting Director
County of Los Angeles
Department of Public Works
900 S. Fremont Avenue
Alhambra, California 91803-1331

Attention: Mr. Robert J. Kroll
Hydraulic/Water Conservation Division

SUBJECT: FINAL REPORT
INVESTIGATION AND ANALYSIS OF
SANTA ANITA DEBRIS DAM
LOS ANGELES COUNTY, CALIFORNIA

Gentlemen:

In accordance with the terms of Agreement No. 64174, dated November 27, 1990, between Woodward-Clyde Consultants (Woodward-Clyde) and the Los Angeles County Department of Public Works (LACDPW), including Supplements 1, 2, and 3, dated June 25, 1991, May 19, 1993, and February 15, 1994, transmitted herewith is our revised final report on the seismic evaluation of Santa Anita Debris Dam.

Most of the information provided in this report was extracted from the May 1992 Interim Report for the project. The report also incorporates responses to comments by the LACDPW on our draft report, dated December 1993. Finally, this report presents the results of our evaluation of the stability and deformation of the downstream slope of the dam embankment, and our modified analysis of the stability of the outlet tower. Both of these analyses were performed in response to the June 12, 1993 letter from the State Division of Safety of Dams (DSOD) regarding the Interim Report.

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We appreciate having had the opportunity to work on this interesting project. If you have any questions or comments regarding our conclusions or recommendations, please do not hesitate to call.

Very truly yours,

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4.0
CONCLUSIONS
AND RECOMMENDATIONS

4.1 CONCLUSIONS

The conclusions from the investigation and analysis of Santa Anita Debris Dam are summarized as follows:

- The postulated design earthquakes for the Santa Anita Debris Dam site are a moment magnitude 8 event originating on the San Andreas Fault, a distance of about 33 kilometers from the site, and a moment magnitude 7 event originating on the Sierra Madre Fault, a distance of less than 0.5 km from the site. The estimated horizontal accelerations corresponding to these two earthquake events are 0.35g and 0.8g, respectively.

- The potential for fault rupture intersecting either the dam embankment, the spillway structure, or the outlet works appears unlikely.

- The potential for liquefaction of the alluvial foundation materials at the dam site is low.

- There is a potential for liquefaction-induced deformations of the dam embankment during a seismic event. However, the potential deformations of the embankment are estimated to be less than 1 foot, and are not likely to cause a catastrophic failure of the embankment.

- There is a potential for differential settlement of the spillway slab due to liquefaction-induced settlement of the underlying embankment. However, the magnitude of differential settlement is estimated to be less than 6 inches, or less than half the minimum slab thickness, and the spillway would be expected to pass limited flows without additional damage following a seismic event.

- There is a potential for failure in bending at the base of the spillway walls due to seismic shaking loads. However, the potential failure can be mitigated with buttresses without loss of spillway capacity.

- There is a potential for moment-induced failure of the outlet tower due to seismic shaking loads, as well as sliding and overturning of the tower foundation block. As a result, there is likely to be loss of control of flow through the outlet conduit

following a seismic event. However, the potential tower failure and the concerns for foundation sliding and overturning can be mitigated with buttresses.

- It is possible that consideration of ground motion data from the Northridge Earthquake could increase the ground motions postulated for the current analyses. In the case of liquefaction-induced embankment deformation, it is unlikely that such an increase would change the results presented herein because the postulated ground motions are already so high that the limiting (N1)60 , above which liquefaction would not occur, would not change. With respect to the outlet tower and the spillway, however, the increased ground motions could be significant to the analysis and design of the mitigation measures.

4.2 RECOMMENDATIONS

In general, the analyses of the Santa Anita Debris Dam embankment, spillway, and outlet works were based on relatively conservative geotechnical material characterizations which were developed from relatively limited as-built information and data. However, while it is possible that the characterizations actually represent localized conditions at the project site, given the limited nature of the data, it can also reasonably be assumed that the characterizations represent general conditions. In either case, the evaluations of the seismic and post-seismic behavior of the dam indicate that, while its performance may be impaired, it is not likely to suffer a catastrophic failure. With this in mind, the following recommendations are made with respect to improving the seismic and post-seismic performance of the dam embankment, spillway, and outlet works:

- It is recommended that the seismic design parameters be reevaluated to take into account ground motion data developed from records of the 1994 Northridge Earthquake. Modifications to the design earthquake magnitudes and peak ground accelerations could be particularly significant with respect to structural analysis and design.

- The recommended mitigation against seismic-induced failure of the spillway walls is the construction of buttresses against the inside surface of the walls. While the buttress concept presented herein is reasonably conservative, it is possible that more refined design-level analyses would result in a more cost-effective design. The final spillway design studies should also include an evaluation of the flow velocities and freeboard in the spillway chute and the downstream channel.

- The recommended mitigation against seismic moment-induced failure of the outlet tower is the construction of external buttresses. However, it is recommended that the design of the buttresses be based on the results of a response spectrum analysis

of the structure rather than the provisions of the Uniform Building Code. The response spectrum analysis will provide a more realistic model of the response of the structure to seismic ground motions, which could result in the development of a more cost-effective buttress configuration.

5.0
GENERAL CONDITIONS

This report presents our conclusions and recommendations pertaining to the Santa Anita Debris Dam, based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed by our exploration program. In view of the general geology of the area, the possibility of different conditions cannot be discounted. This report also presents the results of what we consider to be the most critical analyses.

Professional judgments represented in this report are based on evaluations of the technical information gathered, our understanding of the operation of the dam, and on our general experience in geotechnical engineering and engineering geology. We do not guarantee the performance of the project in any respect, only that our engineering work and judgments rendered meet the standard of care of our profession at this time.

We believe that our conclusions are consistent with the information and data available to us; however, the possibility exists that future studies may identify geologic/ geotechnical and seismicity issues that are not currently apparent and that could affect the performance of the embankment, spillway, and/or outlet works.

FINAL REPORT

SANTA ANITA DEBRIS DAM SPILLWAY RELOCATION CONCEPTS

Prepared for

County of Los Angeles
Department of Public Works
900 South Fremont Avenue
Alhambra, California 91803-1331

May 26, 2004

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2020 East First Street, Suite 400
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1.0 INTRODUCTION

1.1 PROJECT DESCRIPTION AND BACKGROUND

Santa Anita Debris Dam is owned and operated by the Los Angeles County Department of Public Works (Department). It is located near the mouth of Santa Anita Canyon on the south slopes of the San Gabriel Mountains in Arcadia. A plan of the dam and debris basin is shown on Figure 1. The dam has a maximum height of 56 feet with a crest elevation of 796 feet. The dam is approximately 955 feet long, with a crest width of approximately 20 feet, and upstream and downstream slopes of 2:1 (horizontal: vertical). The dam is a zoned earth embankment with a central vertical core zone consisting of silty and clayey sand with transition and shell zones consisting of sandy and gravelly alluvial materials borrowed from within the debris basin. The spillway consists of an ungated, concrete-lined rectangular open channel, founded within the embankment near the left abutment as shown on Figure 1. The spillway is approximately 160 feet wide at the spillway crest, which is at approximately elevation 774.75 feet. The spillway was designed to pass the design flood of 38,000 cubic feet per second (cfs) and provide a freeboard of 3 feet (USACOE, 1955).

Santa Anita Debris Dam was designed and constructed by the U.S. Army Corps of Engineers – Los Angeles District (Corps) in the 1955-59 time frame. The local sponsor was Los Angeles County Flood Control District, now part of the Department of Public Works, which assumed ownership of the dam in 1959. The primary function of the dam was debris retention; however, the Department has historically operated the facility for both debris retention and water conservation. The Santa Anita Spreading Grounds were constructed downstream from the dam for groundwater recharge.

Under the ownership of the Department, Santa Anita Debris Dam fell within the jurisdiction of the State Division of Safety of Dams (DSOD). In 1982, the DSOD issued a Certificate of Approval for operation of the dam, contingent upon a satisfactory seismic safety evaluation by the Corps.

The Corps conducted a Phase I safety evaluation of Santa Anita Debris Dam in 1983 as part of the National Dam Safety Assurance Program (USACOE, 1985). The Corps concluded that, given the Department's then-current operating procedure, the dam embankment and foundation were not susceptible to general liquefaction and significant loss of strength due to earthquake shaking. The Corps also concluded it was highly improbable that earthquake forces on the appurtenant structures of the dam would cause a catastrophic structural failure that would endanger lives and property downstream of the dam.

In their review of the Corps report, the DSOD did not fully concur with the Corps' conclusions, in part because of inadequate data. In response to DSOD comments, the Corps performed additional Phase I studies (USACOE, 1987). However, the DSOD continued to have concerns regarding the adequacy of the information and data used by the Corps in developing its conclusions. Specifically, the DSOD remained concerned about the potential for seismic-induced deformation of the embankment and spillway, and seismic-induced failure of the outlet works.

From 1991 to 1994, Woodward-Clyde Consultants (Woodward-Clyde) performed a seismic evaluation of Santa Anita Debris Dam. This seismic evaluation included performing additional field exploration and laboratory testing, developing design-level seismic ground motion parameters, performing analyses, and developing remedial measures as necessary. The seismic evaluation considered using the dam and basin for water conservation purposes. The conclusions of that seismic evaluation are summarized as follows:

- The potential for surface fault rupture at the dam site appears to be low.
- There is a potential for seismic-induced liquefaction of the dam embankment. The consequences of liquefaction would be limited cyclic mobility; i.e. the response of the material would be a reduction in shear strength to undrained residual values rather than flow-type behavior.
- The potential for liquefaction of the foundation alluvium at the Santa Anita Debris Dam site is relatively low.
- Post-seismic stability analyses of the dam indicate a minimum factor of safety of 1.6.
- Simplified and conservative seismic deformation analyses of the dam indicate seismic deformations of less than 1 foot at the dam crest.
- Post-liquefaction settlement of the dam is estimated to be on the order of 6 inches.
- The spillway has an adequate factor of safety against sliding and gross overturning. However, the spillway slab could undergo lateral spreading and differential settlements as a result of seismic deformation and post-liquefaction settlement of the underlying embankment.
- The spillway walls do not have sufficient moment capacity under seismic conditions.
- The intake tower is not stable under seismic loading.

In response to these findings, in 1995, the DSOD restricted the Department from continued operation of the project for water conservation. Alternative concepts were developed for mitigating the seismic hazards to the spillway and intake tower, with the goal to remove the storage restriction. This current study is the evaluation of the technical feasibility and cost implications of alternatives for relocating the spillway from the dam embankment to a more stable foundation.

1.2 Objective and Scope of Work

The scope of URS' current study for relocating the spillway at Santa Anita Debris Dam is two-fold: 1) identify a technically feasible site for relocation that satisfies the DSOD's stability criteria and the Department's operation requirements, and 2) estimate the costs associated with relocation of the spillway. Tasks performed for this study included: a review of the existing geotechnical and geologic data for the project; a site reconnaissance by a registered geotechnical engineer and engineering geologist; development of preliminary conceptual relocation alternatives; development of preliminary cost estimates for selected alternatives; and recommendations for the next phase of work.

2.0 SITE CONDITIONS

Messrs. Michael Smith, G.E., and Chris Goetz, C.E.G., from URS performed a reconnaissance of the Santa Anita Debris Dam site on January 27, 2003. The site visit included the dam embankment and abutments, the debris basin, and the downstream channel and vicinity. Photographs were taken and notes on rock exposures in the left abutment were made. Selected photographs from the site reconnaissance are presented in this report as Photos 1 through 4. We also reviewed the geologic data available in the seismic evaluation reports (Woodward-Clyde, 1994; USACOE, 1985).

2.1 COMMUNITY SETTING

Santa Anita Debris Dam is located at the mouth of Santa Anita Canyon and Wash, north of the City of Arcadia. The land to the east of the project site is undeveloped, as shown on Figure 1. This area consists of densely vegetated hills and canyons. There are also two fills is this area. There is a larger fill about 900 feet downstream from the left (east) abutment of the dam that may be muck from construction of the Metropolitan Water District of Southern California's Upper Feeder tunnel, or debris from Santa Anita debris basin. A smaller fill is located at the left abutment of the dam, extending over that abutment, apparently providing access to the debris basin from the downstream.

The land to the west of the project site is residential development of the City of Arcadia.

2.2 GEOTECHNICAL SETTING

2.2.1 Regional Geology

Santa Anita Debris Dam is located at the southern margin of the San Gabriel Mountains, which is part of southern California's Transverse Ranges physiographic and geologic province. This province, which is one of the most seismically active regions in California, is characterized by tectonic compression across east-west-trending reverse and strike slip faults. The Sierra Madre fault, which is considered capable of producing an earthquake with a maximum magnitude greater than 7, is located about 1000 feet south of the dam. The San Andreas Fault, which is considered capable of producing an earthquake with a maximum magnitude greater than 8, is located about 33 kilometers northeast of the dam. These and other active faults near the dam are considered capable of producing significant seismic shaking at the dam site.

2.2.2 Local Geology

The Santa Anita Canyon walls are composed of granitic bedrock. Alluvial materials overlie the granitic bedrock along the canyon floor. Near surface bedrock is moderately to highly weathered and very closely fractured by joint sets and numerous minor shears. Alluvial materials consist of older terrace deposits and recent streambed deposits of the Santa Anita Wash.

2.2.3 Foundation Geology

The dam embankment and right abutment are founded on alluvial materials, and the left abutment is founded on granitic bedrock. The right abutment of the dam is a gently sloping alluvial terrace composed of older alluvium. The terrace alluvium consists chiefly of reddish brown silty sand, and clayey sand with occasional gravel (USACOE, 1985; Woodward-Clyde, 1994). The area upstream of the right abutment within the older alluvium was designated as a source for the impermeable core of the embankment. The central portion of the embankment, in the vicinity of the spillway, is founded on recent alluvium. The recent alluvium consists primarily of poorly graded sand with silt and gravel, poorly graded gravel with silt and sand, and silty sand with gravel (USACOE, 1985; Woodward-Clyde, 1994). The left abutment of the dam is a steeply sloping bedrock ridge. Colluvial debris and thick vegetation mostly cover the ridge. The bedrock is typically slightly to moderately weathered, and very closely fractured by multiple joint sets. The Corps (USACOE, 1985) mapped a fault in the granitic rock of the left abutment. The fault was described as a narrow zone of gouged and crushed rock near the southern edge of the rock spur. The fault has a reported strike of N40°W and a dip of 60° northeast. Woodward-Clyde (1994) concluded that the fault in the abutment was not active.

The depth to bedrock beneath the dam ranges from zero at the left abutment to an unknown depth at the right abutment. Based on seismic refraction surveys and drill hole data, Woodward-Clyde (1994) estimated that the top of bedrock descends from the left abutment to greater than 50 feet deep beneath the dam. Woodward-Clyde characterized the bedrock as having shear wave velocities ranging from 2,000 to 2,900 fps and compression wave velocities ranging from 5,000 to 12,500 fps.

The groundwater within the alluvium varies seasonally. The Corps (USACOE, 1985) concluded that the groundwater table is at or near the ground surface within the basin during the wet season (November through April) and up to 18 feet deep during the dry season (May through October). Rotary wash boring RW-SA2 (Woodward-Clyde, 1994) indicated that the elevation of the groundwater measured May 24, 1991 was 2 feet below the dam/alluvial foundation interface, or at elevation 745.5 feet MSL.

3.0 DESCRIPTION OF ALTERNATIVES

Options for relocating the spillway from the Santa Anita Debris Dam embankment are limited to either the right or the left abutment areas. Relocating the spillway on the right (west) abutment is not considered practical for two reasons: 1) the community development in that area, and 2) the depth to bedrock in excess of 50 feet. However, neither of these constraints is present at the left (east) abutment. The estimated surface and subsurface conditions at the left abutment of Santa Anita Debris Dam are depicted on Sections A-A' and B-B' on Figures 2 and 3, respectively (the locations of the cross sections are shown on Figure 1).

Three different alternatives were considered for relocating the spillway to the left (east) abutment of the dam, namely: 1) excavate a tunnel through the bedrock ridge forming the abutment; 2) excavate the existing alluvial soils to bedrock and replace them with mass concrete; and 3) form the spillway in a rock cut in the abutment ridge. The tunnel concept was dismissed as being impractical given the large section that would be required to pass the design flood and the likelihood that an easement would be required for the property located east of the dam site. The other two alternatives are described below.

It is noted that, for this alternatives study, it was assumed that the hydrologic calculations originally performed in 1955 to determine the spillway design flood are valid and updates will not be required. With this assumption, the spillway control section and discharge channel for the relocation concepts were modeled after the existing spillway control section and discharge channel, namely, an ungated broad-crested weir and a rectangular channel, respectively. It is possible that improved hydraulics and reduced costs may be achieved with a different type of spillway, e.g., a side-channel spillway with an ogee weir and a trapezoidal channel. It is assumed that the preliminary and final design tasks will include an optimization study relative to the spillway configuration.

It is also assumed that the findings from Woodward-Clyde's previous studies of the Santa Anita Debris Dam embankment (Woodward-Clyde, 1994) are valid and updated analyses will not be required except at the tie-in between the embankment fill and the new spillway. Thus, it is assumed that upgraded analysis of the embankment is only required for the embankment/spillway contact. It is envisioned that this analysis would be performed using numeric modeling methodology, and would be performed as part of the preliminary and final design effort.

3.1 MASS CONCRETE ALTERNATIVE

This alternative consists of relocating the spillway to the left abutment, excavating to bedrock, backfilling with mass concrete, and constructing the spillway on the new concrete foundation. The relocated spillway for the mass concrete alternative is shown in plan on Figure 4 and in longitudinal and transverse sections on Figures 5 and 6, respectively.

As shown on Figures 4 and 6, the mass concrete spillway alternative would involve an excavation through the existing embankment fill and recent alluvium to bedrock. We have also assumed that the upper 5 feet of bedrock would be weathered and would be excavated. The deepest part of the excavation, measured

from the dam crest, would be approximately 100 feet. The bottom of excavation would extend to 50 feet below the known groundwater table and require dewatering to a depth of 60 feet or more (at least 10 feet below the bottom of excavation). The excavation would then be backfilled to grade with mass concrete. Engineered fill would be used to backfill the portion of the foundation material that does not under lay the spillway structure. A 4H:1V bonding slope would be constructed within the embankment. Zoned embankment fill would be placed between the mass concrete and the existing embankment.

The excavation for the mass concrete alternative would remove the existing spillway. The new spillway would have the same crest elevation as the existing spillway (approximately 774.75 feet MSL), but with a shorter crest width of 148 feet. The new spillway would provide 1.5 feet of freeboard during the design flood. We have assumed that the existing spillway weir and discharge chute would be replaced with engineered fill.

The mass concrete alternative also includes potentially decommissioning the existing intake tower and 48-inch pool drain conduit, and the construction of a new intake structure (a trash rack and slide gate) at the upstream end of the new spillway, with a low-level drain conduit under the spillway, through the concrete as shown on Figures 4, 5, and 6. In this case, final design would need to include a diversion from the new pool drain to the existing downstream spreading grounds. Alternatively, it may be more cost effective to strengthen the existing intake tower for seismic loadings, and simply extend the existing drain conduit to intersect the relocated spillway channel, as shown on Figure 4, thereby maintaining the existing spreading grounds diversion structure.

The mass concrete alternative could be constructed using conventional cast-in-place methods, or roller-compacted concrete (RCC), or a combination of the two methods. A preliminary cost estimate for the mass concrete alternative, based on RCC and a new intake tower and outlet conduit and demolition/decommissioning of existing, is provided in Table 1. The preliminary construction cost estimate is $7.5 million and includes a ***50 percent contingency***. This preliminary cost does not include engineering design or construction management costs.

3.2 PARTIAL ROCK CUT / PARTIAL MASS CONCRETE ALTERNATIVE

This alternative consists of relocating the spillway to the left abutment, excavating into the rock ridge as well as through a portion of the existing embankment fill and recent alluvium, backfilling a portion of the excavation with mass concrete, and constructing the spillway partially on bedrock and partially on the mass concrete foundation. As compared to the mass concrete alternative, this alternative will place a greater portion of the spillway section on bedrock, will reduce the volume of mass concrete, and will reduce the depth of required excavation. The partial rock cut alternative is shown in plan on Figure 7 and in transverse section on Figure 8, respectively. Figure 7 also depicts the approximate limits of the cut slope, which are constrained by the Department's property line.

As shown on Figures 7 and 8, the partial rock cut/partial mass concrete alternative would include the excavation of an approximately 80 foot high 1/2H:1V cut slope in rock. The rock in this ridge generally consists of slightly to moderately weathered granitic rock that is very closely fractured by multiple joint sets. There is little geotechnical information on the rock at this location in terms of its excavation and

stability characteristics. Because it is very closely fractured, it is likely rippable to a significant depth. It is also likely to need, as a minimum, surface treatment to reduce raveling of loose rock blocks, and more likely, rock bolts with or without shotcrete for more global stabilization. We have assumed in our evaluation that the excavation will be performed by drilling and blasting, using controlled blasting and pre splitting of the back cut, and that the slope cut will be supported with rock bolts. It is noted that the closest residences are about 600 feet from the excavation (see Figure 1).

Also as shown on Figure 8, there would also be an excavation through the existing embankment fill and recent alluvium to bedrock. The excavation from the dam crest would be approximately 75 feet in depth. The bottom of excavation would extend to about 20 feet below the known groundwater table and require dewatering to a depth of 30 feet or more (at least 10 feet below the bottom of excavation). The excavation would then be backfilled to grade with mass concrete. Engineered fill would be used to backfill the portion of the foundation material that does not under lay the spillway structure. A 4H:1V bonding slope would be constructed within the embankment. Zoned embankment fill would be placed between the mass concrete and the existing embankment.

It is envisioned that the partial rock cut/partial mass concrete spillway section would be cut to the same dimension as the mass concrete construction. However, it is noted that other spillway configurations, such as a side-channel spillway, may reduce the amount of rock cut required for this alternative. As shown on Figure 8, the excavation for the partial rock cut alternative would remove the existing spillway. The new spillway would have the same crest elevation as the existing spillway (approximately 774.75 feet MSL), but with a shorter crest width of 148 feet. It is assumed that the cut rock surface within the spillway section will require cleaning, mapping, dental concrete and backfill concrete for hydraulic integrity. The new spillway would provide 1.5 feet of freeboard during the design flood. We have assumed that the existing spillway weir and discharge chute would be replaced with engineered fill.

The partial rock cut alternative also includes potentially decommissioning the existing intake tower and 48-inch pool drain conduit, and the construction of a new intake structure (a trash rack and slide gate) at the upstream end of the new spillway in the same manner as the mass concrete alternative. Like the mass concrete alternative, in this case, final design considerations must include a diversion from the pool drain to the existing downstream spreading grounds.

A preliminary cost estimate for the partial rock cut/partial mass concrete spillway alternative is provided in Table 2. The preliminary construction cost estimate is $6.4 million and includes a ***50*** *percent contingency*.. This preliminary cost does not include engineering design or construction management costs.

4.0 EVALUATION OF ALTERNATIVES

Based on our current understanding of the site subsurface conditions, the partial rock cut/partial mass concrete alternative provides several advantages over the mass concrete alternative, namely:

- Reduced impact to existing embankment.
- Less uncertainty with respect to depth of excavation.
- Less embankment reconstruction.
- Less dewatering.

The mass concrete spillway provides the following advantages over the partial rock cut/partial mass concrete alternative:

- Less permanent visual impact.
- Less rock excavation (potential for blasting).
- Reduced concerns for rock slope stabilization.

With regard to cost, it appears that either alternative could be constructed for less than $8 million. The partial rock cut/partial mass concrete alternative appears to be less costly than the mass concrete alternative; however, differences in geologic conditions from those assumed, particularly assumed depths of removals and dewatering requirements for each alternative and rock slope stabilization required for the partial rock cut alternative, could change this comparison.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

Based on available information, it appears technically feasible to relocate the Santa Anita Debris Dam spillway from the dam embankment to the left (east) abutment. However, the final location, and configuration and cost of the spillway will be highly dependent on actual geologic conditions, which will govern depths of excavation and dewatering requirements. It appears that construction costs will be less than $8 million to relocate the spillway to the left abutment and restore the dam embankment to service for both flood control and water conservation.

5.2 RECOMMENDATIONS

We recommend geotechnical exploration of the left abutment be performed to characterize existing bedrock conditions for refinement of conceptual relocated spillway layouts and cost estimates, specifically, geologic mapping and diamond core borings on the east side of the existing dam and in the left abutment rock mass.

Figure 9 presents URS' recommended geotechnical exploration program for a refined evaluation of the bedrock conditions critical to the relocated spillway concepts presented herein. As shown on the figure, the recommended program consists of three core borings. The purpose of the core borings is to explore the depth to competent bedrock and recover samples of the bedrock for engineering characterization and laboratory testing. The borings will be cored using a diamond core bit and triple tube core barrels. Core logs will be prepared with measurements of fracture spacing and orientation and characterizations and descriptions of the rock and the rock mass discontinuities. At completion of drilling, the borings will be backfilled with grout. We recommend at least one day of geologic field mapping in conjunction with the core boring.

Laboratory testing on the rock core samples will depend on the quality of the samples retrieved. As a minimum, we will test the unconfined compressive strength of the intact rock. We will also attempt to perform direct shear tests on samples of joints and other discontinuities.

Engineering analyses associated with the left abutment characterization will include joint set analyses, potential wedge stability analyses, and global stability analyses. We will also refine the longitudinal and transverse cross sections presented herein and the associated quantity take-offs and cost estimates.

If dewatering is considered further, we recommend installing wells within the alluvium to monitor the depth to groundwater and perform pumping tests to develop parameters for dewatering system design. However, we do not consider this program as being critical for the development of alternative concepts for spillway relocation; rather, dewatering issues should be addressed during the preliminary design phase of the project, if the project proceeds that far.

6.0 GENERAL CONDITIONS

This report presents our evaluations and recommendations relative to alternatives for relocating the spillway at Santa Anita Debris Dam. The recommendations and professional judgments represented in this report are based on limited available technical information, our understanding of the Santa Anita Debris Dam and basin and the current and possible future operation of the facility and our general experience in the fields of engineering geology and geotechnical engineering. We do not guarantee the performance of the project in any respect, only that our engineering recommendations and judgments rendered meet the standard of care of our profession at this time.

We believe that our recommendations are consistent with the information and data available to us; however, the possibility exists that future studies may identify geologic/geotechnical or hydrogeologic issues that are not currently apparent and that could affect our interpretation of the advantages of different alternatives.

This report was prepared for the sole and exclusive use of the Los Angeles County Department of Public Works for the Santa Anita Debris Dam project. This report has not been prepared for use by other parties or for other uses, and may not contain sufficient information for the purposes of other parties.

LEGEND

Contact, approximately located, dotted where concealed.	
Kgr	Granitic Bedrock
Qoal	Older Alluvium
Qal	Younger Alluvium
af	Artificial Fill

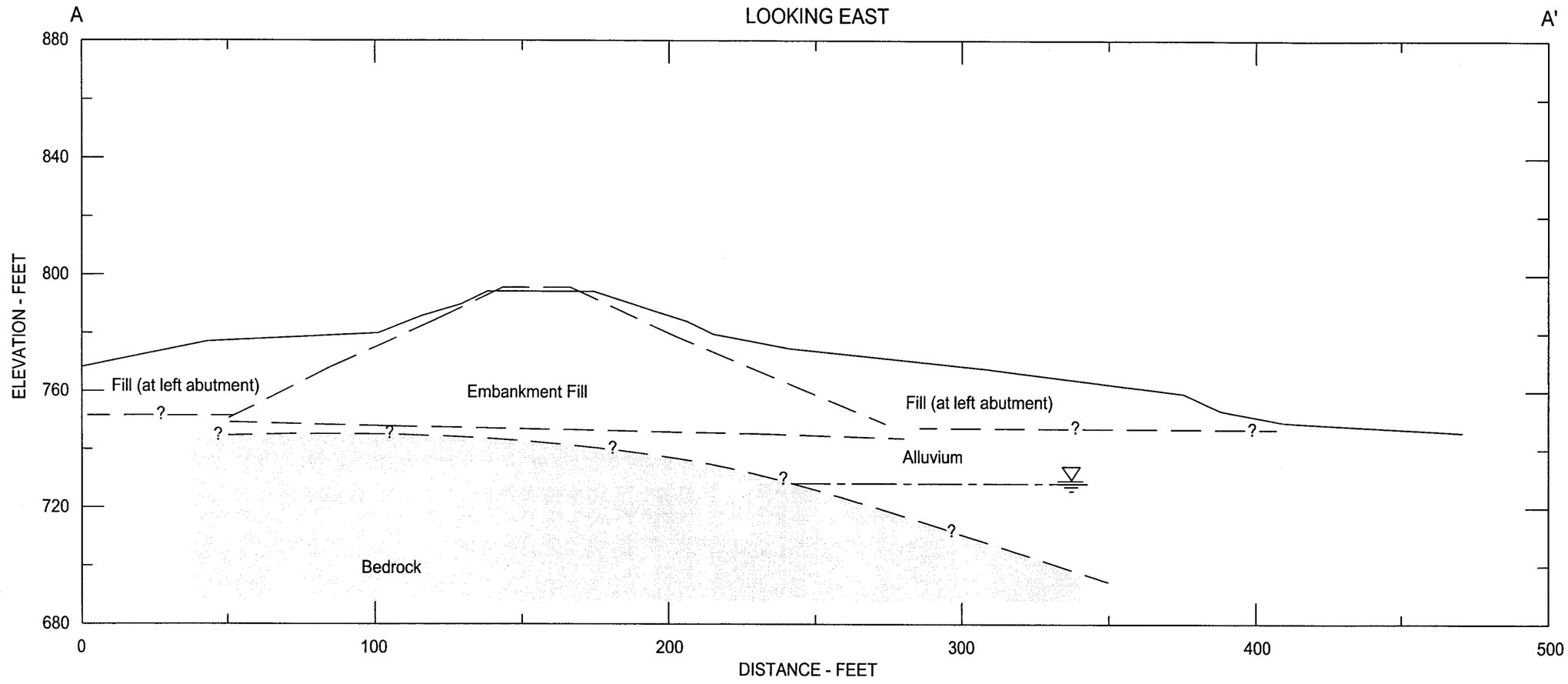
Notes:
1. Base topography from Los Angeles County Department of Public Works drawing "Santa Anita Debris Basin", dated 04-15-93 and revised 09-30-93.
2. Airphoto 2002.
3. See Figures 2 and 3 for Sections.

URS

SITE PLAN AND GEOLOGIC MAP

Proj. No.: 29871204	Date: MAY 2004
Project: SANTA ANITA DEBRIS DAM SPILLWAY RELOCATION	Figure: 1

T:\2002\v12\Debris Dams-Santa Anita\Santa Anita\cad\052504\GeologyPhoto.dwg May 26, 2004 - 11:31am

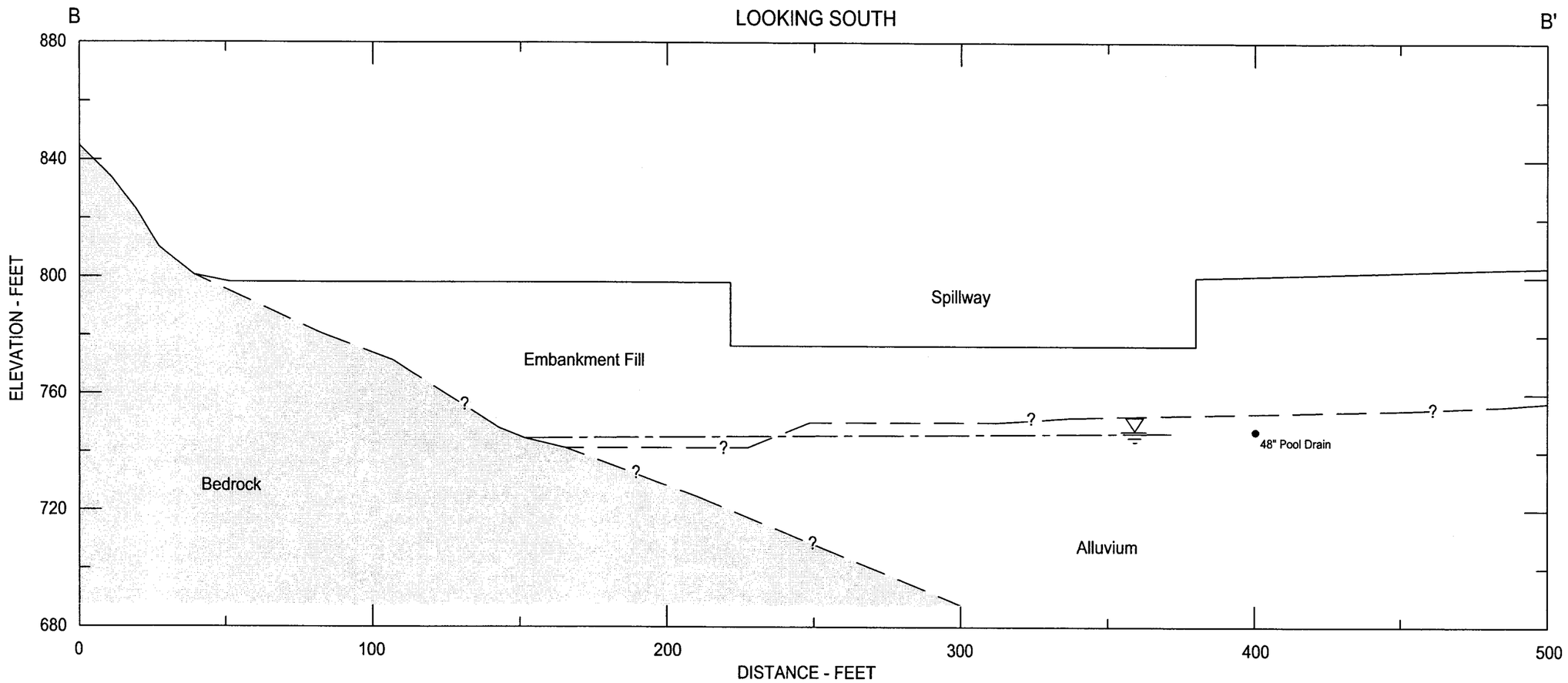


Note:
1. Location of Cross Section shown on Fig. 1
2. The internal zoning of the embankment fill is not shown.

CROSS SECTION A-A' - EXISTING CONDITIONS

Project No. 29871204	Date: May 2004	Project: SANTA ANITA DEBRIS DAM - SPILLWAY RELOCATION	URS	Figure: 2

T:/2002/12/Debris Dams-Santa Anita/Santa Anita/cad/052504/Figure 2.dwg



Note:
1. Location of Cross Section shown on Fig. 1

CROSS SECTION B-B' - EXISTING CONDITIONS

Project No. 29871204	Date: May 2004	Project: SANTA ANITA DEBRIS DAM - SPILLWAY RELOCATION	URS	Figure: 3

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POTENTIAL NEW INTAKE STRUCTURE

POTENTIAL NEW POOL DRAIN

RELOCATED SPILLWAY

EXISTING SPILLWAY

STRENGTHEN OR DECOMMISSION EXISTING TOWER AND CONDUIT

DECOMMISSION EXISTING SPILLWAY AND BACKFILL

POTENTIAL POOL DRAIN EXTENSION

A 5

B 6

Note:
Base topography from Los Angeles County Department of Public Works drawing "Santa Anita Debris Basin". dated 04-15-93 and revised 09-30-93.

0 100 200
SCALE, feet

URS

MASS CONCRETE ALTERNATIVE

Proj. No.: 29871204
Project: SANTA ANITA DEBRIS DAM SPILLWAY RELOCATION

Date: MAY 2004
Figure: 4

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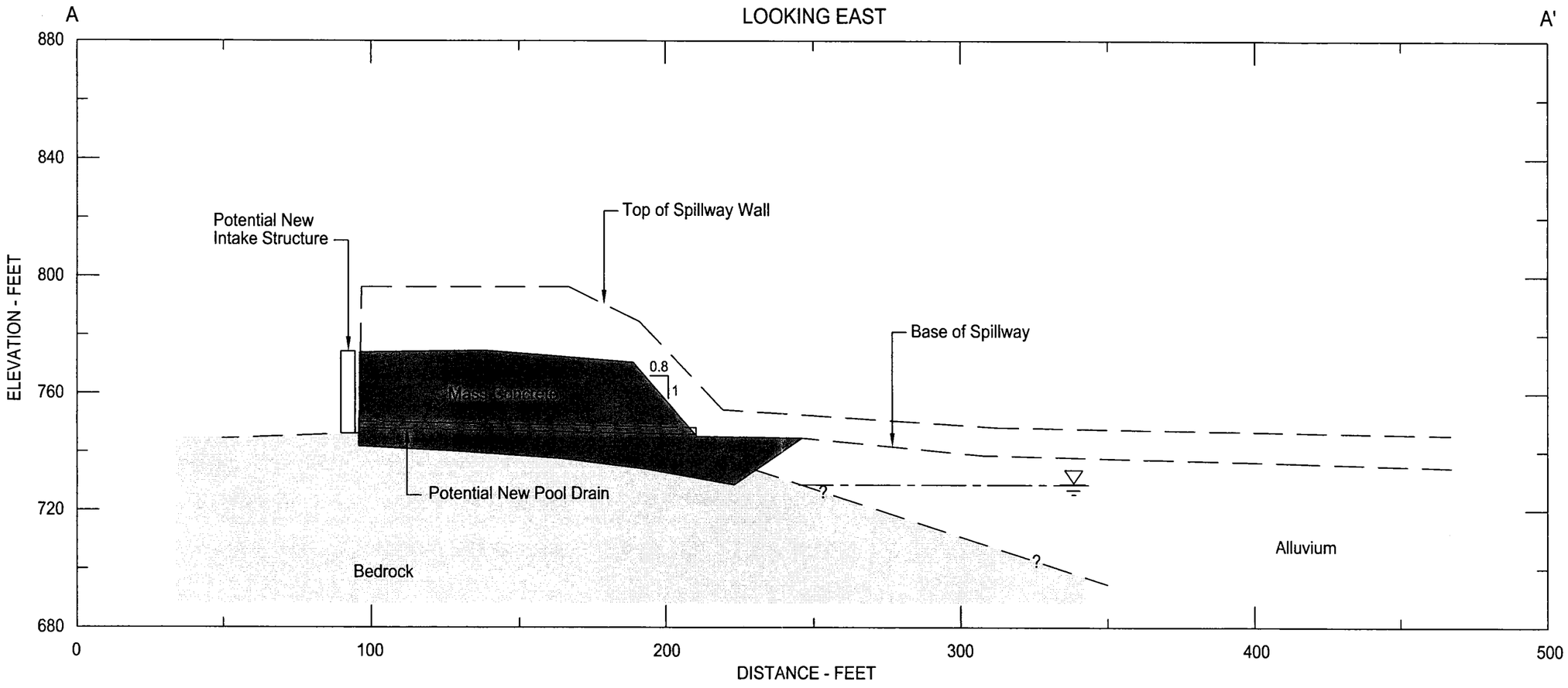


Note:
See Figure 4 for location of Section

CROSS SECTION A-A' - MASS CONCRETE ALTERNATIVE

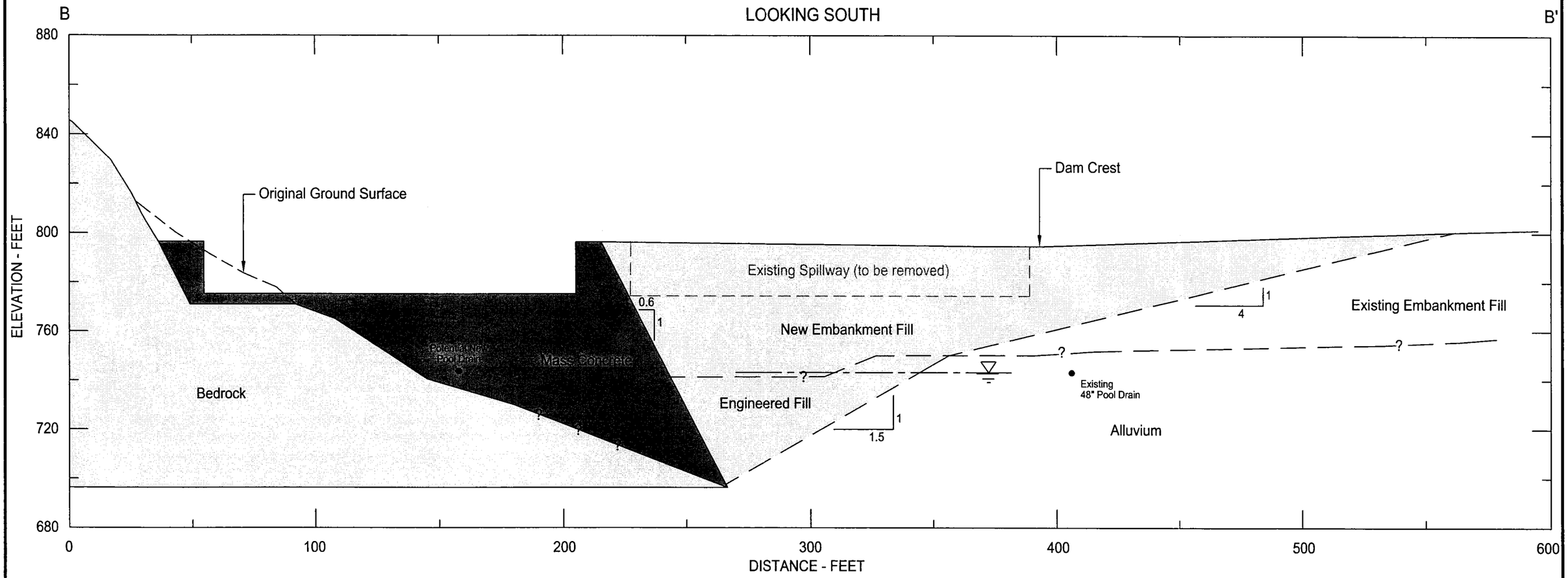
Project No. 29871204	Date: May 2004	Project: SANTA ANITA DEBRIS DAM - SPILLWAY RELOCATION	URS	Figure: 5

T:\2002\1\2\Debris Dams-Santa Anita\Santa Anita\cad\052504\Figure 5.dwg

B

LOOKING SOUTH

B'



Note:
See Figure 4 for location of Section

CROSS SECTION A-A' - MASS CONCRETE ALTERNATIVE			URS	Figure: 6
Project No. 29871204	Date: May 2004	Project: SANTA ANITA DEBRIS DAM - SPILLWAY RELOCATION		

T:/2002/(12)Debris Dams-Santa Anita/Santa Anita/cad/052504/Figure 6.dwg

LIMITS OF ROCK CUT

PROPERTY LINE

POTENTIAL NEW INTAKE STRUCTURE

POTENTIAL NEW POOL DRAIN

EXISTING SPILLWAY

RELOCATED SPILLWAY

STRENGTHEN OR DECOMMISSION EXISTING TOWER AND CONDUIT

DECOMMISSION EXISTING SPILLWAY AND BACKFILL

POTENTIAL POOL DRAIN EXTENSION

A

A'

E 281,600 N 174,400

E 281,400 N 173,800

E 281,000 N 173,600

E 280,800 N 173,200

E 280,800 N 174,000

0 100 200

SCALE, feet

Note:
Base topography from Los Angeles County Department of Public Works drawing "Santa Anita Debris Basin", dated 04-15-93 and revised 09-30-93.

URS

PARTIAL ROCK CUT/ PARTIAL MASS CONCRETE ALTERNATIVE

Proj. No.: 29871204	Date: MAY 2004
Project: SANTA ANITA DEBRIS DAM SPILLWAY RELOCATION	Figure: 7

T:\2002\d12\Debris Dams-Santa Anita\Santa Anita\cad\052504\Figure 7.dwg May 26, 2004 - 1:45pm

HARZA

Consulting Engineers and Scientists

EXISTING CONDITION ASSESSMENT REPORT SANTA ANITA DAM SEISMIC REANALYSIS AND SPILLWAY MODIFICATION STUDY

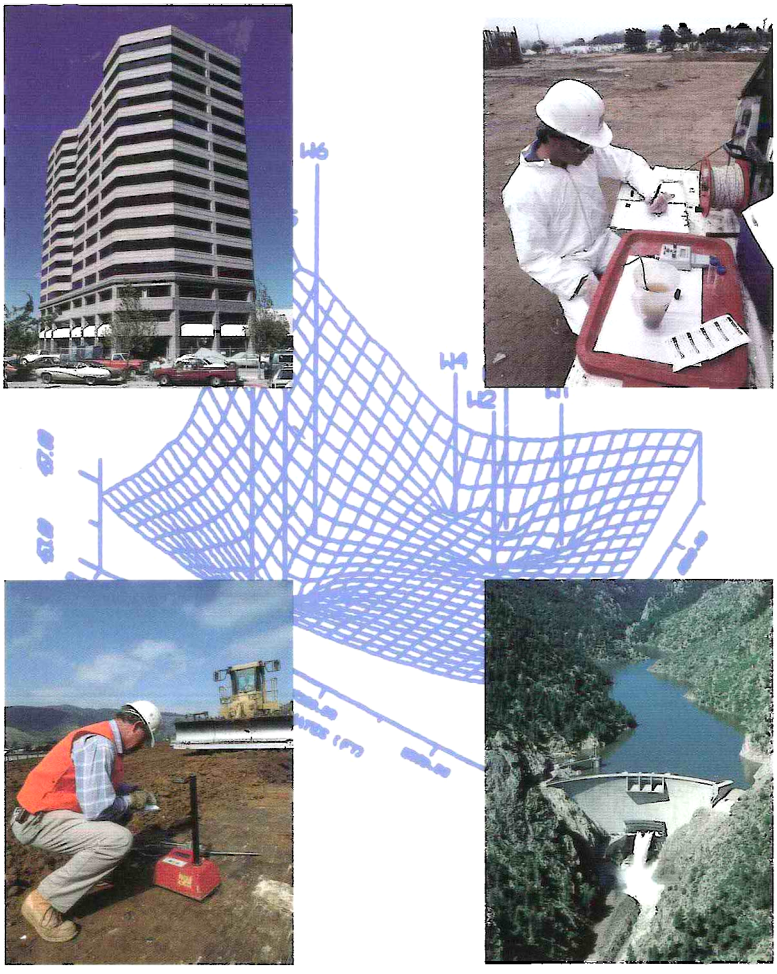
for

County of Los Angeles
Department of Public Works

by

Harza Engineering Company
Concord, California

April 3, 2000



1.0 INTRODUCTION

1 1 General Description and Background

Santa Anita Dam is located on the Big Santa Anita Creek approximately four miles north of the City of Arcadia, Los Angeles County, California. The location of the dam is shown on Figure 1-1 The dam, constructed in 1924 through 1927, is owned and operated by the County of Los Angeles Department of Public Works (County).

The dam is a constant-angle variable-radius concrete arch constructed for the purpose of flood control and water conservation (see Figure 1-2 for plan and profile views of the dam). The dam's crest height above the streambed at the downstream toe of the dam is 225 feet (dam crest El. 1,325), and the crest length of the arch (including the abutments) is 612 feet. The thickness of the dam varies from 10 feet at the crest (7 feet at the arch crest) to 61.5 feet at the base. The mass concrete was poured in blocks between nine full height vertical trapezoidal construction joints spaced approximately 50 feet apart.

The main spillway (spillway crest El. 1,316) consists of a rectangular covered box section channel with rounded entrance lip and sidewalls located on the right abutment. The total length of the chute section is approximately 170 feet along the centerline. An auxiliary spillway consists of a semi bowl-shaped inlet on the upstream dam face discharging into a circular conduit through the dam near the crest. Several valves (two needle valves each with diameter of 30 inches and one gate valve with diameter of 20 inches) are located between 120 to 160 feet below the dam crest in the central portion of the arch to regulate water level in the reservoir A fourth 18-inch diameter gate valve is located near the base of the dam, and a 72-inch diameter hydraulic sluice gate is located approximately 146 feet below the dam crest.

1.2 Previous Studies

Three previous studies by International Engineering Company (IECO, 1977), County (1978) and the State of California Department of Water Resources, Division of Safety of Dams (DSOD, 1980) were performed to assess the hydrologic/hydraulic condition of the project and to analyze the stability of the dam for static, seismic/dynamic, and probable maximum flood (PMF) loading. A brief description and primary findings of each of these studies is provided below.

1.2.1 Final Report on Investigation and Reanalysis of Santa Anita Dam (IECO, 1977)

In mid-1976, the County contracted with IECO for the purpose of conducting an investigation and reanalysis of Big Santa Anita Dam as part of the County's overall program for reevaluating their major dams. The IECO study included review and analysis of available monitoring data, extensive field investigations of dam concrete and foundation rock, laboratory testing of both concrete and rock, comprehensive studies of the local and regional geology, seismicity, and fault capability, and stress analyses of the dam. Linear-elastic stress analyses were performed using

computer program SAP (acronym for Structural Analysis Program), considered to be state-of-the-art in 1977 The primary findings of the final report produced by IECO (1977) were as follows:

- The dam has adequate structural stability for the stipulated static loading which represents a reasonable sustained loading condition, and for the extreme reservoir loading condition corresponding to the probable maximum precipitation (PMP) flood level;

- The dam has adequate safety factors to safely sustain a maximum probable earthquake (MPE) with reservoir full to spillway crest level;

- The maximum credible earthquake (MCE) generates an extremely severe response in the dam in its present condition, with reservoir full to spillway crest level, resulting in inadequate safety factors;

- Appropriate remedial measures are required to ensure adequate safety factors for safe unrestricted operation of the dam for a MCE;

- Reservoir water level El. 1,280 represents a safe operating level for the dam to safely sustain a MCE;

- Both abutments have adequate stability against both the MPE and MCE;

- The dam concrete is hard, compact and free from significant segregation or bleeding of the fresh concrete;

- Petrographic examination of concrete cores indicate evidence of alkali-aggregate reaction (AAR) during service and potential for continued reaction and expansion in future at a slow rate of progression;

- The observed historic continuing rise of the dam crest elevation relative to the abutments is probably due to expansion resulting from AAR, occurring in the dam concrete. Such a rise presently (1977) has no significant effect on the structural stability of the dam.

1.2.2 Santa Anita Dam, Spillway Modifications (County, 1978)

The County conducted a hydrologic and hydraulic study for the dam presented in their project report "Santa Anita Dam, Spillway Modifications" dated June 1978. The results of the hydrologic study determined that a peak flow rate of 12,800-cfs was generated based on the probable maximum precipitation (PMP) developed from the National Weather Service's "Hydrometeorological Report No. 36, Probable Maximum Precipitation in California" (HMR36, NWS, 1961). The existing spillway capacity as determined by the County is 5,660-cfs.

As a result of the County's 1978 study, several spillway modifications were recommended for improving the project's discharge capacity. However, the recommendations were not implemented.

1.2.3 Phase 1 Inspection Report for Big Santa Anita Dam (DSOD, 1980)

The DSOD (1980) study was prepared as part of the National Dam Inspection Program funded by the Department of the Army Corps of Engineers, Sacramento District. This report included a description of the project (i.e., data regarding the reservoir, dam, spillways, outlet works, watershed, operation, and emergency warning procedures), geology, hydrology/hydraulics, and a visual inspection of the dam. The primary findings of the DSOD (1980) were as follows:

- The dam is founded on bedrock which, although somewhat fractured, is considered an adequate foundation for the structure, and which could withstand a MCE (i.e., in 1980, this was considered to be an event of Richter Magnitude 7.0 on the Sierra Madre fault);

- Various tests and inspections of the dam concrete indicate it is sound and in good condition;

- Based on the IECO (1977) study, there is potential for high tensile concrete stresses under the MCE. As a result, the maximum operating level of the reservoir has been restricted to Elevation 1280, which would reduce the tensile stresses to allowable values. DSOD believes the restricted water surface elevation to be safe for continued operation of the dam;

- The combined capacity of the main, auxiliary, and crest spillways of the dam are inadequate to pass the PMF from either the general or local PMP storms without overtopping the crest parapet wall up to a maximum of about 4 feet. DSOD believes this would not seriously endanger the safety of the dam because of the relatively short period of overtopping, and the high degree of downstream channel and slope protection;

- Heavy siltation and consequent loss of reservoir storage capacity has been a continuing problem at the dam, which has necessitated periodic draining of the reservoir and removal of the silt by various methods;

- Precise crest monument surveys have indicated a slow rate of both residual upstream and vertical movement of the dam. Although not considered serious, continued close surveillance and evaluation of these movements have been recommended as part of the 1977 investigation and reanalysis; and

- Overall, DSOD believes that the dam is in good condition, functioning satisfactorily for its intended purpose, and considered safe for continued operation.

1.3 Purpose of the Current Study

Given the rapid scientific advance in the fields of seismology and numerical analyses (i.e., linear and non-linear computer programs) over the past two decades, the studies performed by IECO (1977), County (1978), and DSOD (1980) can no longer be considered state-of-the-art due to the following reasons:

The design earthquake ground motions developed in 1977 were based on the relatively scant database of near-field ground motion records available at that time. Recent seismologic studies and ground motion attenuation relationships predict that the MCE will be a larger event than predicted in 1977, with significantly higher peak ground acceleration

All of the stress analyses performed in 1977 were linear-elastic, which did not take into account opening and closing of the vertical construction joints during a seismic event. As shown in Chapter 5.0 of this report, inclusion of the vertical joints in the finite element model have a significant impact on stress distribution within the dam.

Given the above, the County contracted with Harza Engineering Company (Harza) to perform an updated study for Santa Anita Dam. The purpose of this study is to provide a comprehensive updated reanalysis of the dam, including the hydraulic adequacy of the existing spillways/outlet works and structural evaluation of the static and seismic stability of the dam.

Harza's approach for reanalysis was presented in the Analysis Plan dated November 18, 1999. Upon request from DSOD, Harza subsequently submitted revised ground motion records and response spectra on January 21 and February 7 and 21, 2000. DSOD issued approval for the Analysis Plan and revised ground motion records in their letters dated January 26 and March 17, 2000.

The purpose of this report is to present results of data compilation, field reconnaissance and investigations, laboratory testing and engineering analyses which provide an assessment of the existing condition of Santa Anita Dam.

1.4 Analytical Approach

As previously presented in the Analysis Plan, Harza's services for Santa Anita Dam will be performed in two phases. Phase I will be reanalysis of the existing dam and development of rehabilitation concepts, including 30 percent "conceptual" design level documents for the preferred concept.

The first objective of the tasks and activities within Phase I is the analysis (using the latest technological methods and site data) and evaluation of the safety issues of the Santa Anita Dam. The issues of concern are:

- Seismic stability;
- Hydrology and hydraulic performance; and
- Alkali aggregate reactivity (AAR) in dam concrete.

Since the above main issues rely heavily on the geology (foundation rock) and local seismicity (ground motion affecting the project), the information and data available on geology and seismicity have been closely reviewed. In general, the work will proceed as follows:

- Compile, synthesize and review the available data for Santa Anita Dam, including geologic/seismologic data, hydrology/hydraulic data, material properties for foundation rock and dam concrete, monitoring data, and regional temperature data;

- Following the data compilation effort, identify data gaps with respect to geology/seismology, hydrology/hydraulics, and Alkali Aggregate Reactivity (AAR);

- Perform and present a preliminary linear-elastic seismic "reference" analysis with computer program ANSYS (i.e., no contraction joints included in the finite element model);

- Prepare the Analysis Plan (submitted November 18, 1999);

- Based on the results of this preliminary reference analysis, provide recommendations for further static and seismic stability analyses using both a linear-elastic finite element model (computer program ANSYS) and a non-linear finite element model with vertical contraction joints incorporated in to the model (computer program ADAP-NS);

- Plan and conduct a field and laboratory investigation program to evaluate the potential impact of AAR and joint strength on the static and seismic stability of the dam;

- Using updated material properties from the field and laboratory investigation, perform both of the above-cited linear-elastic and non-linear finite element stress analyses using computer programs ANSYS and ADAP-NS, respectively Provide comparative results and conclusions regarding the two analyses;

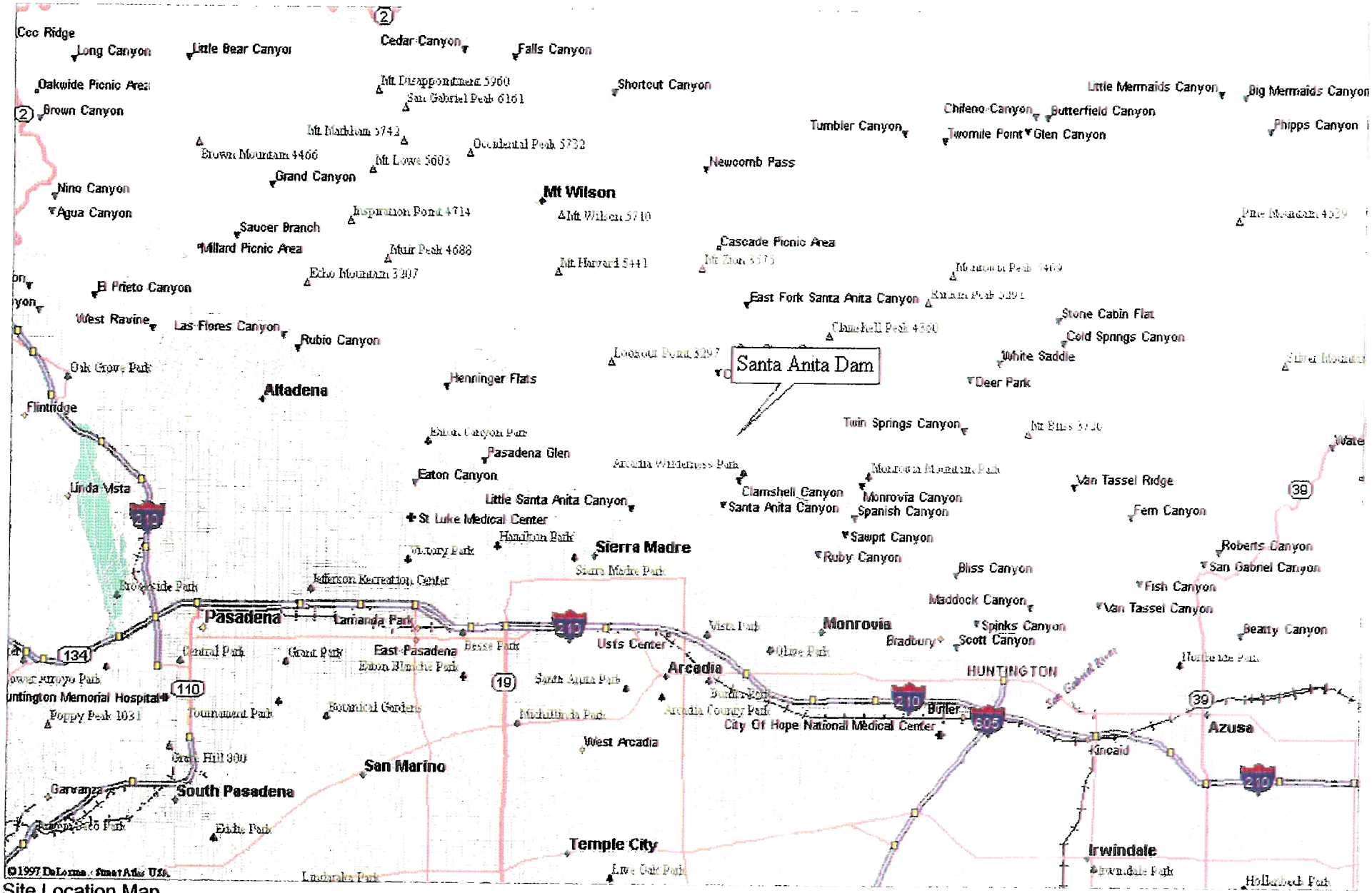
- Plan and conduct hydrologic/hydraulic analyses to assess the hydraulic adequacy of the existing spillways/outlet works; and

- Upon implementation of the above-cited investigations, prepare this Existing Condition Assessment Report which presents conclusions and recommendations regarding the hydraulic and seismic safety of the dam.

The second objective of Phase I is to develop alternative modification (rehabilitation) concepts to improve the safety conditions, and to develop such conceptual designs to the 30 percent design level. The deliverable documents for Phase I will include the following:

- Analysis Plan previously submitted November 18, 1999;
- This Existing Condition Assessment Report;
- Rehabilitation Concepts Report;
- Conceptual 30% Design Level Documents for the preferred rehabilitation concept; and
- Final Report which will provide narratives to summarize project development as well as the 30% design level documents.

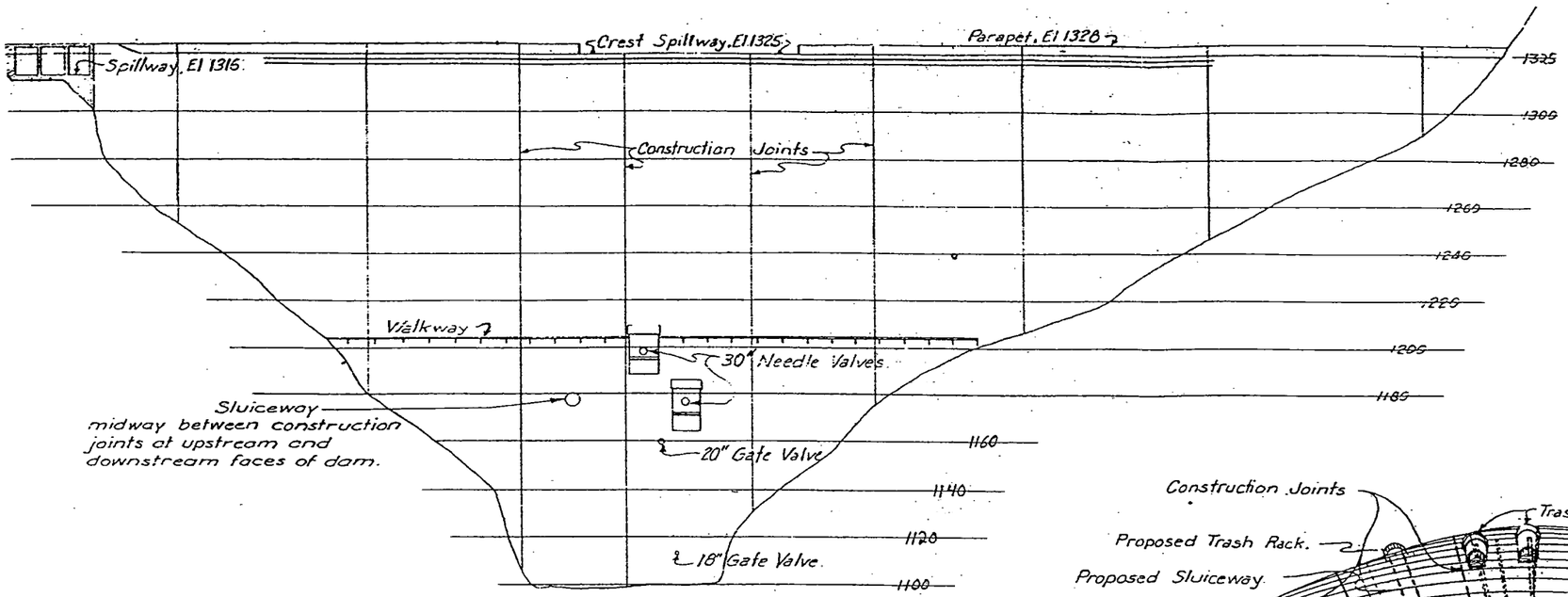
Following this introductory section, **Section Two** of this Existing Condition Assessment Report documents the data compilation effort, **Section Three** documents the results of investigations and analyses related to AAR, **Section Four** presents the DSOD-approved earthquake ground motions, **Section Five** presents the static and seismic structural reanalysis, **Section Six** presents the results of hydrologic / hydraulic reanalysis and **Section Seven** presents a summary of findings and conclusions. References are listed in **Section Eight**.



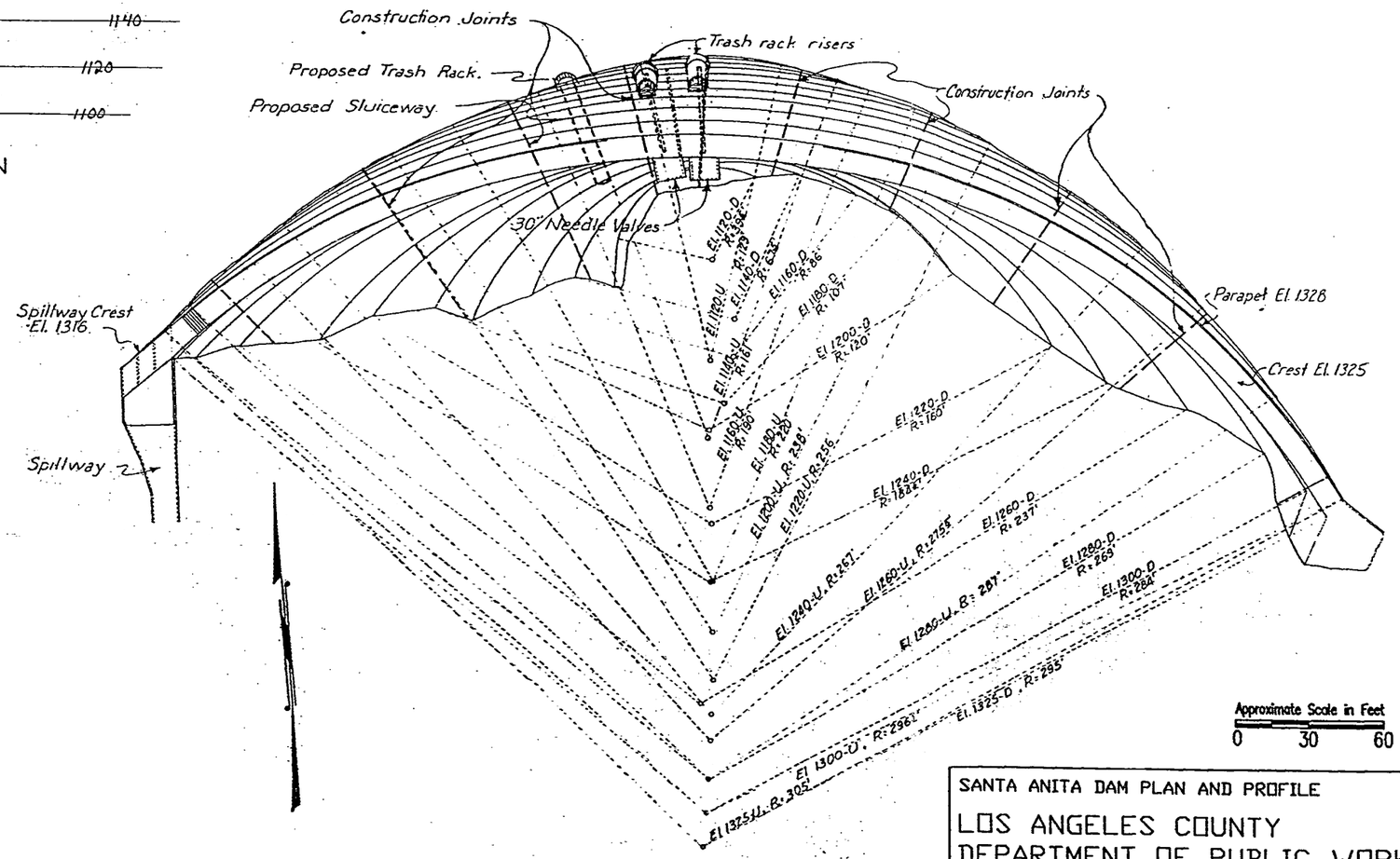
Site Location Map

HARZA

Figure 1-1



DOWNSTREAM ELEVATION



PLAN

HARZA
ENGINEERING COMPANY

Basemap provided by: LA Co. Flood Control District
"Santa Anita Dam - Alterations to Dam for Sluice Control Outlet" (1942)

SANTA ANITA DAM PLAN AND PROFILE

LOS ANGELES COUNTY
DEPARTMENT OF PUBLIC WORKS

OCTOBER 1999 Figure 1-2

HARZA Engineering Co. CONCORD, CALIFORNIA (925) 685-6275

4.0 EARTHQUAKE GROUND MOTIONS

The science and practice of seismology has made significant advances in the last 20 years such that the discussions regarding design response spectra and earthquake ground motions presented in the 1977 IECO report need to be updated. The purpose of this section is to present the methodology used to develop the target response spectra, evaluation and selection of reference ground motions, and modification of the reference motions to be used in the seismic dynamic re-analysis of Santa Anita Dam.

The reader should note that the reference ground motions previously submitted in the Analysis Plan dated November 18, 1999 have been superceded by the revised reference motions presented herein in accordance with DSOD requests and guidance given in a meeting held on December 20, 1999. These revised ground motions were submitted in letters dated January 21 and February 2 and 21, 2000, and were approved by DSOD in their letter dated March 17, 2000

4.1 Maximum Credible Earthquake

As previously stated in Section 2.0, the surface trace of the north-dipping Sierra Madre fault lies about 0.5 km south of the dam and the fault plane is located about 0.35 km beneath the dam. After considering several earthquakes, including a M8+ San Andreas event, it was determined that a M7 25 ± 0.25 Sierra Madre earthquake would produce the strongest shaking at the dam site, and therefore represents the maximum credible earthquake (MCE) for Santa Anita Dam.

The maximum magnitude of the Sierra Madre fault of 7.25 ± 0.25 was estimated by considering both potential rupture lengths associated with various segmentation models as well as slip per event data from a recent trenching study in Altadena. The Loma Alta trench in Altadena revealed evidence for large slip earthquake (Rubin and others, 1997). A minimum dip-slip displacement of about 3.5 m appears to have occurred in the last event and possibly as much as 5 m in each of the past two events The methods used to estimate maximum magnitude included: 1) seismic moment calculations that consider rupture area and average slip, and 2) the regressions of Wells and Coppersmith (1994) that predict magnitude given the parameters of rupture length, rupture area, maximum displacement, and average displacement.

4.2 Earthquake Ground Motions

The approach taken to generate the three maximum earthquake motions are to develop the 50th and 84th percentile target spectra and scale the recorded ground motions to the target. This method requires only minimal modification of the recorded motions and best preserves the motions as 'real' earthquakes, as opposed to other methods that generate synthetic accelerograms.

4.3 Target Response Spectra

The 84th, 50th, and 16th percentile target response spectra were developed to represent the maximum credible earthquake (MCE) on the nearby Sierra Madre fault for both the horizontal and vertical components of ground motion. Four recent spectral attenuation relations for rock site conditions of Campbell (1997), Abrahamson and Silva (1997), Sadigh et al (1997), and Idriss (1991) were averaged to construct the 16th percentile, 50th percentile, and 84th percentile target spectra shown in Figures 4-1, 4-2 and 4-3 for two horizontal and one vertical components of ground motion, respectively In the period of interest of approximately 0.15 second to 0.3 second for the dam, the average of the 84th percentile horizontal spectral accelerations range from approximately 3.5g to 3.7g, with the peak spectral acceleration at the 0.2 second period. In the case of the vertical target response spectrum (see Figure 4-3), the average 84th percentile vertical peak ground acceleration is approximately 1.3g, with a 84th percentile peak spectral acceleration of approximately 3.1g at the 0.1 second period.

Because the dam site is located on the hanging wall of the Sierra Madre fault, the hanging wall factor, included in the Abrahamson and Silva (1997) relation, was used to account for stronger motions above the reverse fault. The other attenuations used in this study do not consider the hanging wall effect. Because of the large magnitude (M7.25 ± 0.25) of the maximum earthquake, the close proximity to the dam (0.35 km), and the use of recent attenuation relationships that predict strong motions, the ground motion response spectra in the range of 50th to 84th percentile target spectrum represents a conservative motion.

4.4 Evaluation and Selection of Reference Motions

After an extensive review of recorded near-field motions, and following the guidance provided by DOSD during our meeting of December 20, 1999, three reference motions were selected for modification into the three evaluation earthquakes. While no recorded earthquake motion exists that perfectly matches the characteristics of the target response spectrum or the magnitude, distance, site conditions, and focal mechanism parameters of the maximum earthquake, an attempt was made to satisfy as many of these conditions as possible.

More than 20 recorded near-field motions were reviewed and evaluated to determine which records would make the best candidates for scaling to the target spectrum. This was accomplished by comparing the recorded spectra with the target spectrum for the period range critical to the structure and considering the parameters of magnitude, distance, site conditions, and mechanism.

Many of the near-field motions had spectral shapes that were not suitable for scaling to the target spectrum because their spectral shapes were significantly higher in the longer periods (2 ½ second) and lower in the shorter periods that contained the fundamental period of the dam. Scaling these records up to the target (in the period range of the dam) would produce motions that were unrealistically strong in the long periods. Other records were not used because they contained large peaks or valleys in their response spectra in the period range critical to the dam, estimated to span from approximately 0.15 to 0.3 seconds.

In a meeting held with DSOD on April 6, 1999, consensus was obtained that reasonable spectral-compatible scaling of the following earthquake records may be utilized as the basis for design level earthquakes and respective response spectra:

- 1978 M7.4 earthquake recorded in Tabas, Iran;
- 1994 M6.7 Northridge, California earthquake recorded at Sylmar Converter Station East; and
- 1994 M6.7 Northridge earthquake recorded at the Sepulveda VA Hospital.

However, after the December 20, 1999 meeting and after further review of the records, it was determined that the 1994 Northridge event recorded at Sepulveda VA Hospital would not be used. Therefore, design level earthquakes were developed from the 1978 Tabas record and the 1994 Northridge event recorded at Sylmar Converter Station East as described below

4.5 Modification of Reference Motions

A description of each of the modified ground motions for development of design level earthquakes is provided below

4.5.1 Earthquake 1 (EQ1)

EQ1 is a modified version of the original M7.4 1978 Tabas record. This design level earthquake was developed to spectrally match the 84th percentile target response spectra by Dr Norman Abrahamson using his non-stationary spectral matching program RSPMATCH as documented in Abrahamson (1999). This program is based on methodology presented by Kaul (1978) and Lilhanand and Tseng (1987,1988) with some modifications that help preserve the non-stationary character of the reference ground motion for a wider range of time histoires. In this method, "minimal modification" is made to the recorded time history in the time domain by adding wavelets to the reference time history This method has good convergence properties and in most cases preserves the non-stationary character of the reference time history.

Figures 4-4 through 4-7 present response spectra and the acceleration time-histories of the two horizontal and one vertical components of EQ1 This design level earthquake as presented herein is the same EQ1 motion that was used in the "reference analysis" presented in Section 4.0 of the November 18, 1999 Analysis Plan.

4.5.2 Earthquake 2 (EQ2)

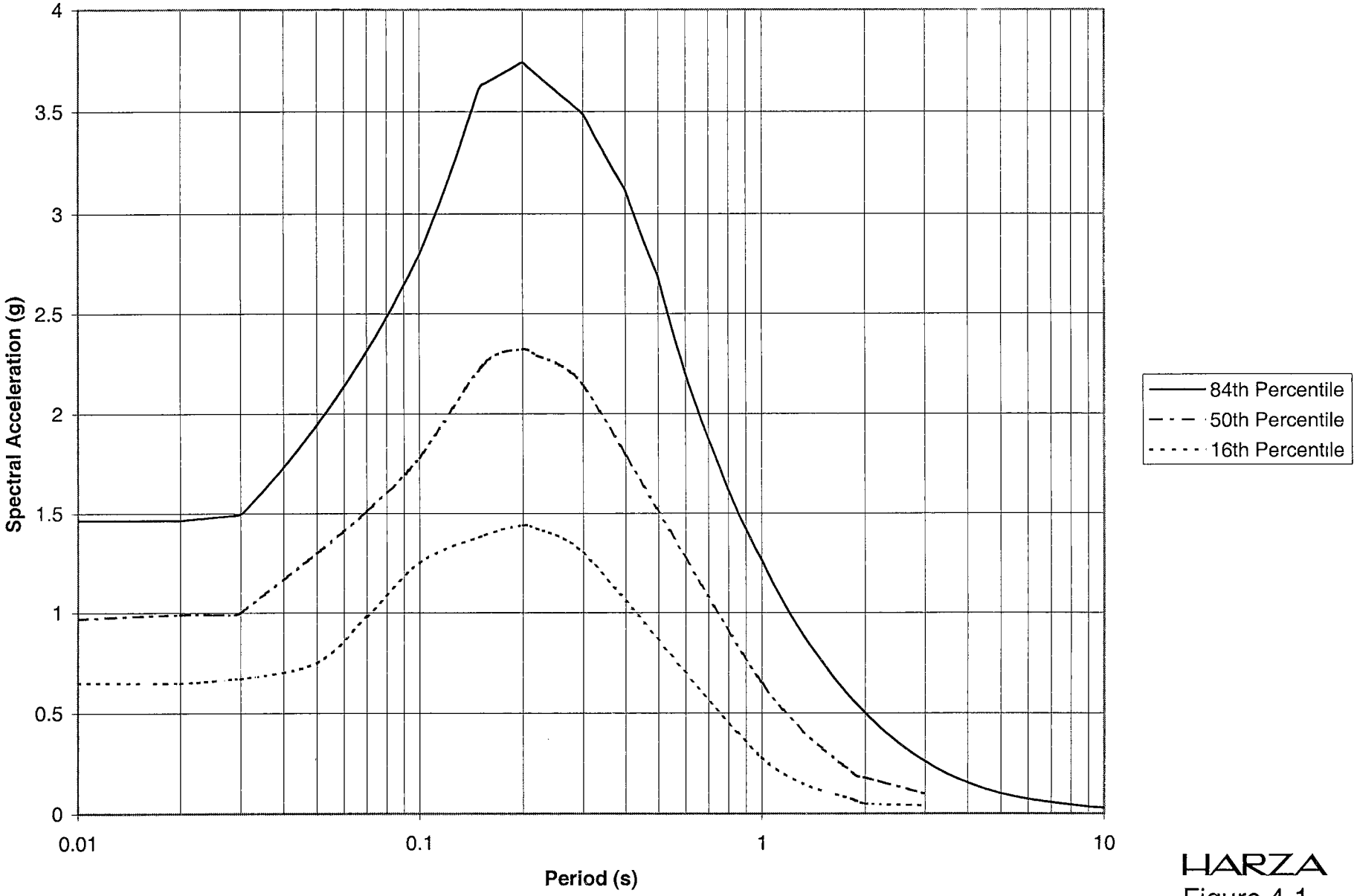
EQ2 was produced by "simple scaling" of the original M7.4 1978 Tabas earthquake records. This was done applying different scaling factors (1.0, 1.1, and 1.2) to the recorded ground motion

time history and subsequently calculating and plotting of the response spectra of each component. These response spectra were then compared to the 50th and 84th percentile target response spectra. The EQ2 response spectra are presented in Figures 4-8 through 4-10. Comparison of the spectra for scaled motions with the 50th and 84th percentile target spectra indicate that a scaling factor of 1.2 would provide for the spectra to be reasonably within the 50th and 84th percentile of the target. Accordingly, the scaled (1.2 factor) time histories for three components of ground motion are presented in Figure 4-11.

4.5.3 Earthquake 3 (EQ3)

EQ3 was produced by "simple scaling" of the original M6.7 1994 Northridge earthquake recorded at Sylmar Converter Station East. The process of selection and production of the EQ3 time-history was the similar to that of EQ2. For EQ2, scaling factors of 2.9, 2.1 and 2.4 were found to be appropriate for the cross-canyon (x), canyon (y) and vertical (z) directions of ground motion, respectively The response spectra and scaled ground motion time-histories for EQ3 are presented in Figures 4-12 through 4-15.

Santa Anita Dam Target Spectra
Cross-Canyon (x)



HARZA

Figure 4-1

Santa Anita Dam Target Spectra
Canyon Parallel (y)

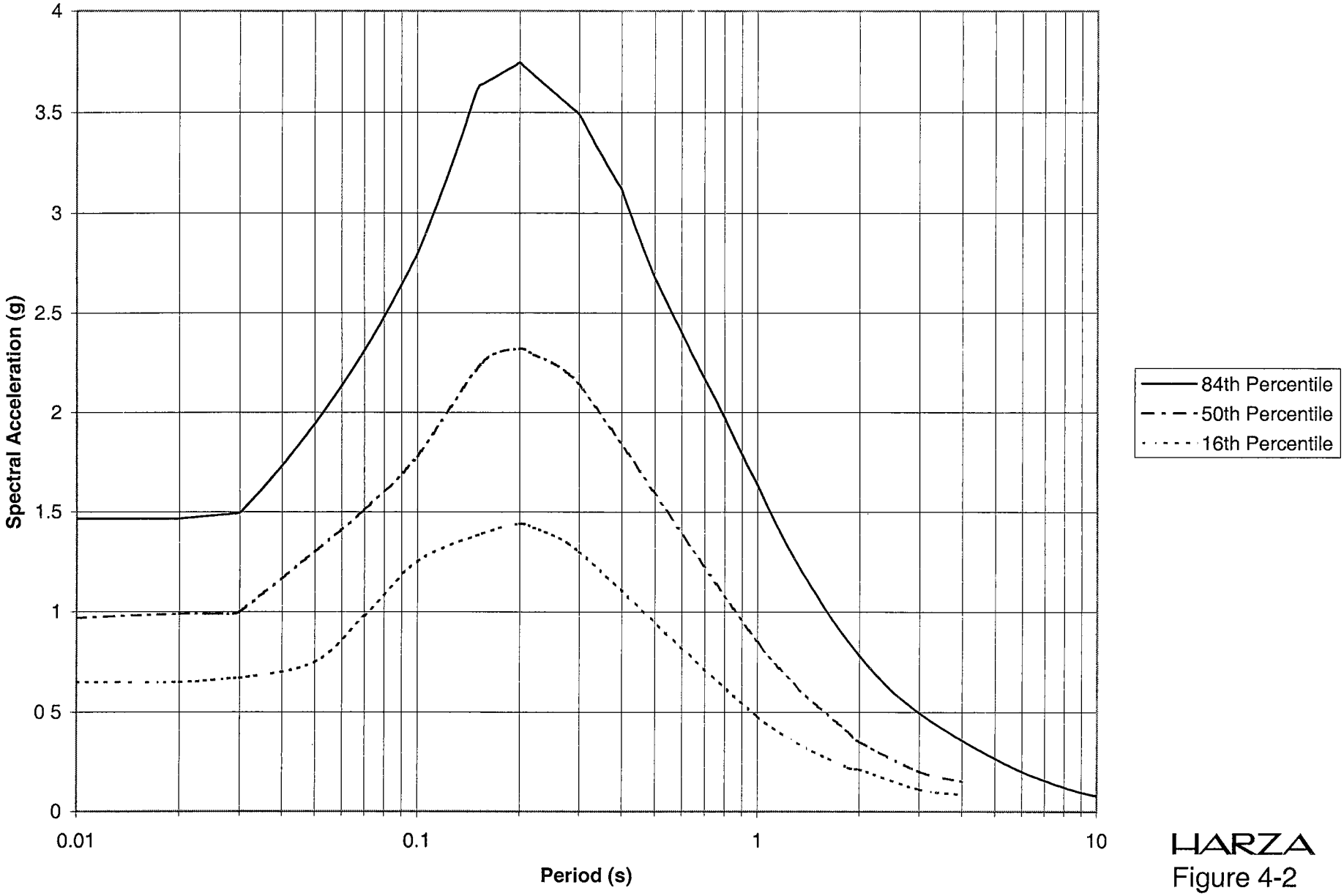


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Figure 4-2

Santa Anita Dam Target Spectra
Vertical (z)

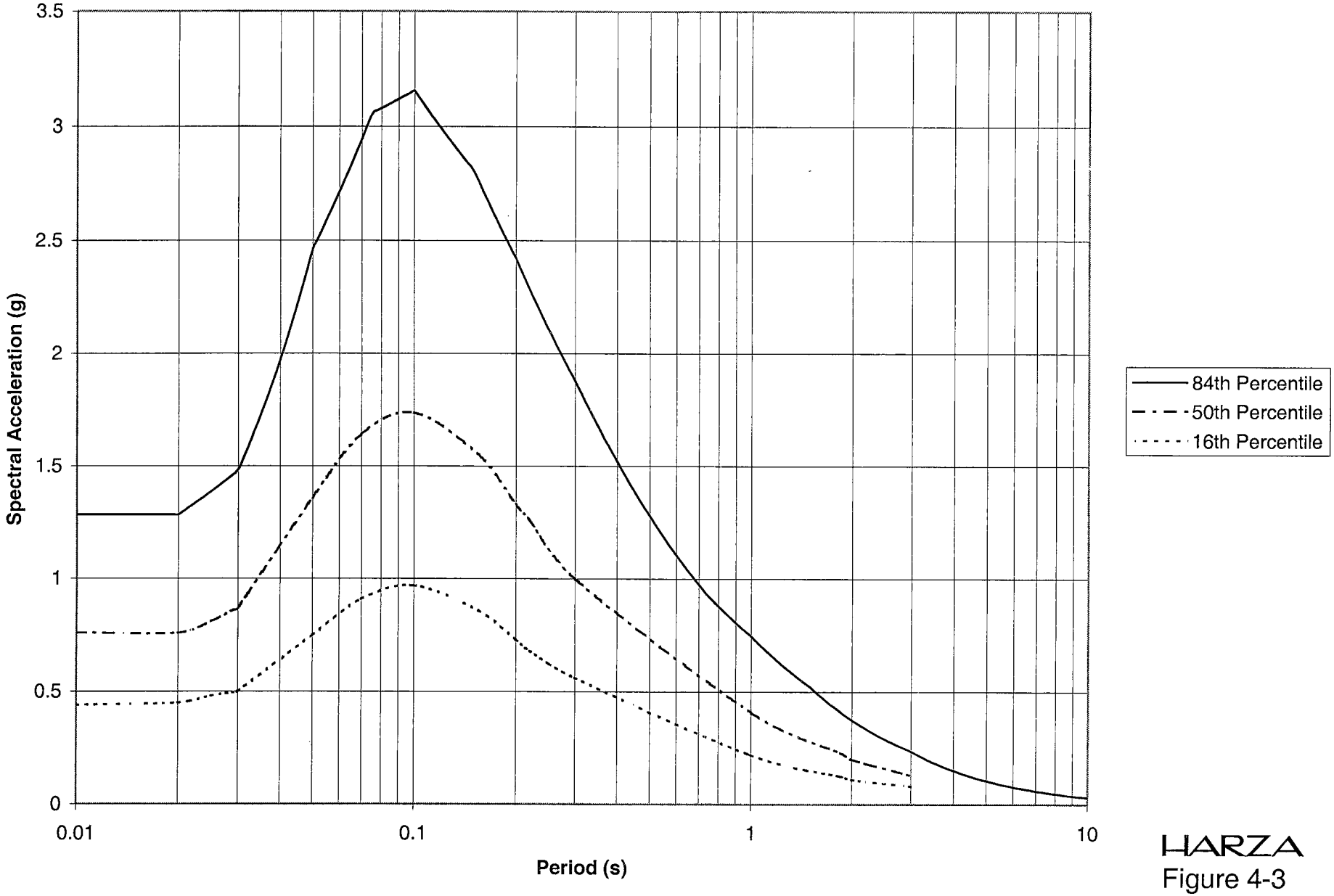

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Figure 4-3

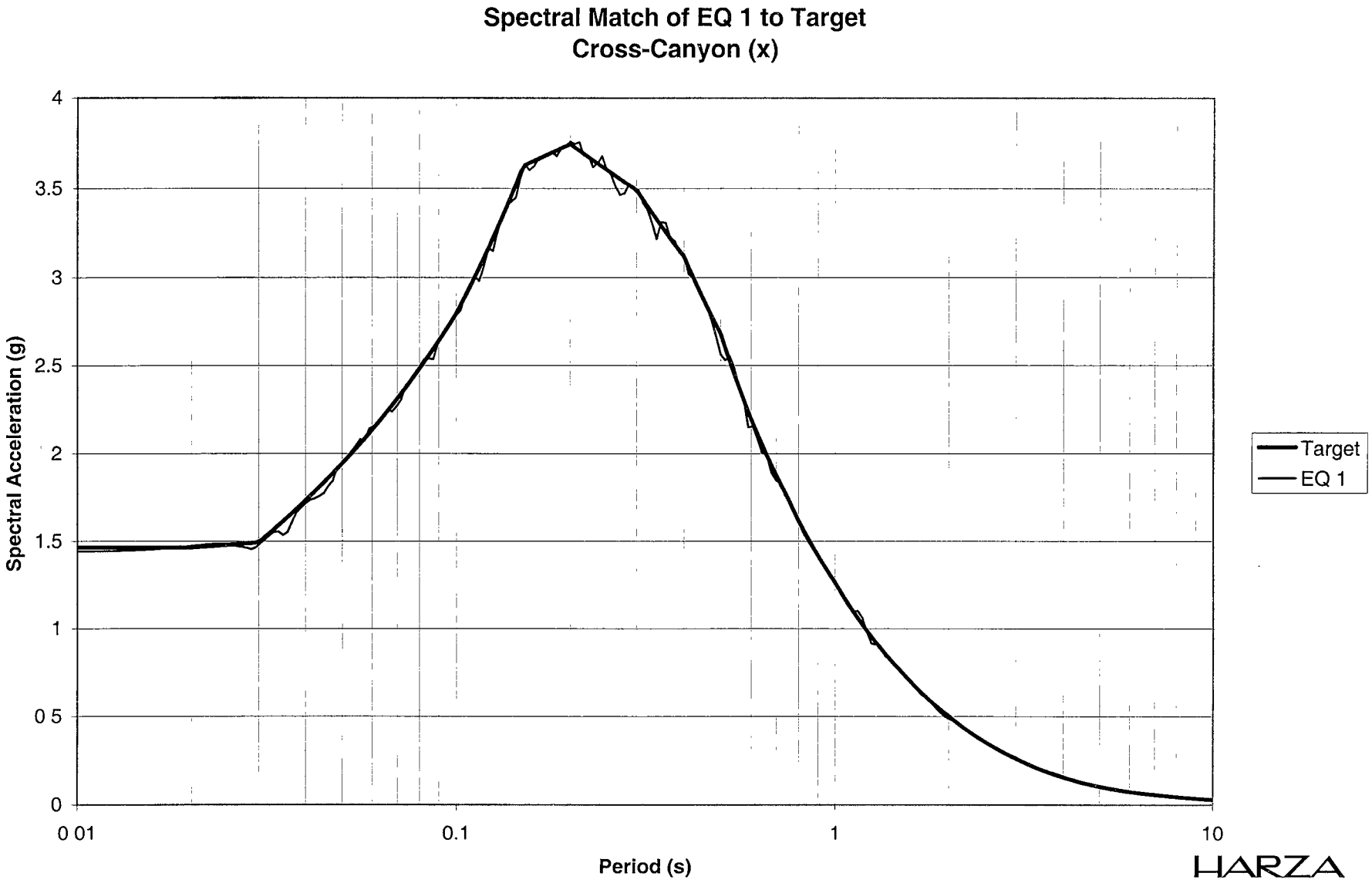

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Figure 4-4

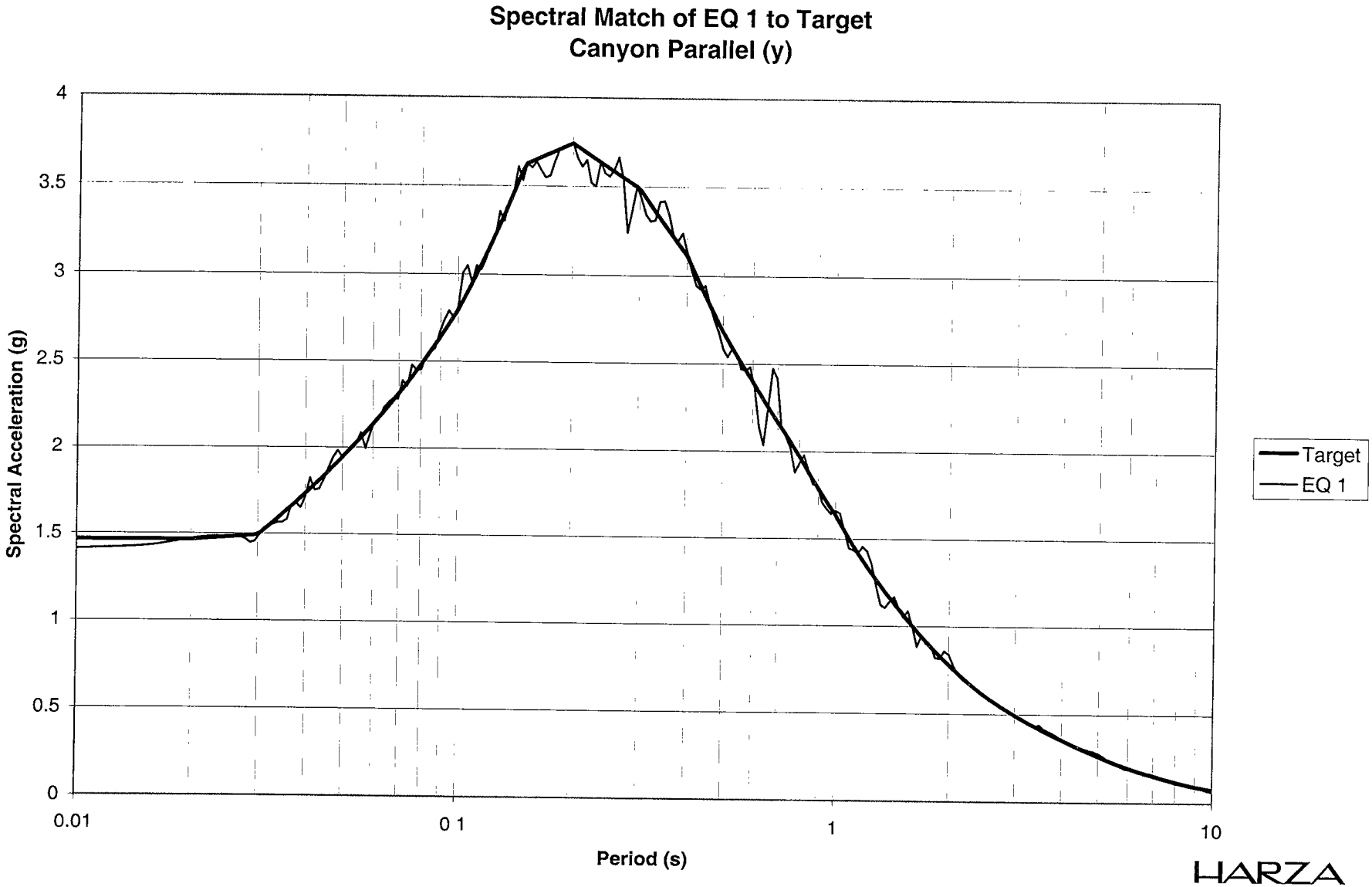

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Figure 4-5

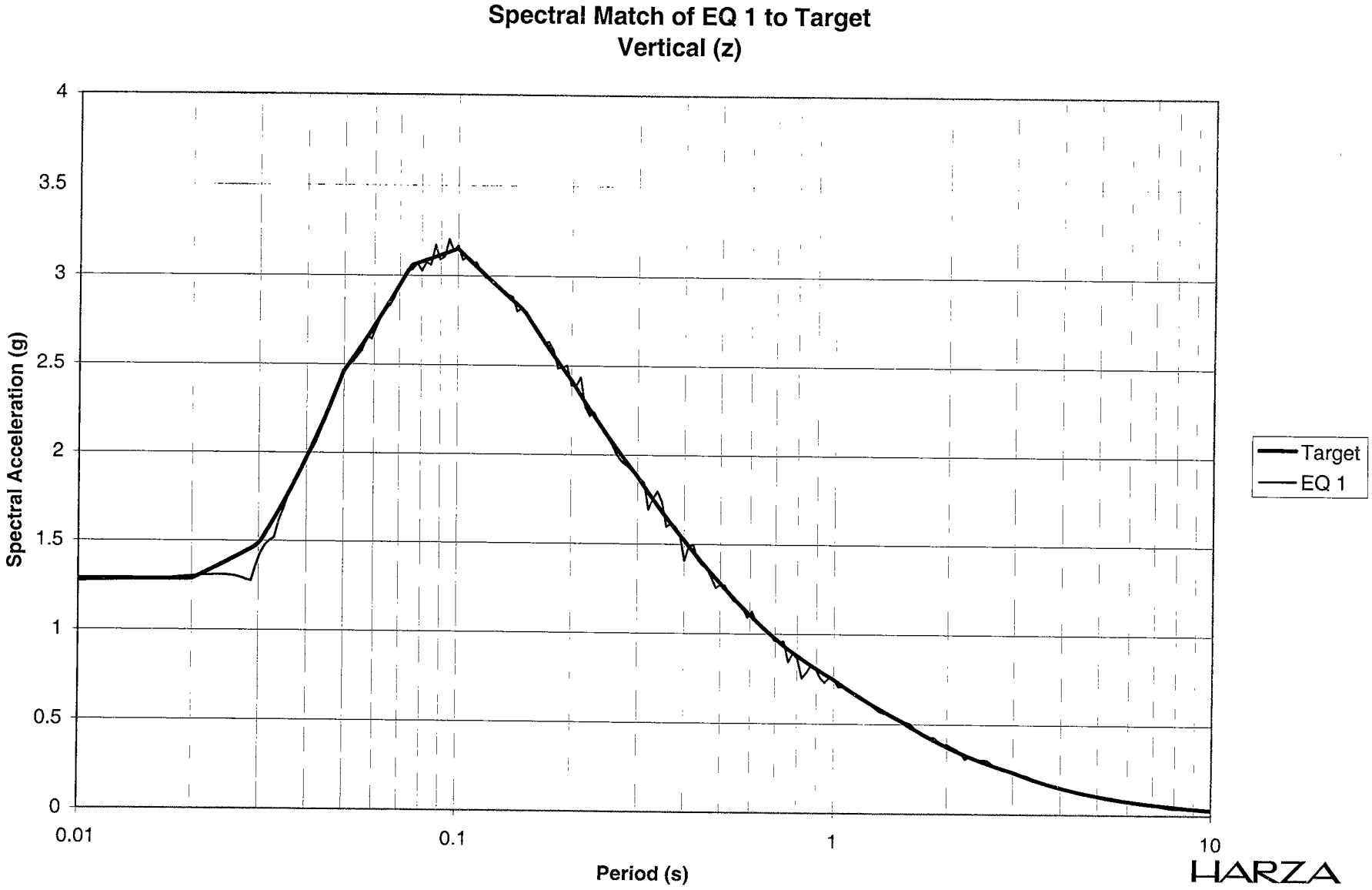

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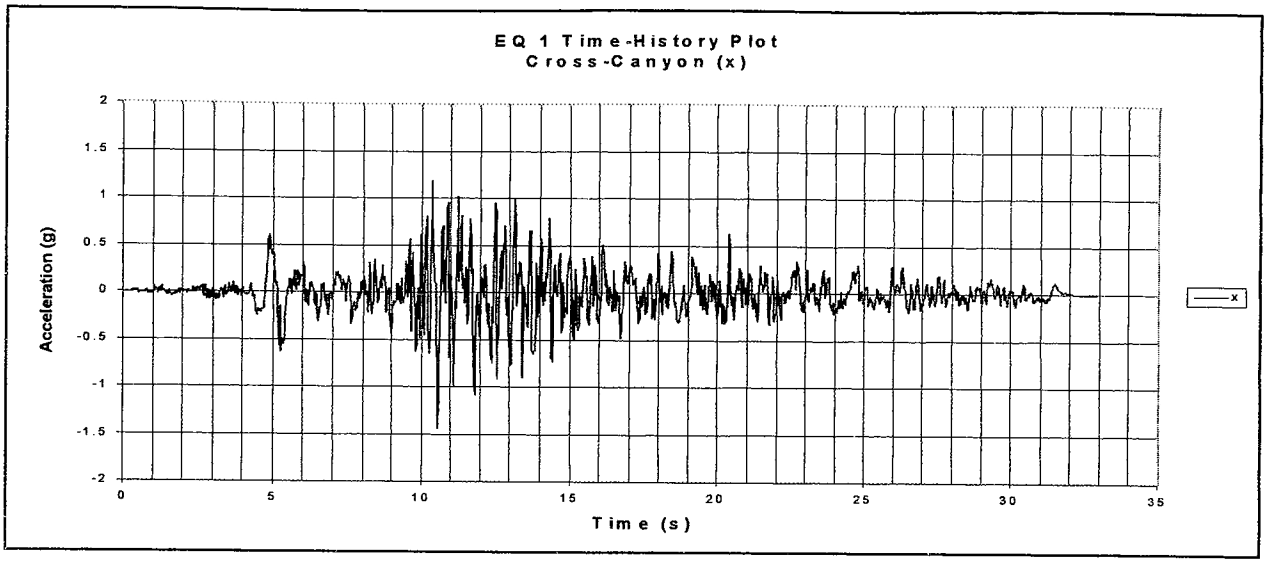
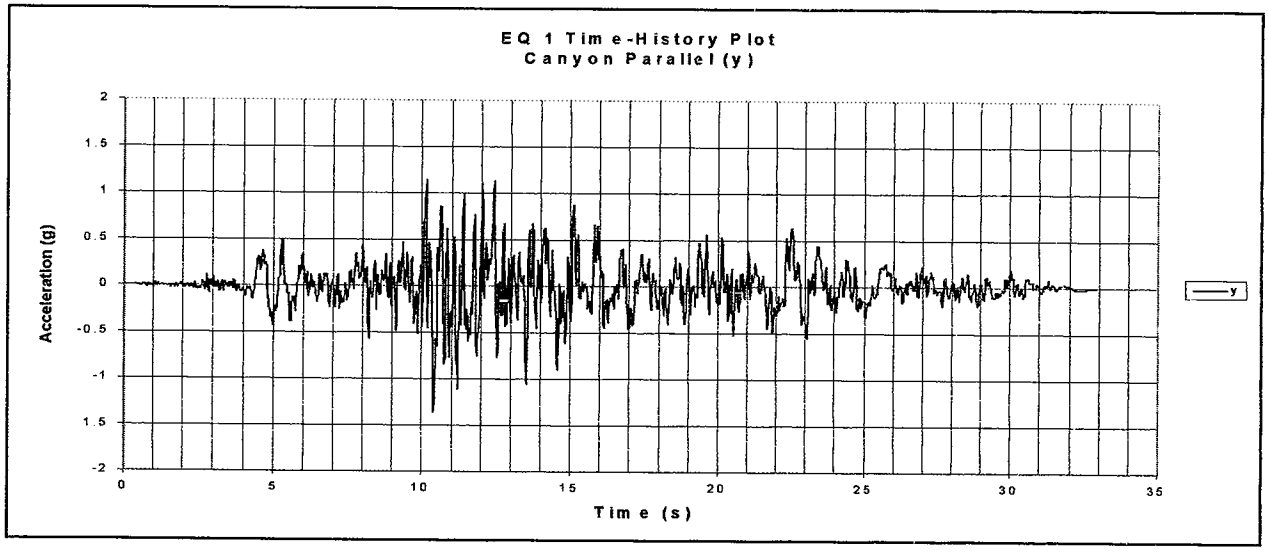
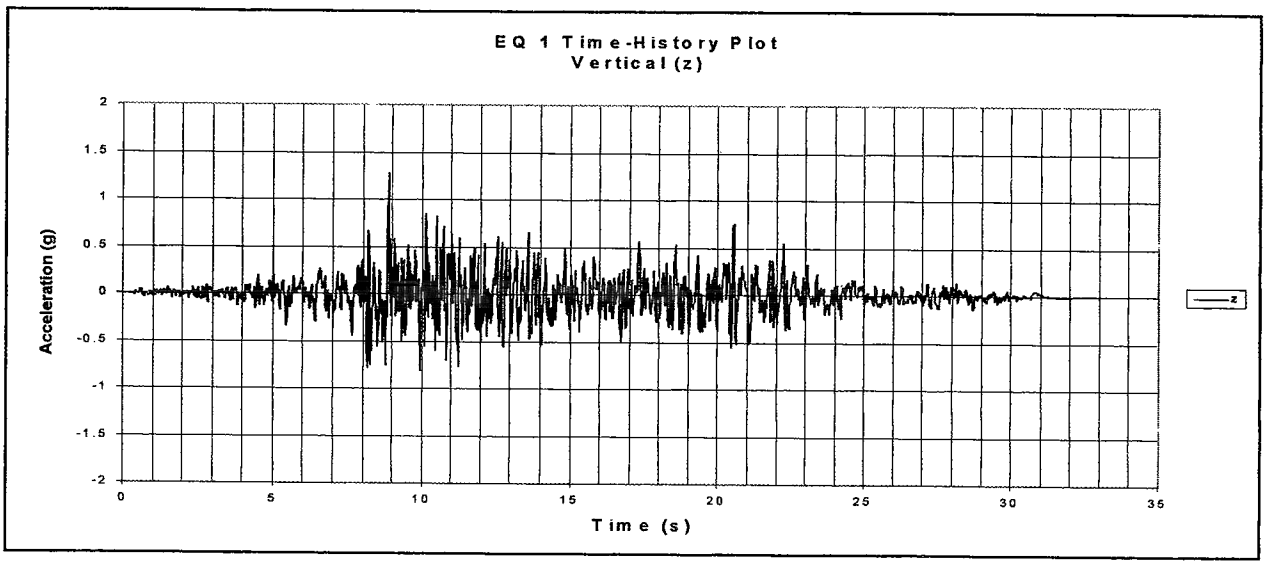
Figure 4-6

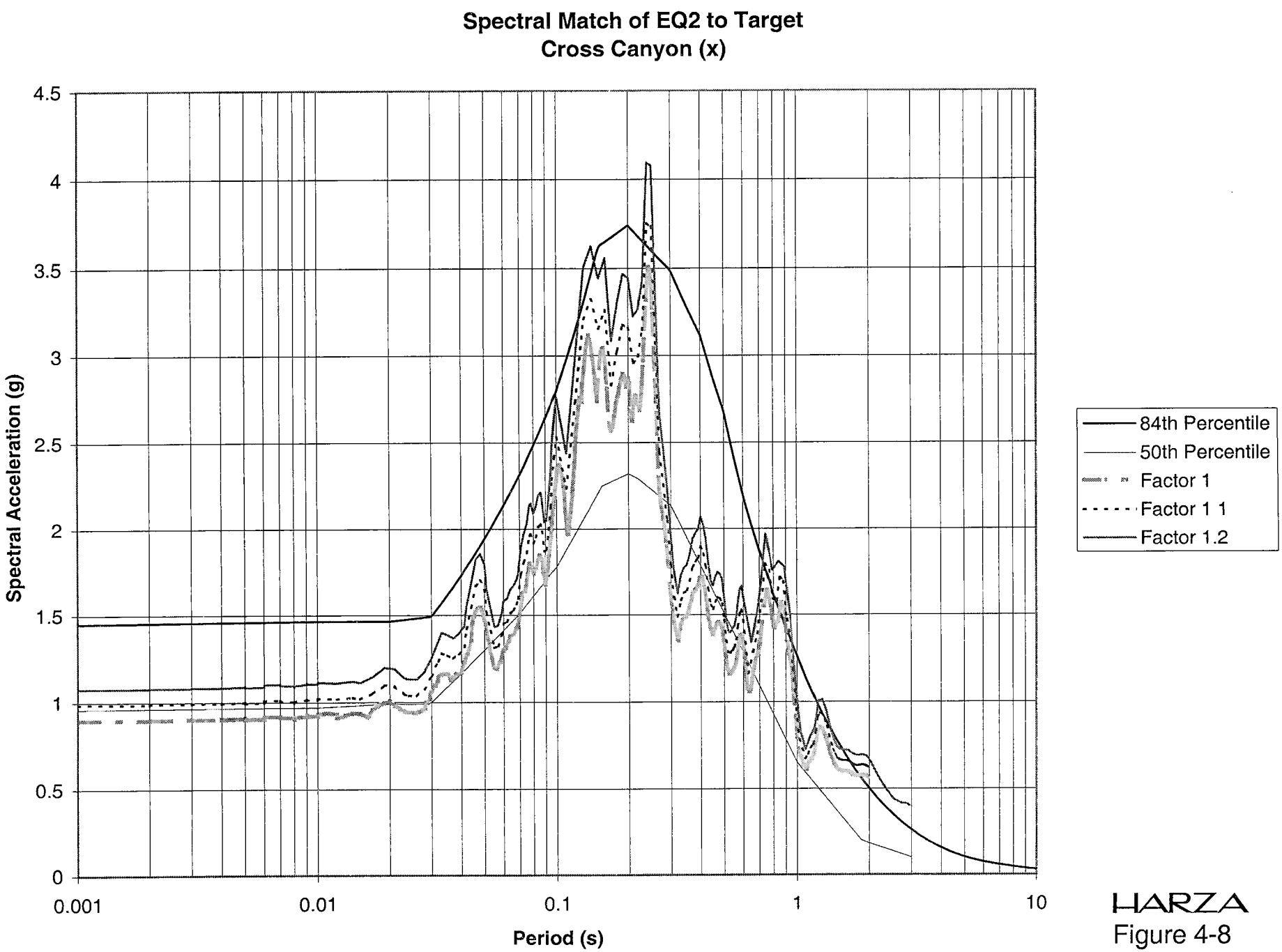






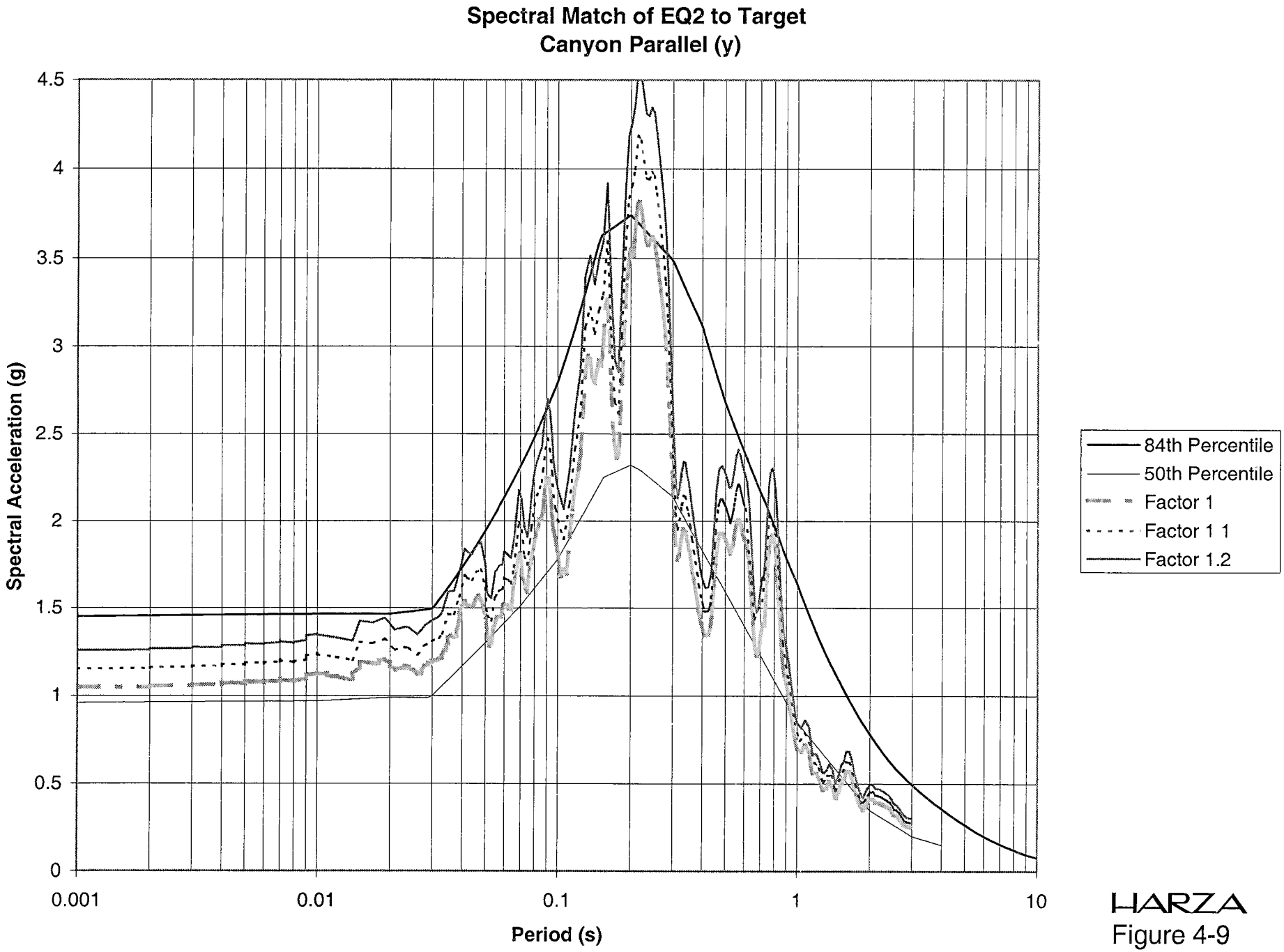
HARZA

Figure 4-7



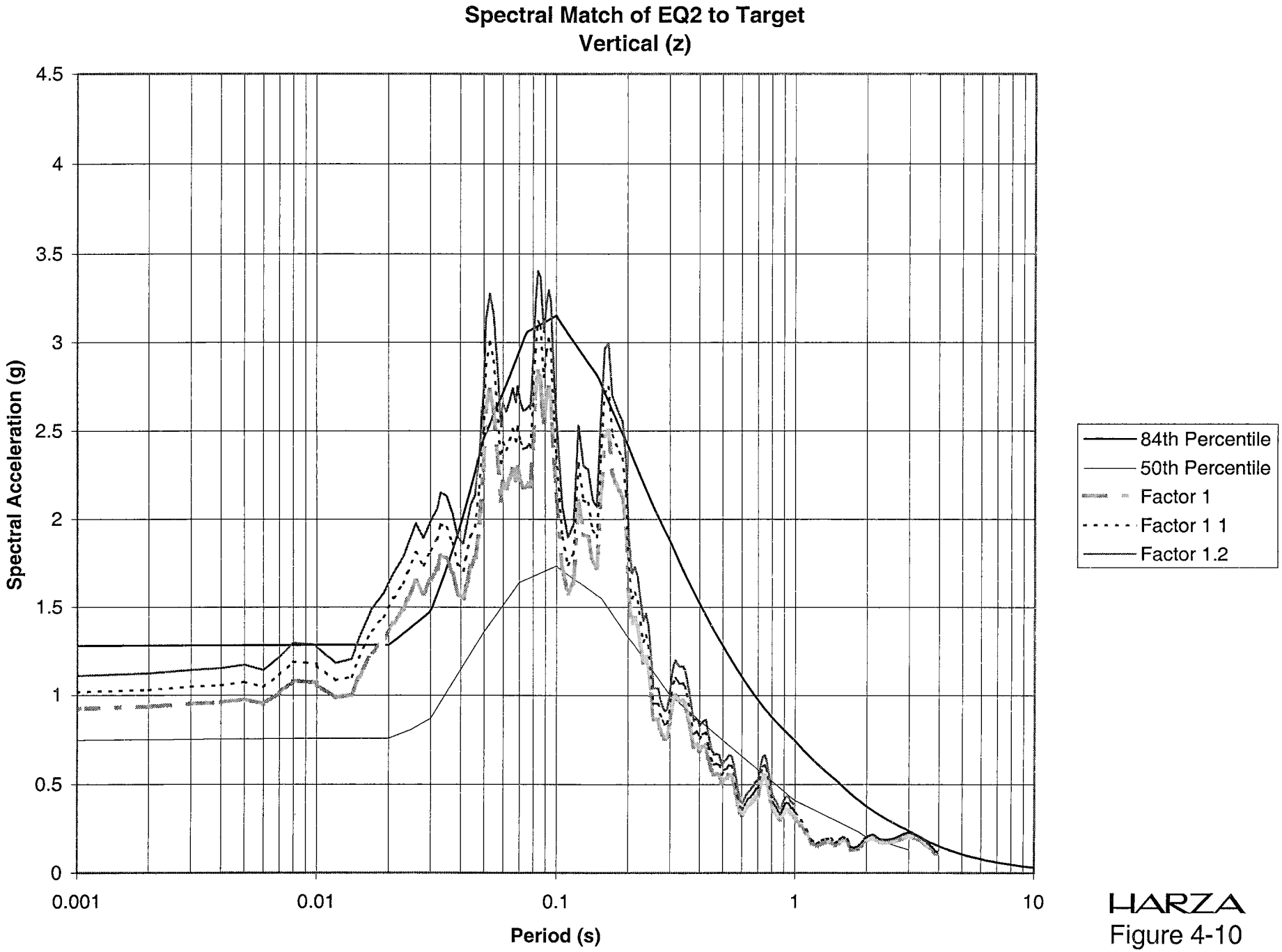
HARZA

Figure 4-8



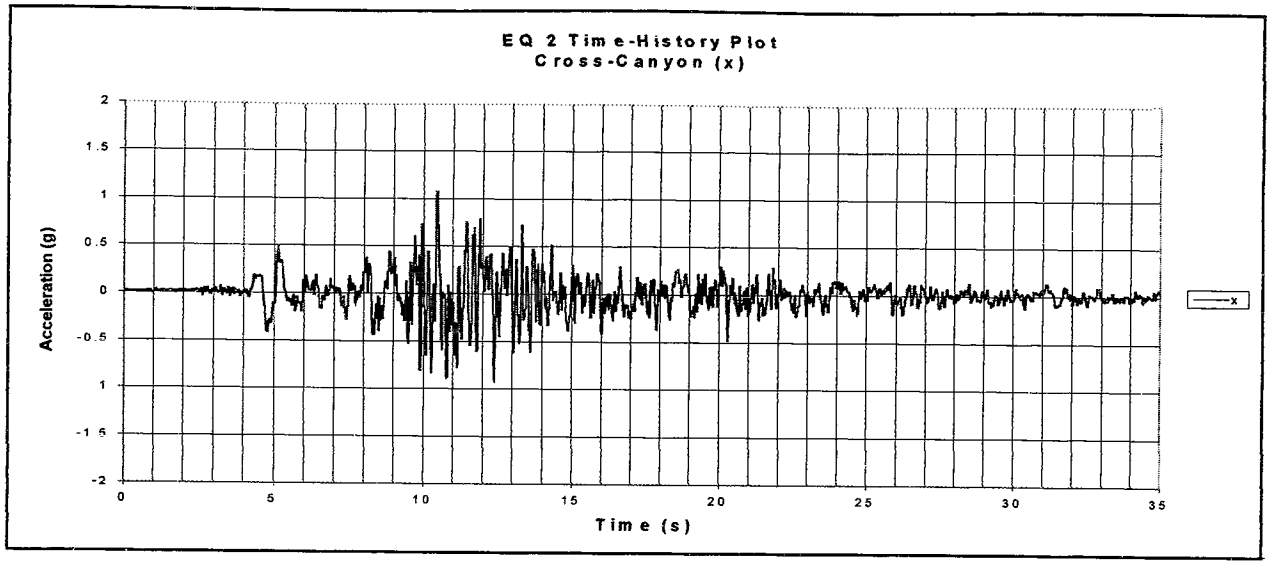
HARZA

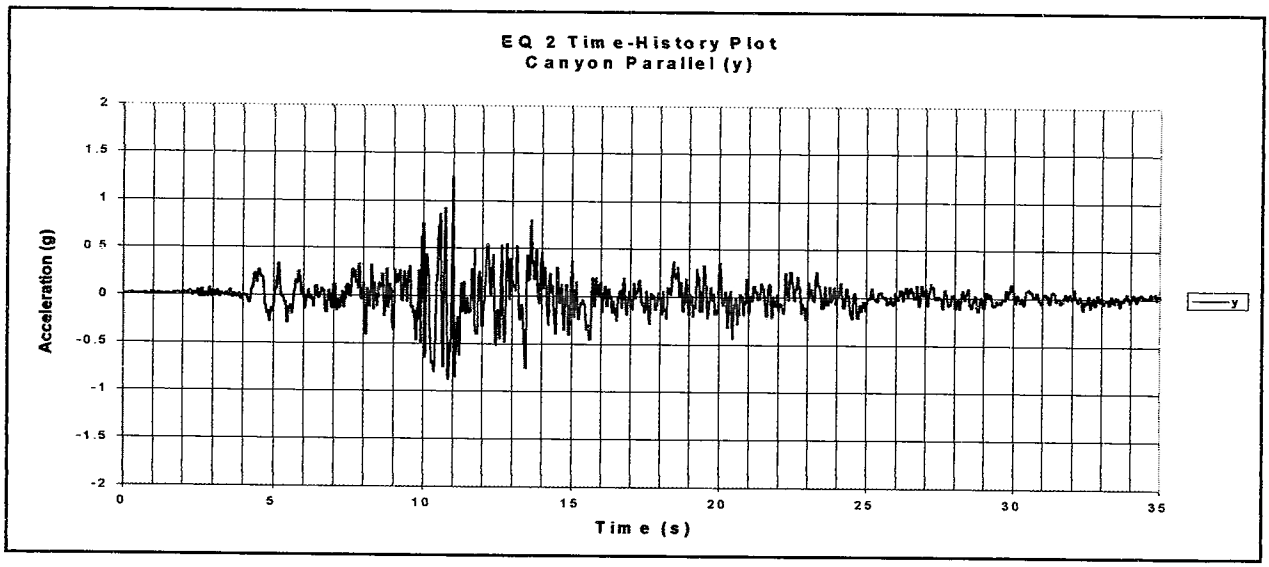
Figure 4-9

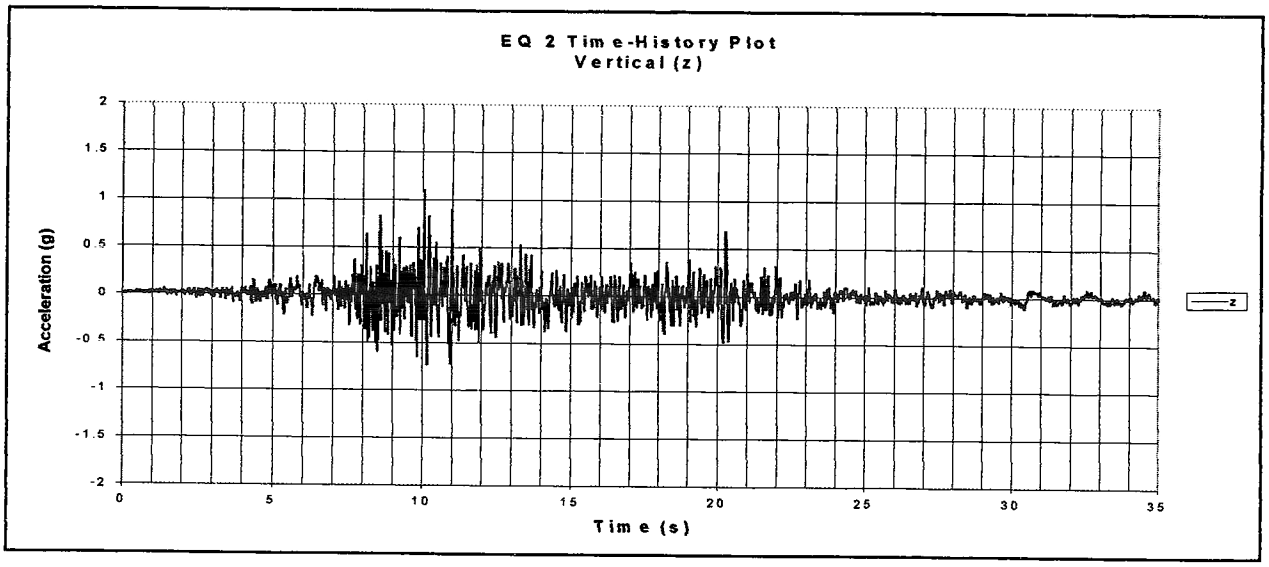


HARZA

Figure 4-10



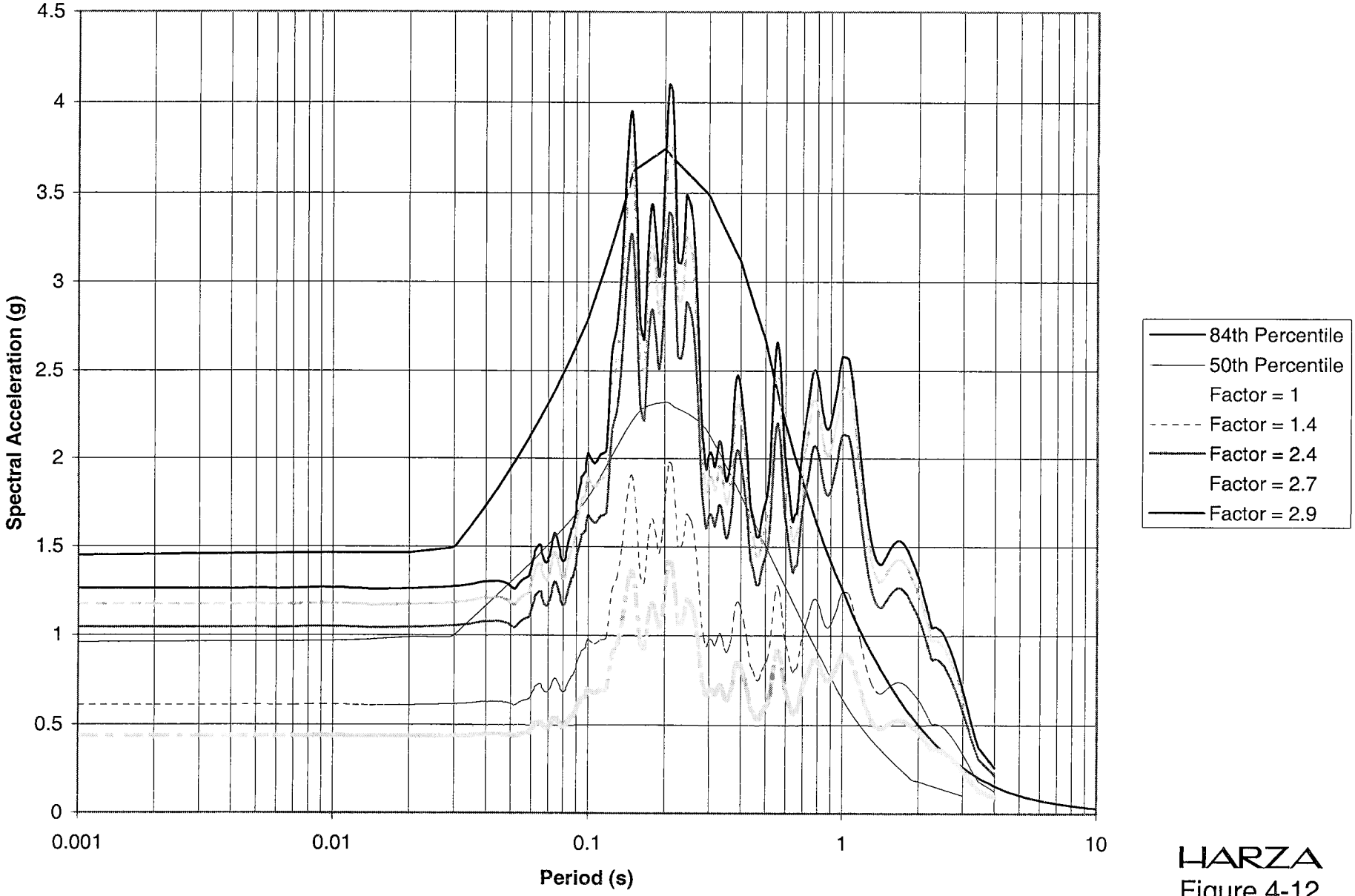




HARZA

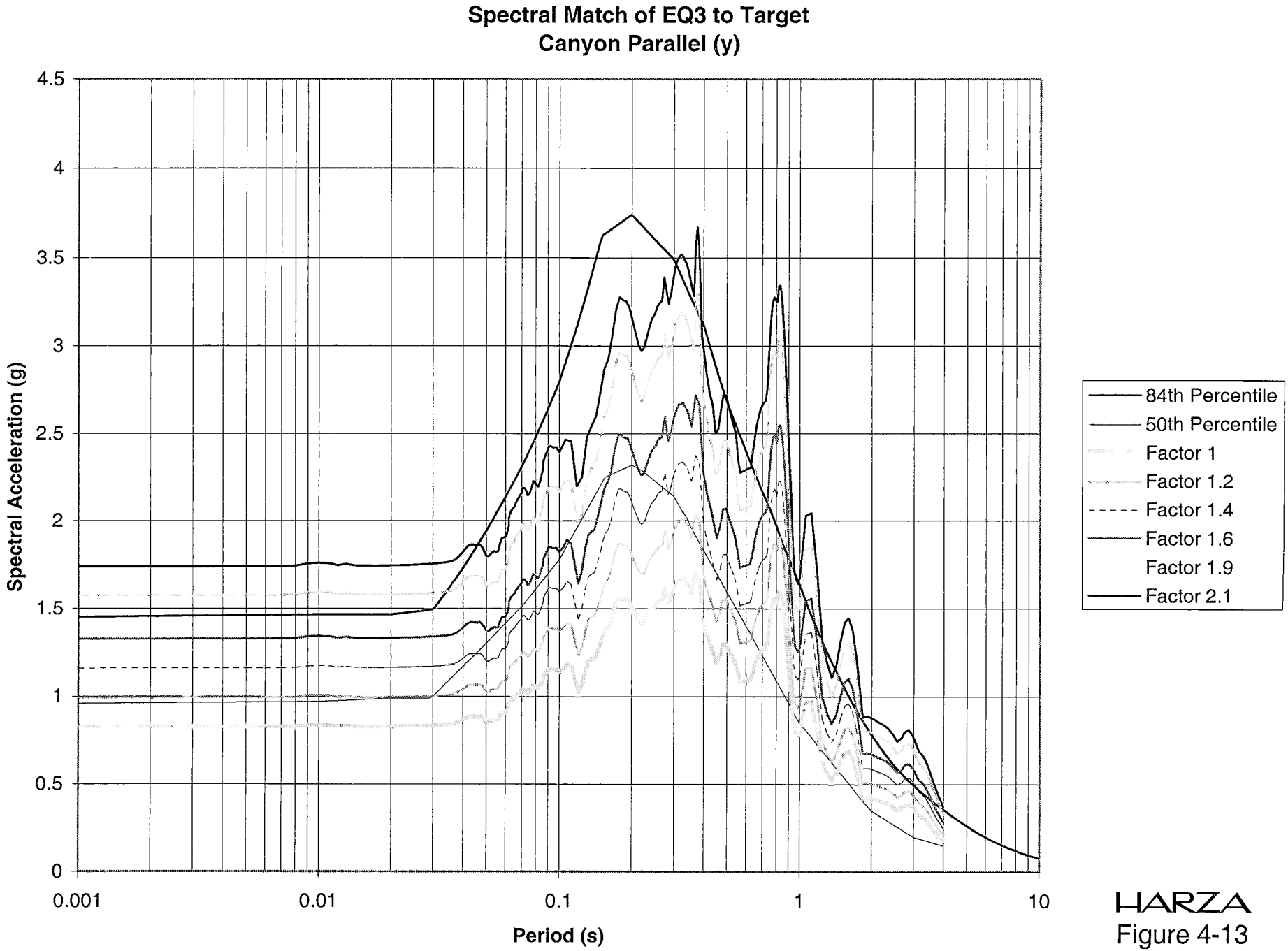
Figure 4-11

Spectral Match of EQ3 to Target
Cross-Canyon Direction (x)



HARZA

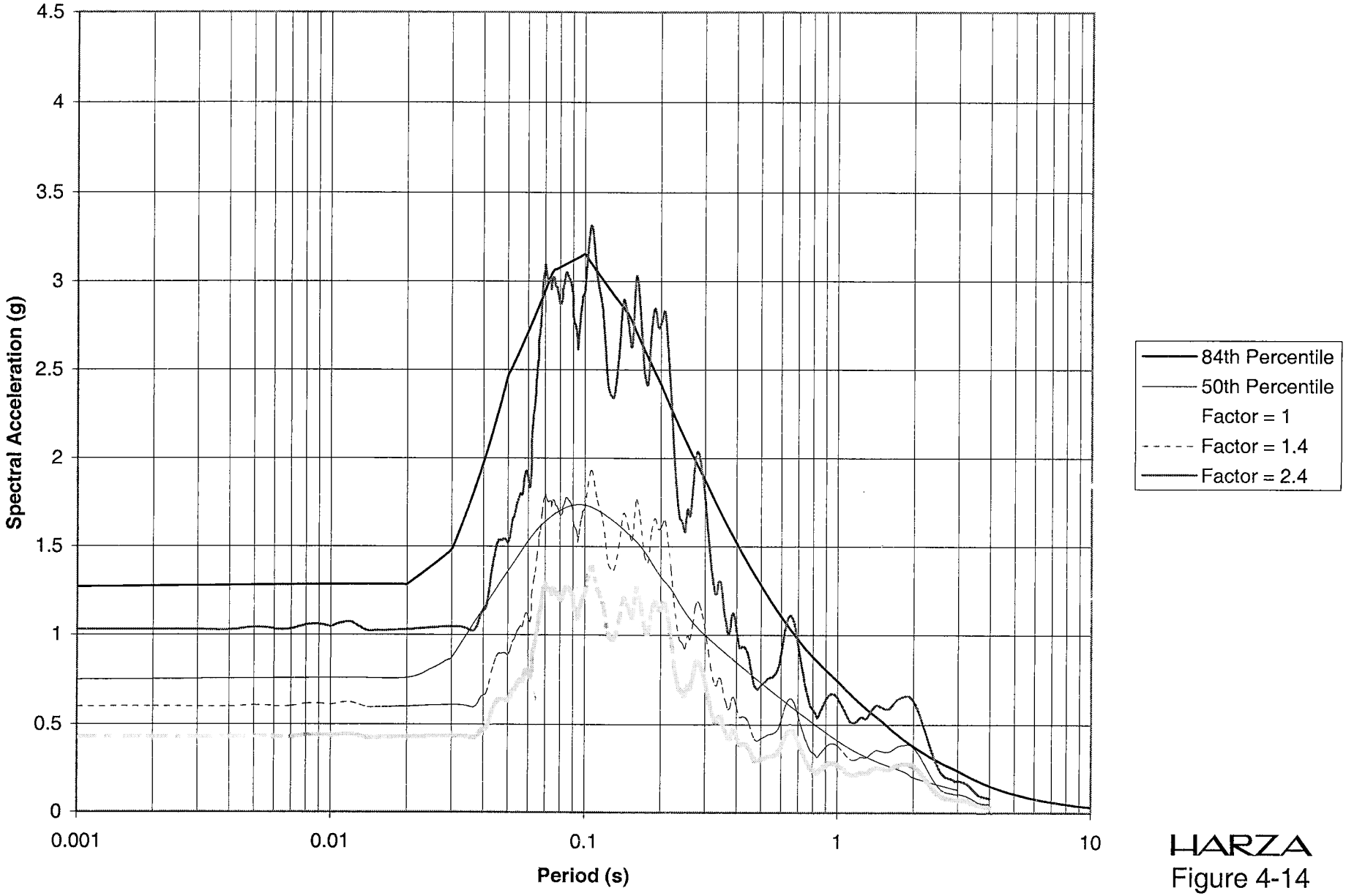
Figure 4-12



HARZA

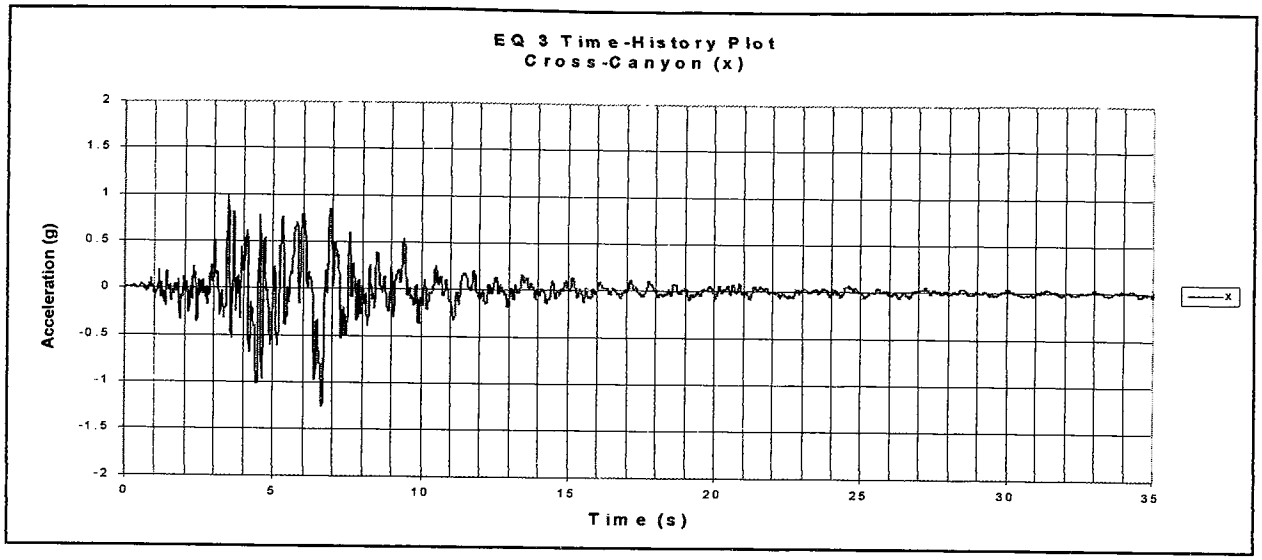
Figure 4-13

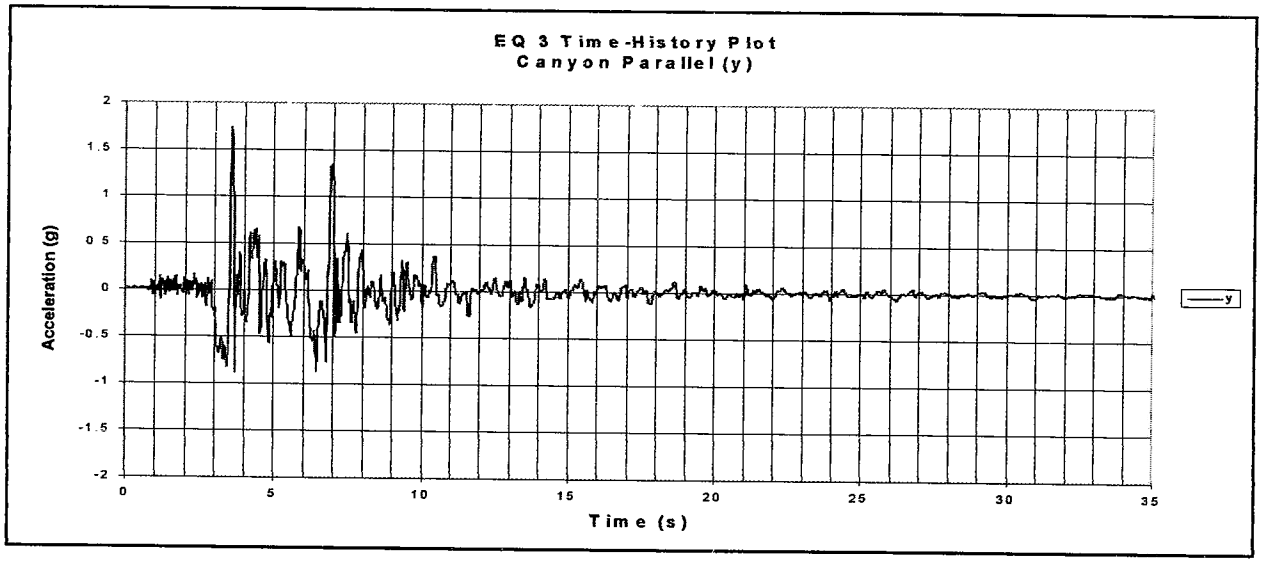
Spectral Match of EQ3 to Target
Vertical (z)

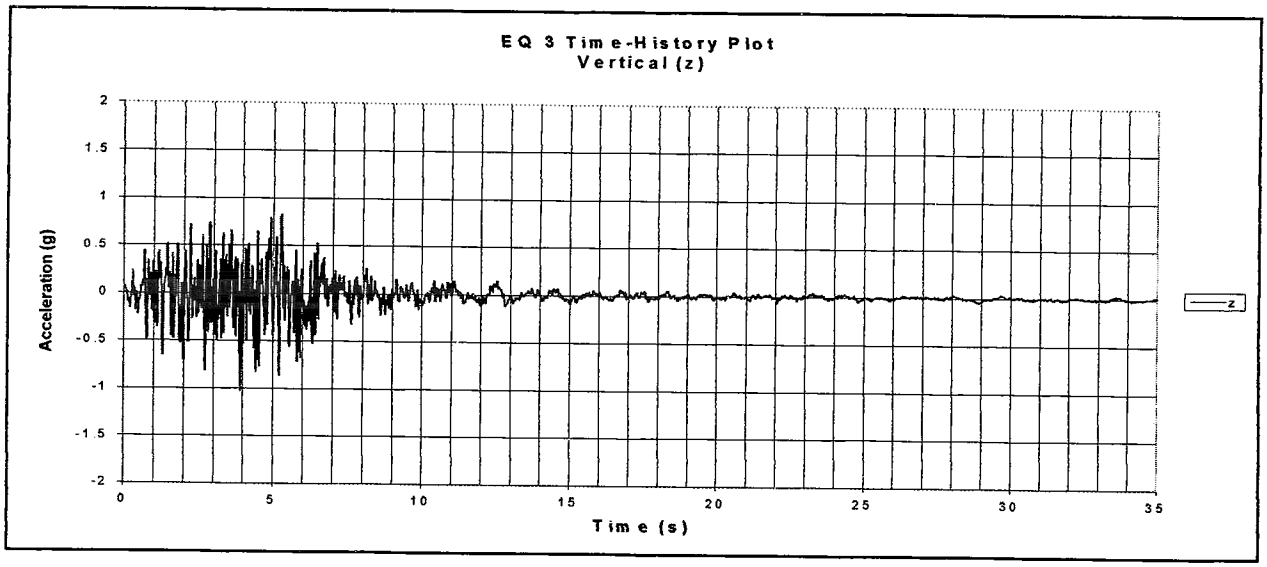


HARZA

Figure 4-14







HARZA

Figure 4-15

SECTION FIVE

5.0 STATIC AND SEISMIC STRUCTURAL REANALYSIS

This section presents the findings of the static and dynamic finite element analyses of Santa Anita Dam. Two computer programs were used to model linear-elastic (no vertical construction joints in the model) and non-linear elastic (9 vertical construction joints included in the model) analyses of Santa Anita Dam. The purpose of these analyses are to provide an indication of locations where the dam may be potentially overstressed during the MCE.

The finite element models, material properties, modal analyses, hydrodynamic effects, results and conclusions of the static and seismic analyses are summarized below.

5.1 Finite Element Models

As previously stated, two different finite element modeling programs were used to model the existing dam. A description of each computer program is given in the following paragraphs.

ADAP-NS is a state-of-the-art finite element method computer program, which was developed specifically for arch dam analysis under seismic conditions. The analysis method used by this program explicitly includes joint opening and closure during a seismic event. This allows one to address assumptions regarding the behavior of the dam when the tensile capacity of the joints are reached or exceeded. The model explicitly performs the load redistribution that occurs when joints open during a seismic event, thus releasing tensile stresses. ADAP is essentially the same as ADAP-NS, except that it is not capable of modeling joint opening and closing.

ANSYS is a start-of-the-art commercially available general finite element method program, which is widely used as a research and development tool in a number of industries, including a variety of civil/structural engineering applications (including dams). As a commercially available program, the pre- and post- processing capabilities in ANSYS enable the designer to modify an existing model and process the results without the difficulty generally attributable to large finite element models. This fact, coupled with its extensive capabilities, makes it an attractive program for use in analyzing various seismic strengthening options.

These two programs were utilized side by side to calibrate the models, compare results of seismic analyses performed by each program with the joints closed, and then to utilize the non-linear capabilities of ADAP-NS with the joints open. Harza attempted to match the models as closely as possible, however, due to the limitations of each software package, the models could not be matched exactly.

5.1.1 Linear-Elastic Modeling with Computer Program ANSYS

For the static and linear-elastic dynamic analyses, Harza developed finite element models using both computer programs ANSYS and ADAP under the assumption that the vertical joints would be closed and grouted. Both ADAP and ANSYS were used for linear-elastic modeling so that results could be compared prior to using ADAP-NS for non-linear elastic modeling (i.e., vertical construction joints open).

The geometry of the dam and rock foundation was digitized and imported into ANSYS from existing drawings. The automated meshing capabilities of ANSYS were then utilized to create the finite element model shown in Figures 5-1 and 5-2. This model consists of 3213 elements and 10272 nodes. The main arch of the dam and most of the rock foundation utilizes solid hexagonal ("brick") elements, while tetrahedral elements are utilized in portions of the model where the geometry is not regular, including the concrete thrust blocks on both abutments, the spillway, and irregular portions of the rock foundation.

The ADAP finite element model is shown in Figures 5-3 and 5-4. This model consists of 1428 elements and 5148 degrees of freedom. The model is meshed in a similar fashion to the ANSYS model, where brick elements are used in the main arch of the dam and tetrahedral elements are used where the geometry is not regular.

5.1.2 Non-Linear Modeling with Computer Program ADAP-NS (includes 9 joints)

Harza utilized computer program ADAP-NS to develop a non-linear elastic finite element model and perform static and seismic analyses of the existing dam with the nine vertical construction joints allowed to open and close. This finite element model, presented in Figures 5-5 through 5-8, consists of 1872 elements with 8970 degrees of freedom. The model is meshed in a similar fashion to the ANSYS and ADAP model, where brick elements are used in the main arch of the dam and tetrahedral elements are used where the geometry is not regular.

5.1.3 Material Properties and Damping

Tables 5-1 and 5-2 summarize the material properties of concrete and rock used in the stability analyses performed by IECO (1977) and the modified Harza values used for the current analyses, respectively. These properties were based on in-situ geophysical testing, borehole photography, water pressure testing, and the laboratory testing programs conducted by IECO (1976) and Harza (1999).

As can be seen in the tables, Harza slightly reduced the concrete density and modulus of elasticity values based on the 1999 laboratory testing program presented in Section 2.0. Harza conducted a thorough review of the IECO (1977) rock testing data and concluded that there is not a compelling reason to alter the rock foundation material properties used in 1977. Therefore, the rock foundation material properties used in the current study are the same as those used by IECO (1977).

TABLE 5-1: MATERIAL PROPERTIES USED IN 1977 FINITE ELEMENT ANALYSIS BY IECO
Santa Anita Dam

	Static Properties		Dynamic Properties	
Property	Concrete	Rock	Concrete	Rock
Density	152 pcf	0 (Massless)	152	0 (Massless)
Poisson's Ratio	0.2	0.2	0.2	0.2
Elastic Modulus	3.0×10^6 psi	2.6×10^6 psi*	3.5×10^6 psi (above El. 1,275) 4.4×10^6 psi (below El. 1,275)	2.6×10^6 psi*

* Mass modulus of elasticity of foundation and abutment rock is 2.6×10^6 psi except as follows:

- Right abutment above El. 1,300: 5.0×10^5 psi
- Right abutment below El. 1,300: 2.0×10^6 psi
- Right abutment between El. 1,170 and 1,190: 1.0×10^6 psi
- Left abutment above El. 1,300: 1.2×10^6 psi

TABLE 5-2: MATERIAL PROPERTIES USED FOR FINITE ELEMENT ANALYSIS BY HARZA (2000)
Santa Anita Dam

	Static Properties		Dynamic Properties	
Property	Concrete	Rock	Concrete	Rock
Density	148 pcf	0 (Massless)	148	0 (Massless)
Poisson's Ratio	0.2	0.2	0.2	0.2
Coefficient of Thermal Expansion	5.6×10^{-6} / °F	-	-	-
Elastic Modulus	2.9×10^6 psi	2.6×10^6 psi*	3.4×10^6 psi (above El. 1,275) 4.2×10^6 psi (below El. 1,275)	2.6×10^6 psi*

* Mass modulus of elasticity of foundation and abutment rock is 2.6×10^6 psi except as follows:

- Right abutment above El. 1,300: 5.0×10^5 psi
- Right abutment below El. 1,300: 2.0×10^6 psi
- Right abutment between El. 1,170 and 1,190: 1.0×10^6 psi
- Left abutment above El. 1,300: 1.2×10^6 psi

The damping ratio used for dynamic analyses is five percent (5%). This is the same value as was previously used in the preliminary seismic "reference analysis" presented in the Analysis Plan (Harza, 1999) and the 1977 IECO study.

5.1.4 Permissible Stresses for Stability Evaluation

The following permissible strength values are based on the field and laboratory data from IECO (1976) and obtained by Harza in 1999 previously presented in Sections 2.0 and 3.0:

- Permissible compressive strength: 4500 psi (static loading)
  5800 psi (dynamic loading)

- Permissible tensile strength: 450 psi (static loading)
  650 psi (dynamic loading)

These permissible strength values will be compared to stresses within the dam to evaluate the stability of the dam under both static and seismic dynamic loading conditions.

As stated previously, the scope of the 1999 investigation was limited to the upper portion of the dam (i.e., above El 1300). Accordingly, one can see that the data have been used appropriately in conjunction with the 1976 data to assign realistically conservative permissible strength values for both the upper and lower portions of the dam. Furthermore, since it is Harza's experience that the effects of AAR would be seen primarily in the upper portion of the dam, and the 1999 test data show no significant reduction in concrete strength as compared to the data obtained in 1976, we feel that the existing data are adequate for the purpose of the current study.

5.1.5 Modal Analyses and Hydrodynamic Effects

The first 50 modal periods of the dam were calculated for both empty-reservoir and full-reservoir (water surface at El. 1,316 and silt level at El. 1,179.5) conditions. The full-reservoir hydrodynamic effects (due to both water and silt) were modeled in ANSYS using the added mass approach in accordance with the Westergaard procedure as updated by Zanger (1952), and in ADAP using a full reservoir finite element model as shown in Figure 5-8. The unit weights of water and equivalent unit weight of saturated silt were assumed to be 62.5 pcf and 82.5 pcf, respectively.

The modal periods calculated by four different models of the dam for both the full and empty reservoir conditions are presented in Table 5-3. The four finite element models listed in Table 5-3 include:

- IECO (1977) - This is the linear-elastic model of the dam developed by IECO in 1977. Hydrodynamic effects were modeled using the Westergaard procedure.

- ANSYS (1999) – This is the linear-elastic model of the dam developed by Harza using ANSYS program and previously used in the preliminary "reference

analysis" presented in the Analysis Plan (Harza, November 8, 1999). Similar to IECO (1977), hydrodynamic effects were modeled using the Westergaard procedure.

- ANSYS (2000) – This model, used in the current study, is a refined version of the ANSYS (1999) linear-elastic model. The primary differences between the ANSYS (1999) and the ANSYS (2000) models are that the finite element mesh was refined, and also the added mass applied to the upstream face of the dam to account for hydrodynamic effects was adjusted so that the modal periods match the ADAP (2000) model, as explained further in the next section; i.e., used equivalent of finite element calculated hydrodynamic effect (instead of Westergaard added mass).

- ADAP (2000) – This model, also used in the current study, is the linear-elastic version of the dam as modeled in ADAP. A finite element model of the reservoir was used to apply a diagonalized added mass matrix to the upstream face of the dam to account for hydrodynamic effects (in contrast to the Westergaard added mass used in IECO-1977 and ANSYS-1999 analyses).

Mode shapes of the first 3 modes calculated by ANSYS (2000) for the full reservoir condition are shown in Figures 5-9 through 5-10, and the first 3 modes calculated by ADAP (2000) are shown in Figures 5-11 and 5-12.

As can be seen in Table 5-3, the modal periods calculated by each of the ANSYS (1999), ANSYS (2000) and ADAP (2000) models for the empty reservoir condition closely match those calculated by IECO (1977). However, as explained in the next section, the modal periods calculated for the full reservoir condition for the current study are not similar to the IECO (1977) values due to differences in the way the added mass was applied to the upstream face of the dam.

TABLE 5-3: RESULTS OF MODAL ANALYSES
JOINTS CLOSED
Santa Anita Dam

	FULL RESERVIOR				EMPTY RESERVIOR			
MODE NO.	ADAP (2000) (period, s)	ANSYS (2000) (period, s)	ANSYS (1999) (period, s)	IECO (1977) (period, s)	ADAP (2000) (period, s)	ANSYS (2000) (period, s)	ANSYS (1999) (period, s)	IECO (1977) (period, s)
1	0.18	0.18	0.23	0.23	0.15	0.15	0.15	0.15
2	0.16	0.16	0.20	0.21	0.14	0.14	0.14	0.15
3	0.12	0.12	0.15	0.16	0.11	0.11	0.11	0.12
4	0.10	0.11	0.12	0.14	0.09	0.10	0.09	0.10
5	0.10	0.09	0.12	0.12	0.08	0.08	0.08	0.09
6	0.09	0.09	0.10	0.12	0.08	0.08	0.08	0.08
7	0.07	0.08	0.10	0.10	0.07	0.07	0.07	0.08
8	0.07	0.07	0.09	0.10	0.07	0.06	0.06	0.07
9	0.07	0.07	0.09	0.09	0.07	0.06	0.06	0.07
10	0.06	0.07	0.08	0.08	0.06	0.06	0.06	0.06
20	0.04	0.04	0.05		0.04	0.04	0.03	
30	0.03	0.03	0.04		0.03	0.03	0.03	
40	0.02	0.03	0.03		0.02	0.02	0.02	
50	0.02	0.02	0.03		0.02	0.02	0.02	

5.1.6 Model Calibrations

5.1.6.1 Empty Reservoir Condition

Two researchers, Dr. John F. Hall, Professor of Civil Engineering at California Institute of Technology in Pasadena and Dr. Ziyad H. Duron (former student of Dr. Hall and now Professor of Engineering at Harvey Mudd College in Claremont, California) performed forced vibration tests on Santa Anita Dam in 1984 with the objective of studying dynamic dam-foundation interaction effects. Measurements included accelerations on the dam, the dam-foundation interface, and on the adjacent foundation away from the dam. The reservoir level was low at the time of the field tests. The results of their work are described in detail in Duron and Hall (1991).

The first three modes of vibration measured by Drs. Hall and Duron included two symmetric modes (periods of 0.16 second and 0.11 second) and one asymmetric mode (period of 0.15 second). These experimental results closely match the calculated modal periods summarized in Table 5-3 for empty reservoir conditions. There are no experimental data available for full-reservoir conditions.

5.1.6.2 Full Reservoir Condition

One can see from Table 5-3 that the modal periods calculated by ANSYS (1999) were very similar to the modal periods calculated by IECO (1977) for the full reservoir condition. This is due to the fact that the Westergaard approach for applying added mass to the upstream face of

the dam was used in both of these models to account for hydrodynamic effects on the upstream face of the dam. Although it is well known that the Westergaard approach generally overestimates the hydrodynamic effects, this approach was used in the preliminary "reference analysis" presented in Harza (1999) as a preliminary method to obtain an indication as to where the dam may be potentially overstressed during the MCE.

For the current study, however, a finite element model of the reservoir was used to apply a diagonalized added mass matrix to the upstream face of the dam in the ADAP (2000) model. As can be seen, the first modal period in the ADAP (2000) model was significantly less that the IECO (1977) or the ANSYS (1999) models due to the significantly lower total added mass applied to the upstream face of the dam. Since the Westergaard approach was known to be overly conservative, the total added mass applied to the ANSYS (2000) model was adjusted in an iterative procedure to closely match the modal periods of the ADAP (2000) model.

Although the model calibration described above produces modal periods in the ANSYS (2000) model which very closely match the modal periods in the ADAP (2000) model, it needs to be emphasized that the total added mass between the two models are not exactly the same. This is due to the two different methods used to model the hydrodynamic effects. In the Westergaard approach, an equal amount of mass is applied in all directions on the upstream face of the dam, while the full reservoir finite element model approximates more closely hydrodynamic effects by applying unequal components of mass directionally. This is an important consideration to remember when comparing stresses between the two models.

5.2 Results of Static Reanalysis

Static linear elastic analyses were performed on the existing dam using ANSYS. The results are summarized in Table 5-4 in the form of arch, cantilever and principal stresses as summarized below.

TABLE 5-4: STATIC STRESS ANALYSES*
MAXIMUM STRESSES
Santa Anita Dam

Location	Gravity Only		Water Only		Silt Only		Temperature Only			
							Summer		Winter	
	Max Tension (psi)	Max Compression (psi)	Max Tension (psi)	Max Compression (psi)	Max Tension (psi)	Max Compression (psi)	Max Tension (psi)	Max Compression (psi)	Max Tension (psi)	Max Compression (psi)
ANSYS MODEL										
Upstream Face										
Arch Stress	25*	80	55*	292	14	5	245*	100*	190*	0
Cantilever Stress	0*	271	125*	83	12	3	170*	70*	225*	70*
Principal Stress	20*	328	200*	278	26	0	--	--	--	--
Downstream Face										
Arch Stress	25*	45	125	153	0	4	100	556	200**	0*
Cantilever Stress	0*	138	125	83*	0	5	100*	280*	225*	70*
Principal Stress	40*	208	200	350*	0	13	--	--	--	--

* Reports the maximum value within the dam excluding stress concentrations on or near the dam abutments due to non-regular element shape, transition from hexagonal to tetrahedral elements and change in material properties.
** Geometric anomaly (i.e., sharp geometric transition in the model) caused fictitiously high tensile stresses to be calculated at the bottom of the dam on the downstream face when applying temperature load. The value shown is the overall winter tensile stress on the downstream face.

5.2.1 Stresses Due to Gravity Load

Stresses due to the self-weight of the dam as modeled in ANSYS are shown in Figures 5-13 to 5-16. As shown in Table 5-4 (at end of Chapter 5), the maximum and minimum principal stresses due to gravity are relatively low.

5.2.2 Stresses due to Hydrostatic Load

Stresses due to hydrostatic loading only are shown in Figures 5-17 through 5-20. Pressures of 62.5 pcf x height (depth of element below reservoir level at spillway crest El. 1316) were applied on the upstream face of the dam and foundation in ANSYS to model the effects of hydrostatic loading. As shown in Table 5-4, the stresses due to hydrostatic loading are somewhat high, however, the high tensile stresses that occur are at the edges of the dam-foundation. The high tensile stresses that occur at these locations can be attributed to two factors: the change in geometry of the model and the change in material properties at these locations. When using engineering judgement to consider the physical action at this location, it is predicted that the dam would relieve tensile stresses by breaking the bond between the concrete and foundation. Therefore, the modeled tensile stresses at dam-foundation interface may be disregarded.

5.2.3 Stresses due to Static Silt Load

Stresses due to silt loading only are shown in Figures 5-21 through 5-24. Pressures of 20 pcf x height (depth of element below silt level at El. 1179.5) was applied on the upstream face of the dam and foundation in ANSYS to model the effects of static silt loading. As shown in Table 5-4, the stresses due to silt load are insignificant when compared to other loading conditions on the dam.

5.2.4 Stresses due to Temperature

The stress free temperature used for the temperature stress analyses was taken to be the mean annual temperature of 62° F as presented in Section 2.0, Figure 2-24. Average maximum daily temperatures shown in Figure 2-24 for each month from May to October were averaged (81° F) to use as the summer temperature load. Average minimum daily temperatures shown in Figure 2-24 for each month from November to April were averaged (43° F) to use as the winter temperature load. These values were applied to the air exposed surfaces on the ANSYS model during full reservoir conditions. The temperature at the bottom of the reservoir was constant for both cases (50° F.). The thermal capabilities of ANSYS were utilized to compute the temperature gradient within the dam as shown in Figure 5-25.

Stresses due to temperature loading only are shown in Figures 5-26 through 5-29. As shown in Table 5-4, the tensile stresses due to temperature loading are relatively high; however, the peak stresses that occur are at the edges of the dam-foundation interface during winter loading. During the winter, the concrete and tends to contract and pull away from the foundation, thus producing high tensile stresses at the dam-foundation interface. Since this linear elastic model does not incorporate a joint at the dam-foundation interface, the high stresses at these locations may be disregarded.

5.2.5 Displacements and Stresses due to AAR

Expansion of the dam due to AAR was modeled by applying a uniform body load in the form of temperature load to the dam in increasing increments. The peak principal stresses shown in the lower portion of Figure 5-30 are near the permissible static tensile strength of the concrete (450 psi), and occur at a point when the upstream vector displacement is one half inch (0.5 in.) at Station 0+90. These peak principal stresses occur in the same vicinity as observed cracks on the dam reported in Section 3.0 on the downstream face of the dam.

5.3 Results of Seismic Reanalysis with EQ1

Dynamic response was calculated by mode superposition using the first 50 modes of the dam vibration. As can be seen in Table 5.3 (section 5.1.5), the 50th mode has a period of 0.02 second. By inspection of the response spectrum for EQ1, the contribution of modes with periods lower than 0.02 second appears to be negligible when compared to the overall response of the

structure. Usually, the first 20 to 30 modes adequately represent the dynamic response of a concrete arch dam.

The results of both ANSYS and ADAP/ADAP-NS seismic analyses are presented in the sections below.

5.3.1 Linear-Elastic Seismic Analyses

Dynamic linear elastic analyses were performed on the existing dam with EQ1 ground-motion record. ANSYS analyses were performed with Westergaard hydrodynamic added mass applied for full reservoir conditions (water level at the top of the spillway crest at El. 1,316 and silt level at the top of the sluice gate at El. 1,179.5). The hydrodynamic effect for ADAP analyses was calculated with a finite element model of the reservoir. A constant damping ratio of 5 percent was used for all analyses.

The results of dynamic analyses of the existing dam are presented in Figures 5-31 through 5-40, and maximum tensile and compressive stresses are summarized in Table 5-5.

TABLE 5-5: DYNAMIC LINEAR-ELASTIC STRESS ANALYSIS
MAXIMUM STRESSES COMPUTED IN ANSYS
Santa Anita Dam

Location	Time = 10.40 sec		Time = 11.78 sec		Time = 14.16 sec	
	Max Tension (psi)	Max Compression (psi)	Max Tension (psi)	Max Compression (psi)	Max Tension (psi)	Max Compression (psi)
Upstream Face						
Arch Stress	1035	2435	861	1868	1962	688
Cantilever Stress	1329	458	0	1775	354	1257
Downstream Face						
Arch Stress	653	1667	861	354	1375	0
Cantilever Stress	0	1049	743	0	813	569

Figure 5-31 shows the time-history of relative displacement of the top of the dam in the cross-canyon direction with respect to a fixed point on the boundary of the foundation in the ANSYS model. As can be seen in this figure, the maximum relative displacements occurred in the range of 10 to 15 seconds into the earthquake. Therefore, stresses were expanded for this range of time at 0.02 second increments. The time-stress histories at six different locations on the dam are included as Figures 5-32 through 5-34. The stresses at each increment in time were then examined to determine that maximum tensile and compressive stresses occurred at the time points of 10.40, 11.78, and 14.16 seconds during the earthquake.

Accordingly, Figures 5-35 and 5-36 present the stresses at the 10.40 time increment, Figures 5-37 and 5-38 present the stresses at the 11.78 time increment, and Figures 5-39 and 5-

40 present the stresses at the 14.16 time increment. Used together, these figures present the maximum stresses within the dam for this linear dynamic analysis, summarized in Table 5-5.

It is to be noted and emphasized that the values of maximum stresses shown in Table 5-5 are not to be accumulated. These are calculated maximum values, which occur in different locations (elements) in the dam at different times. Furthermore, we emphasize that stresses shown in the figures and summarized in Table 5-5 are due to seismic dynamic loading only for this analysis. Stresses due to static loading (i.e., gravity, static water pressure, static silt load, AAR stresses and temperature) are not included in these results; however, the results of static analyses will be formally superimposed, element by element, in future analyses of the dam.

As can be seen in the time-stress history plots, Figures 5-32 through 5-34, the tensile stresses on both the upstream and downstream faces of the dam exceed the permissible dynamic tensile concrete strength of 650 psi. As one would expect, the arch tensile stresses are highest at the top of the dam on the upstream face.

The upper portion of Figure 5-32 and the lower portion of Figure 5-33 shows that arch stresses at the top of the dam on the downstream face are approximately one-half the magnitude of arch stresses at the top of the dam on the upstream face. However, the stresses on the downstream face still exceed the maximum permissible value of 650 psi. Comparison of Figure 5-34 to the lower portion of Figure 5-33 shows that arch and cantilever stresses mid-height on the downstream face of the dam are similar in magnitude to the arch tensile stresses at the top of the dam on the downstream face.

Figure 5-39 shows that the maximum tensile arch stress of 1962 psi occurs at 14.16 seconds at the top of the dam at the center of the arch on the upstream face. Given that the cantilever stresses on the upstream face at the top of the dam are near zero at the same point in time (see Figure 5-40), it follows that the tensile stresses have an instantaneous peak concentration at this location. The maximum compressive arch stress of 2435 psi occurs at the same location at 10.40 seconds into the earthquake (see Figure 5-35). As one would expect, the cantilever stresses on the downstream face are relatively low at this point in time.

Figure 5-36 shows that the maximum tensile cantilever stress of 1329 psi occurs on the upstream face of the dam at mid-height at the center of the arch at 10.40 seconds. The maximum compressive cantilever stress of 1775 psi occurs on the upstream face of the dam at mid-height near the right abutment at 11.78 seconds as shown in Figure 5-38.

5.3.2 Non-Linear Seismic Analyses

Dynamic non-linear elastic analyses were performed on the existing dam with EQ1 ground-motion record with the nine vertical construction joints allowed to open and close. ADAP-NS analyses were performed with a reservoir finite element mesh for hydrodynamic effect for full reservoir conditions (water level at the top of the spillway crest at El. 1,316 and silt level at the top of the sluice gate at El. 1,179.5). A constant damping ratio of 5 percent was used for all analyses.

The results of dynamic analyses of the existing dam are presented in Figures 5-41 through 5-52, and maximum tensile and compressive stresses are summarized in Table 5-6. Seismic dynamic stresses were computed in ADAP-NS using EQ1 with gravity, water and silt static loads superimposed as well. As can be seen in the results, stresses due to static loading were found to be insignificant in comparison to stresses due to dynamic loading, and conclusions regarding the analysis would be the same whether or not static loads were superimposed.

TABLE 5-6: DYNAMIC STRESS ANALYSIS
ADAP-NS NON-LINEAR ELASTIC MODEL
Santa Anita Dam

Location	Max Tension (psi)	Max Compression (psi)
Upstream Face		
Arch Stress	729	2864
Cantilever Stress	2003	2307
Downstream Face		
Arch Stress	786	2445
Cantilever Stress	2300	1932

The top portion of Figure 5-41 shows the time-history of relative displacement at the top of the dam in the cross-canyon direction with respect to a fixed point on the boundary of the foundation. As can be seen in this figure, the maximum relative displacements occurred in the range of 10 to 12 seconds into the earthquake. Therefore, stresses were expanded for this range of time at 0.02 second increments. The time-stress histories at four different locations on the dam are included as Figures 5-42 through 5-43.

Figure 5-44 shows that joint number 7 opens over thirty (30) times during EQ1, and, more notably, the joint opens ¼ inch or more 14 times during EQ1. Joint opening occurs during the non-linear analysis when the tensile strength of the joint is exceeded during the earthquake.

The stresses at each increment in time as plotted in the above-cited figures were examined to determine that maximum tensile and compressive stresses occurred at the time points of 10.87, 11.28, 11.36, and, 11.46 seconds during the earthquake. Accordingly, Figures 5-45 and 5-46 present the stresses at the 10.87 time increment, Figures 5-47 and 5-48 present the stresses at the 11.28 time increment, Figures 5-49 and 5-50 present the stresses at the 11.36 time increment, and Figures 5-51 and 5-52 present the stresses at the 11.46 time increment. Used together, these figures present the maximum stresses within the dam for this non-linear elastic dynamic analysis summarized in Table 5-6.

As can be seen in the time-stress history plots, Figures 5-42 through 5-43, the tensile stresses on both the upstream and downstream faces of the dam exceed the permissible dynamic tensile concrete strength of 650 psi. As one would expect, the arch tensile stresses are

highest at the top of the dam on the upstream face. The cantilever tensile stresses, however, have the highest values overall.

Figure 5-42 shows that the maximum permissible arch tensile stresses are only exceeded by a small amount at both the upstream and downstream faces of the dam. This figure shows that the maximum arch tensile stress of 786 psi occurs at 11.36 seconds. Figure 5-49 shows this maximum occurring mid-height near the right abutment on the downstream face. The maximum compressive arch stress of 2864 occurs at 11.28 seconds at the top of the dam at the center of the arch on the upstream face (Figure 5-47).

Figure 5-52 shows that the maximum tensile cantilever stress of 2300 psi occurs on the downstream face of the dam at mid-height near the right abutment at 11.46 seconds. The maximum compressive cantilever stress of 2307 psi occurs on the upstream face of the dam at mid-height near the left abutment at 10.87 seconds as shown in Figure 5-46.

5.3.3 Comparison of Results to the 1976-77 Stability Evaluation

Comparison of the results of the current study to that of the previous study performed by IECO (1977) shows that the maximum tensile stresses occurred at approximately the same locations in both studies (i.e., the maximum arch tensile stress occurred at the top center of the dam on the upstream face). Table 5-7 shows the maximum arch and cantilever tensile stresses for the current linear and non-linear analyses as well as the 1977 IECO values.

TABLE 5-7 – COMPARISON OF MAXIMUM TENSILE STRESSES FOR IECO (1977) AND HARZA (2000)
Santa Anita Dam

Stresses	Harza (2000) *Non-Linear Model (ADAP-NS)*	Harza (2000) *Linear Model (ANSYS)*	IECO (1977) *Linear Model*
Maximum Tensile Arch Stress	786	1965	1408
Maximum Tensile Cantilever Stress	2300	1625	1482

As shown in the table, the maximum cantilever tensile stress of 2300 psi calculated in the current non-linear analysis is approximately one and one-half times the maximum tensile stress of 1482 psi calculated by IECO. Also, the maximum arch tensile stress of 1965 psi calculated in the linear analysis is approximately 1.4 times (40%) the maximum tensile stress of 1408 psi calculated by IECO.

The primary reason for the significantly higher stresses in the current analysis over the previous study is that the EQ1 ground motion record is a significantly larger event than the ground motion record considered to be the MCE in 1977 (i.e., a Richter magnitude 7.0 event on

the Sierra Madre Fault). The peak horizontal ground acceleration (pga) of 1.45g for the current MCE (EQ1) is more than twice the horizontal pga of 0.7g from the ground motion record used in the 1977 analysis. Furthermore, the peak horizontal spectral acceleration in the 1977 response spectrum was approximately 1.6g, while the peak horizontal spectral acceleration in the present reference analysis is 3.7g, more than two times higher. Even more striking is the difference in vertical ground motion and response between the two studies. Both the vertical pga of 1.3g and maximum spectral response of 3.2g in the current study are nearly three times the respective values of 0.45g and 1.2g from the 1977 study.

It should be noted that although the peak spectral accelerations and peak ground accelerations for the current study are more than double the accelerations in the 1977 study, the peak stresses in the current study are only a maximum of 1.5 times larger. The reason for the discontinuity in this correlation is due to the reduction of hydrodynamic load on the upstream face of the dam for the current study; i.e. – if the hydrodynamic loads were similar in both studies (as is the case when comparing the Harza 1999 Analysis Plan to the 1977 IECO study), the stresses would increase in proportion with the increase in peak spectral acceleration and peak ground acceleration.

5.3.4 Conclusions Regarding Seismic Stability

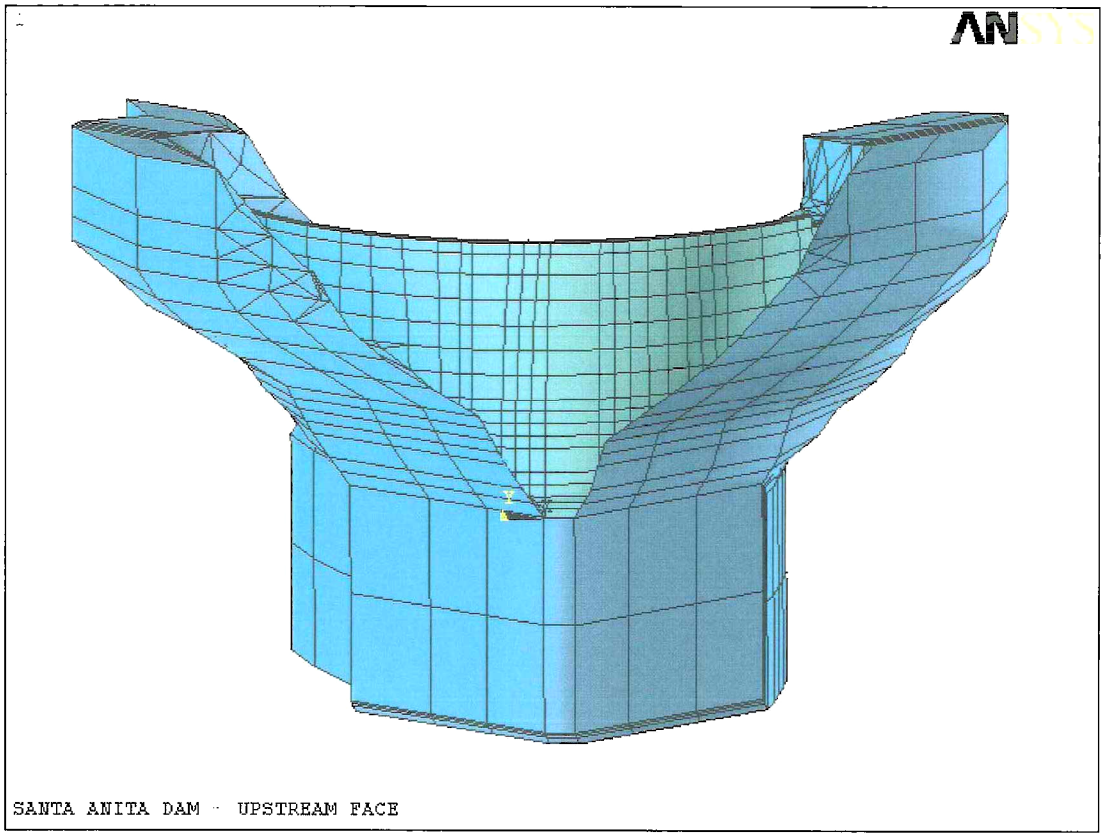
The primary finding of the linear-elastic and non-linear analyses is that the maximum tensile stresses generated during earthquake loading exceed the maximum permissible tensile strength of concrete. The arch tensile stresses are highest at the top of the dam on the upstream face for the linear analysis, exceeding permissible tensile stresses over 30 cycles during EQ1. The non-linear analysis shows similar results; however, the arch tensile stresses are exceeded only three cycles during EQ1, as shown in the time-history plots presented in section 5.3.1. Permissible tensile stresses for cantilever action in the linear analysis are exceeded fourteen cycles on the upstream face of the model. For the non-linear analysis, cantilever stresses are significantly higher than the arch stresses; the tensile cantilever stress peaks exceed the permissible stress of 650 psi seventeen cycles on the upstream face of the model (see Figure 5-43).

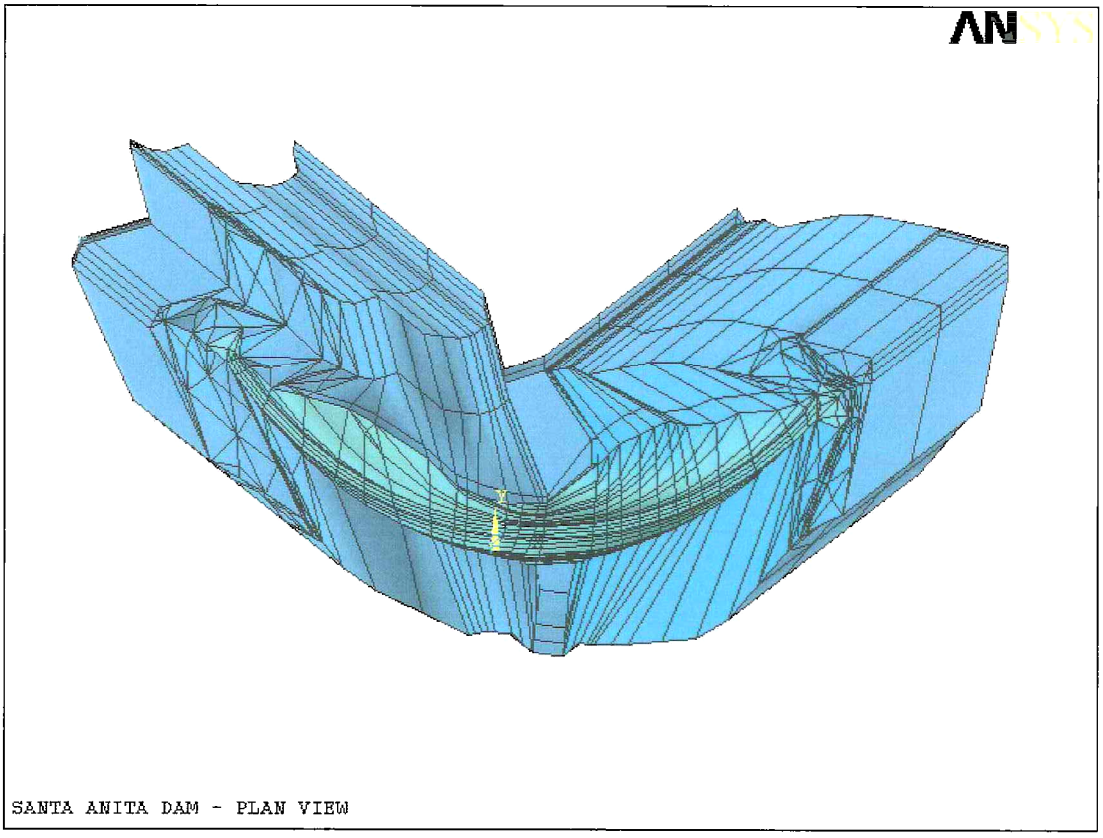
For the linear model, arch stresses at the top of the dam on the downstream face are approximately one-third the magnitude of arch stresses at the top of the dam on the upstream face. Arch and cantilever stresses mid-height on the downstream face of the dam are approximately two-thirds of the magnitude of arch tensile stresses at the top of the dam.

For the non-linear model, peak tensile arch stresses at the top of the dam on the downstream face occur approximately two-thirds less than on the upstream face of the dam. Peak tensile cantilever stresses occur in nearly the same frequency and magnitude on the upstream and downstream faces of the non-linear model.

Based on *both* the linear and non-linear analyses in ADAP and ANSYS, it is very clear that tensile stresses exceed permissible concrete strength values. This implies that the dam is not capable of withstanding the Maximum Credible Earthquake without incurring significant

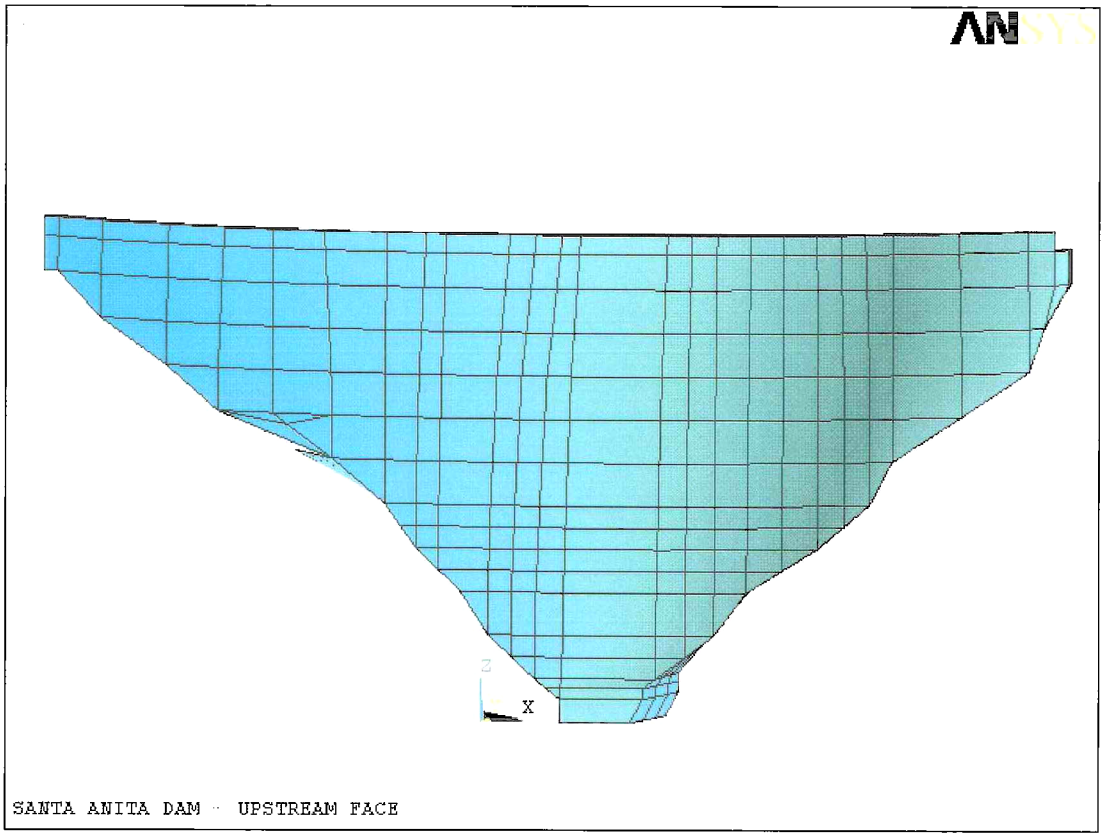
structural damage; thus, rendering the dam unsafe for operation. Remedial measures are necessary to ensure safe operation of the dam for a Maximum Credible Earthquake.

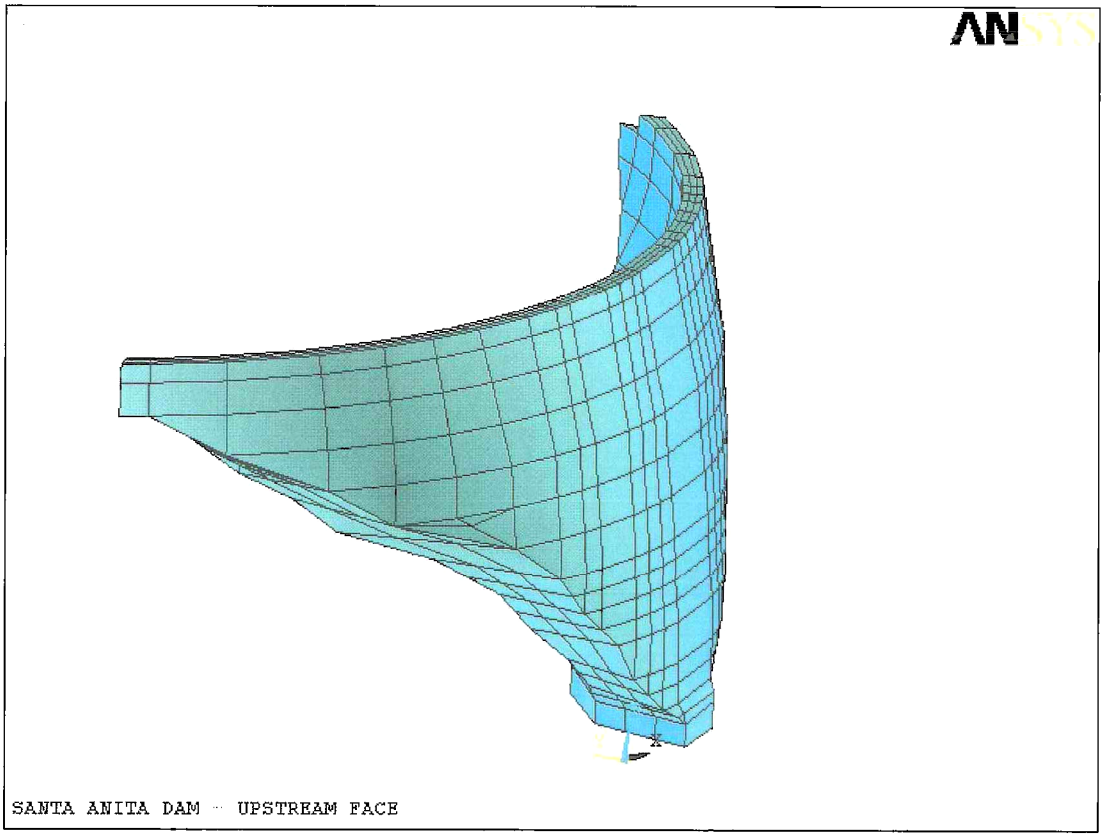




HARZA

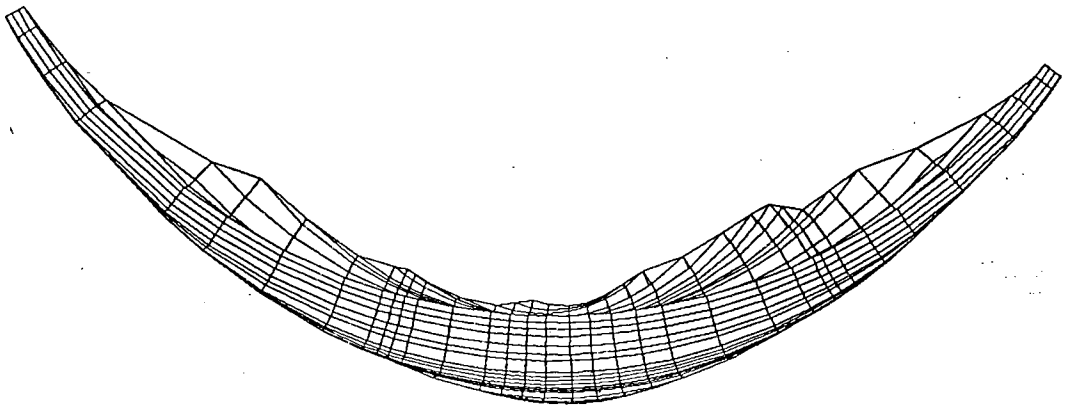
Figure 5-1



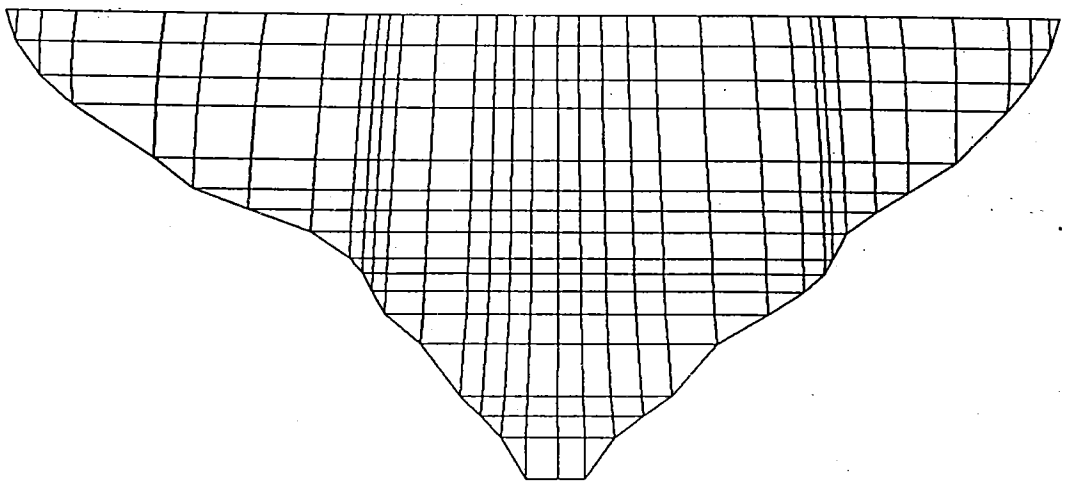


HARZA

Figure 5-2



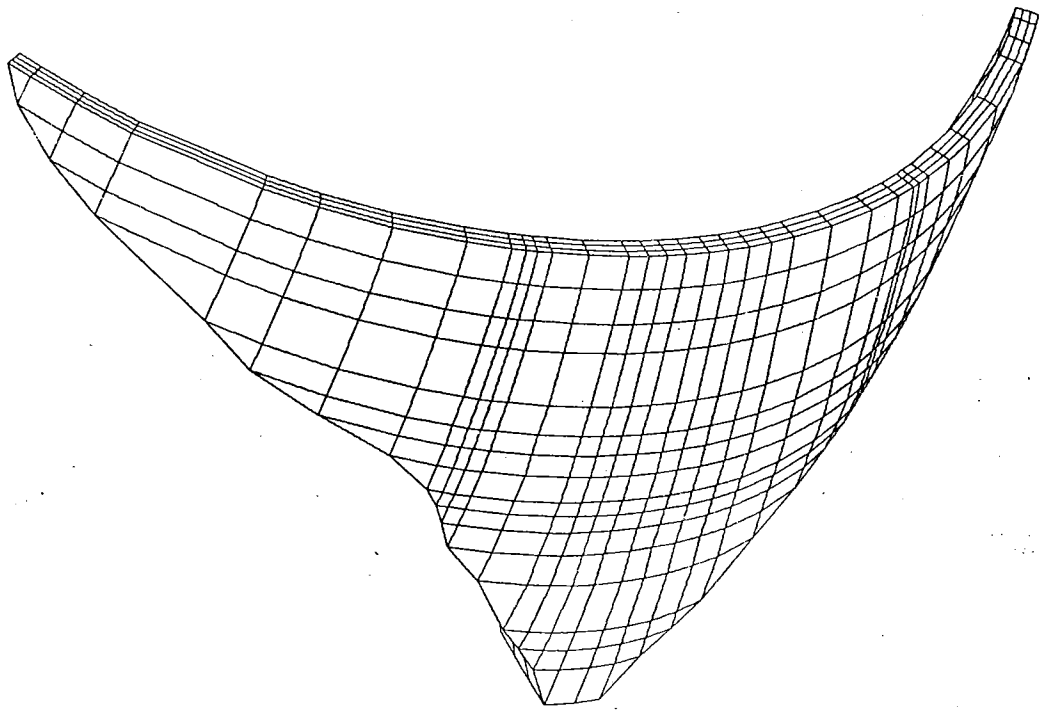
Plan View of ADAP
Linear Model



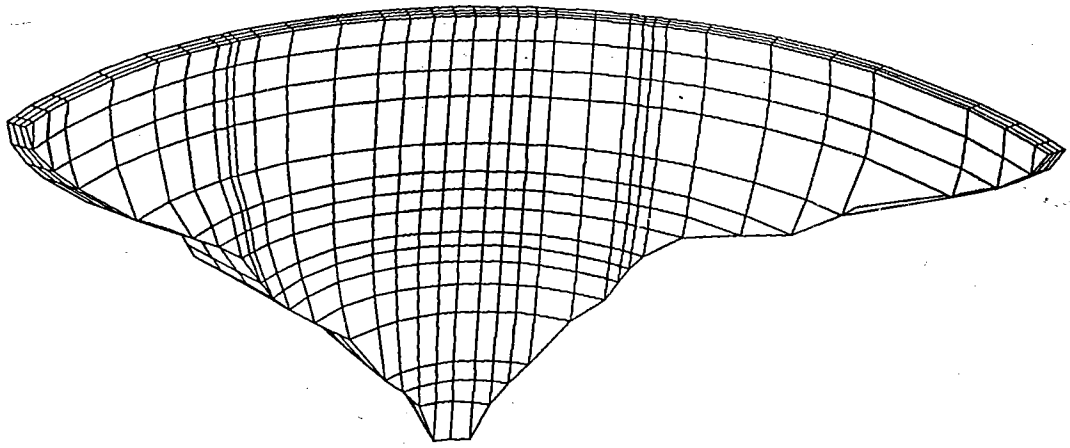
Elevation View of ADAP
Linear Model

HARZA

Figure 5-3



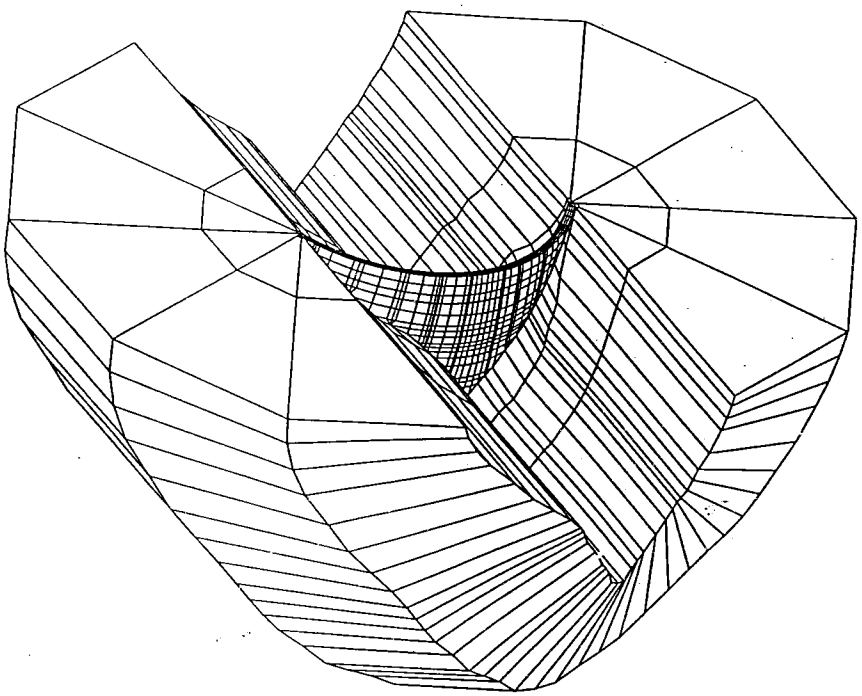
Upstream View of ADAP
Linear Model



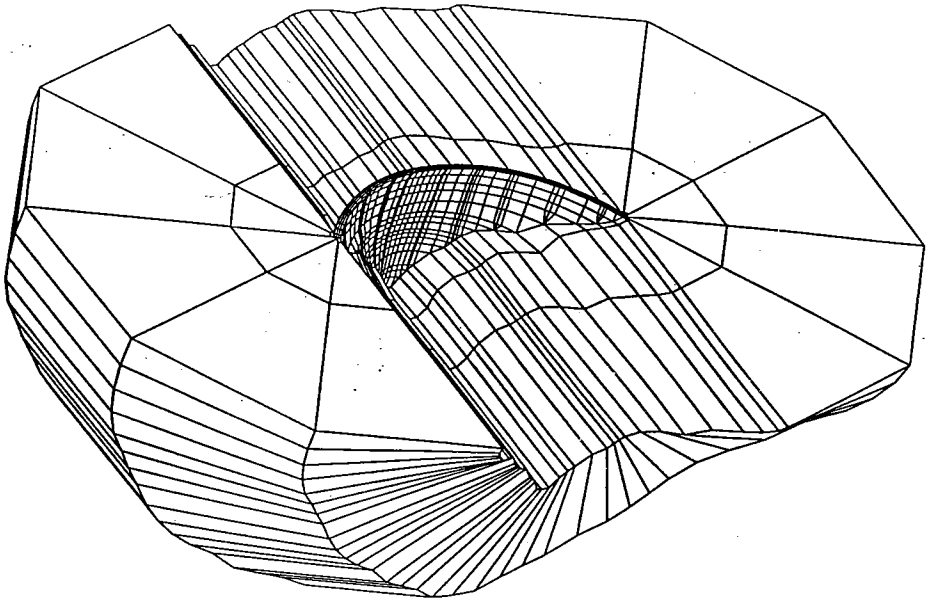
Downstream View of ADAP
Linear Model

HARZA

Figure 5-4



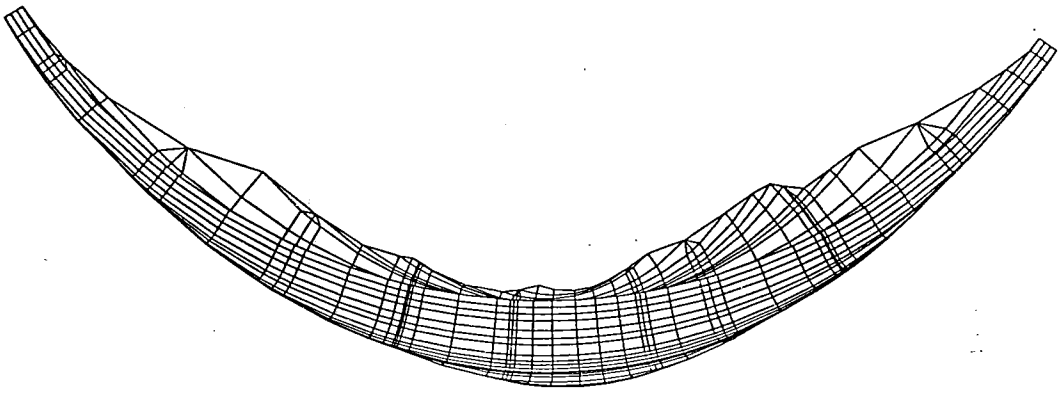
Upstream View of ADAP-NS
Nonlinear Model



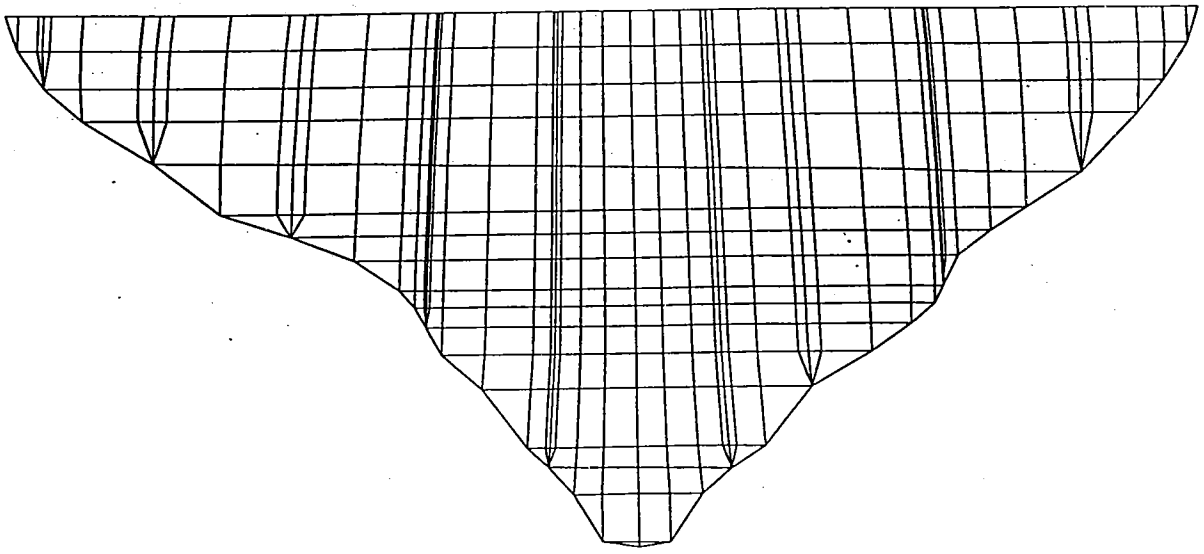
Downstream View of
ADAP-NS Nonlinear
Model

HARZA

Figure 5-5



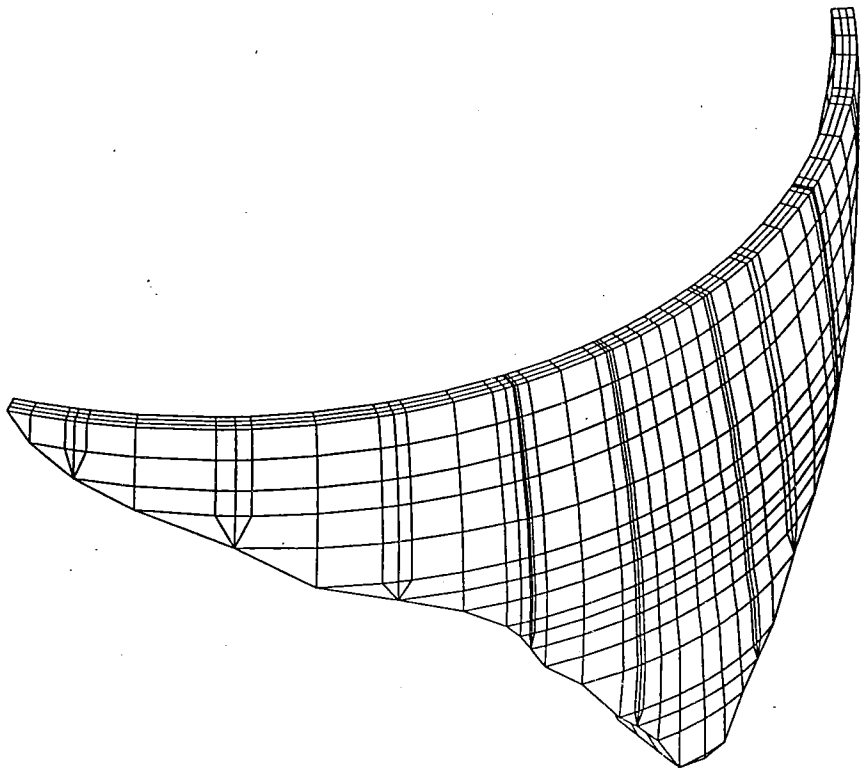
Plan View of ADAP-NS
Linear Model



Elevation View of ADAP-NS
Nonlinear Model

HARZA

Figure 5-6



Upstream Profile of
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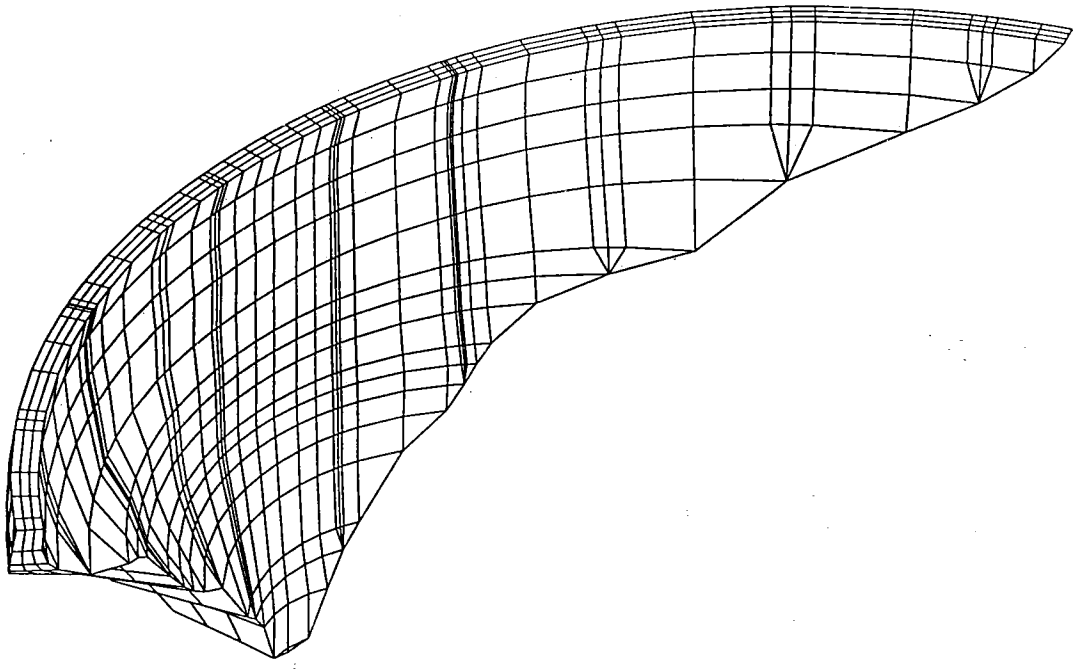
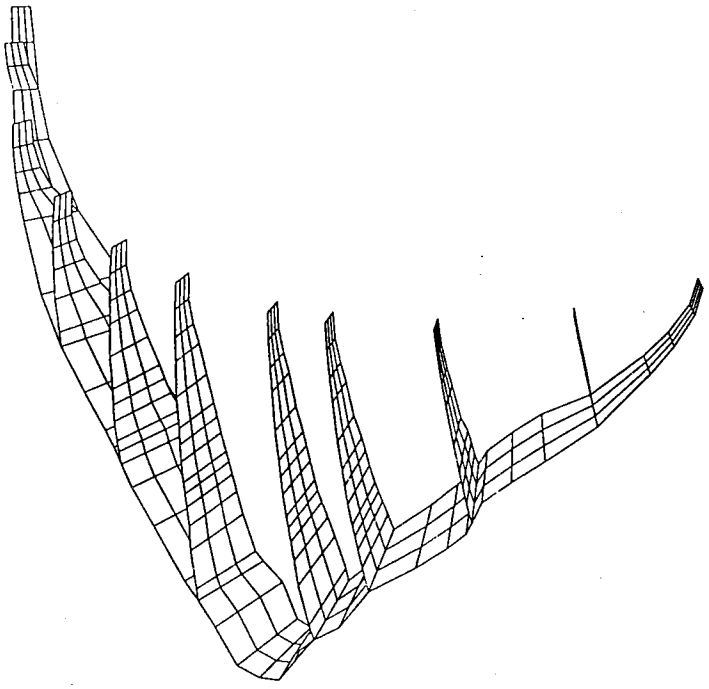
Model



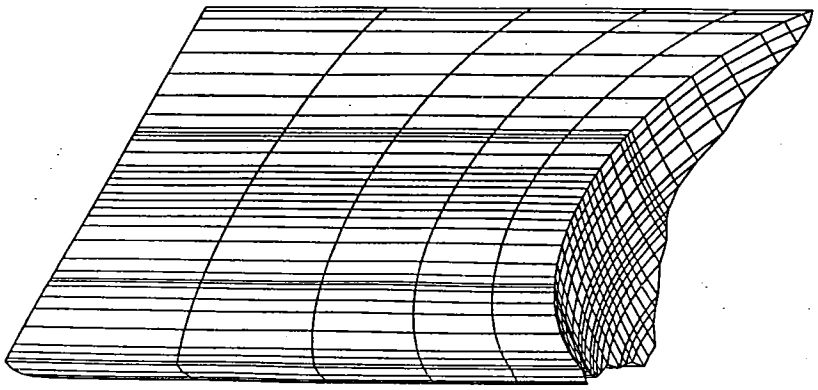
Downstream Profile of
ADAP-NS Nonlinear
Model

HARZA

Figure 5-7



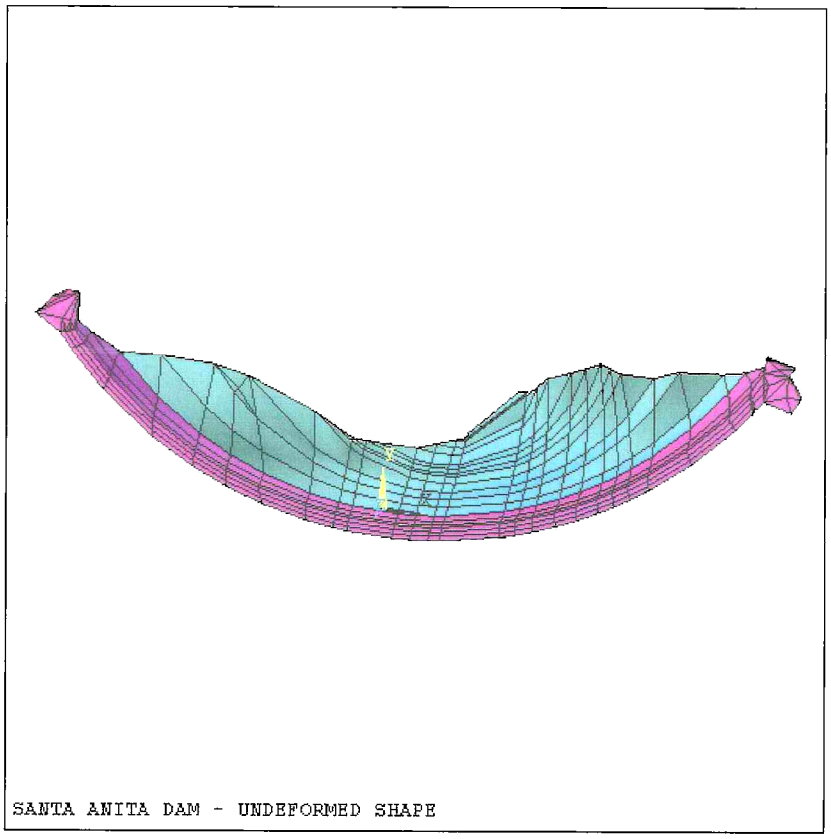
Joint Elements of ADAP-NS
Nonlinear Model



ADAP-NS Reservoir
Model

HARZA

Figure 5-8



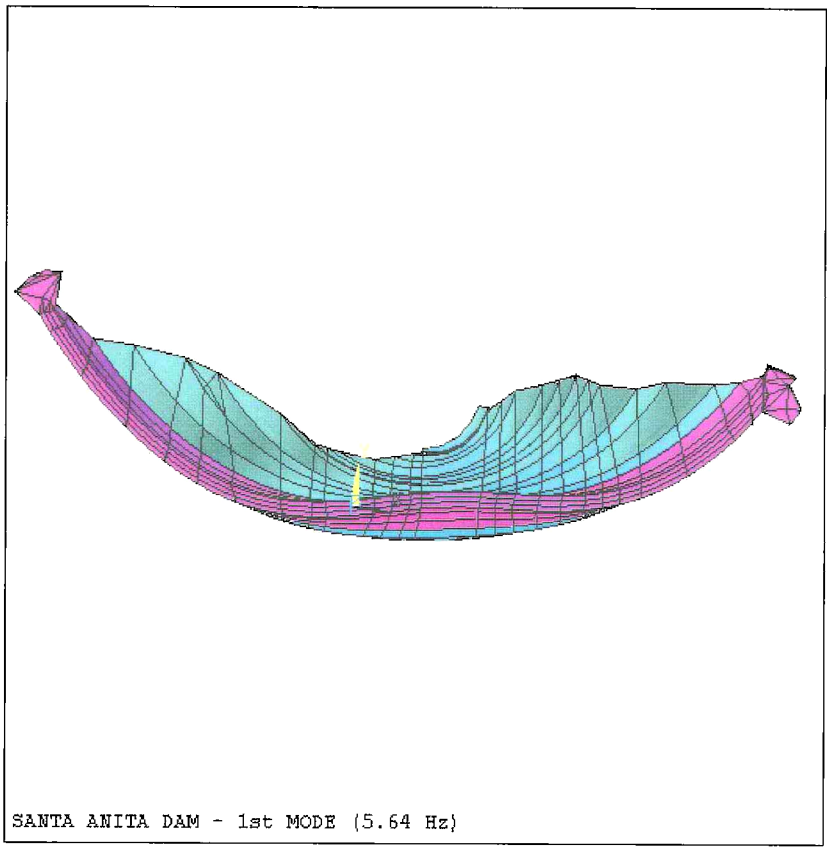


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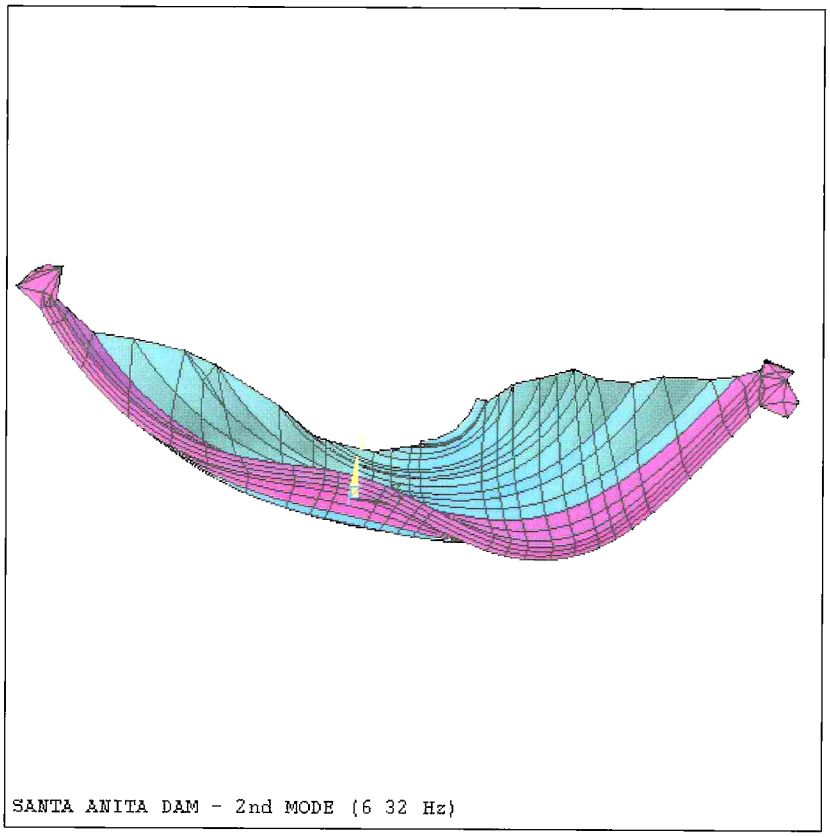
Figure 5-9



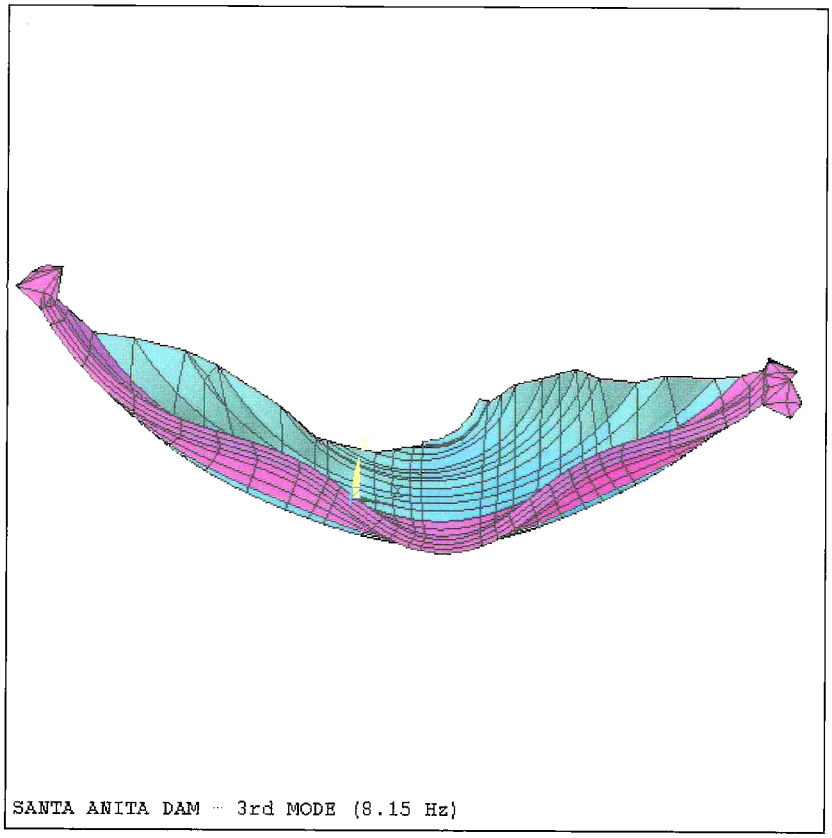


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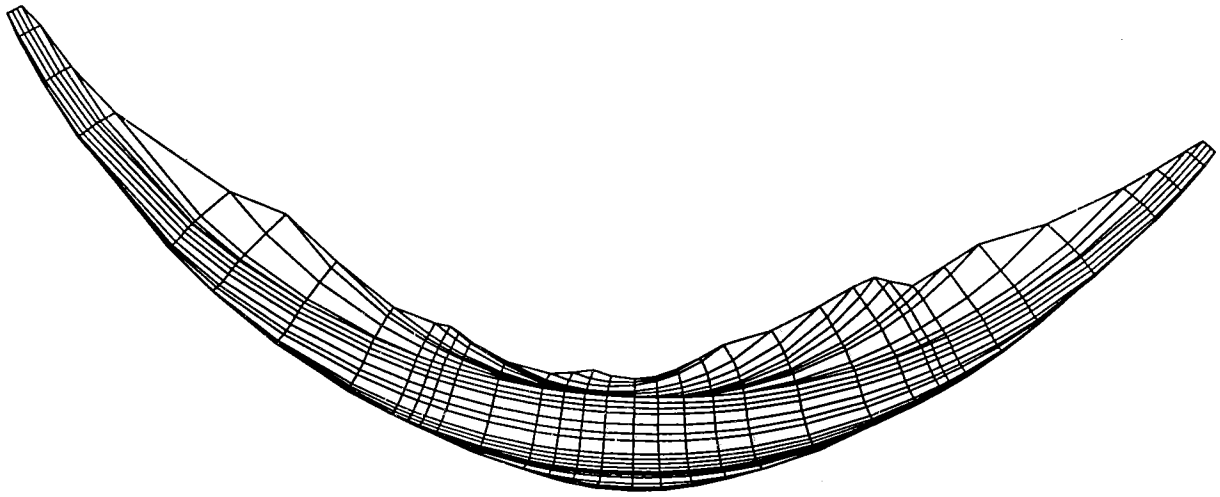
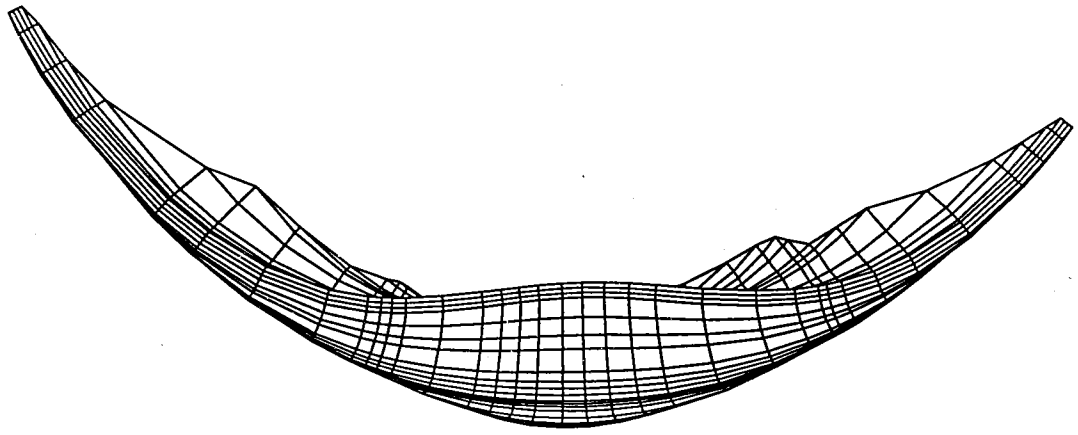
Figure 5-10



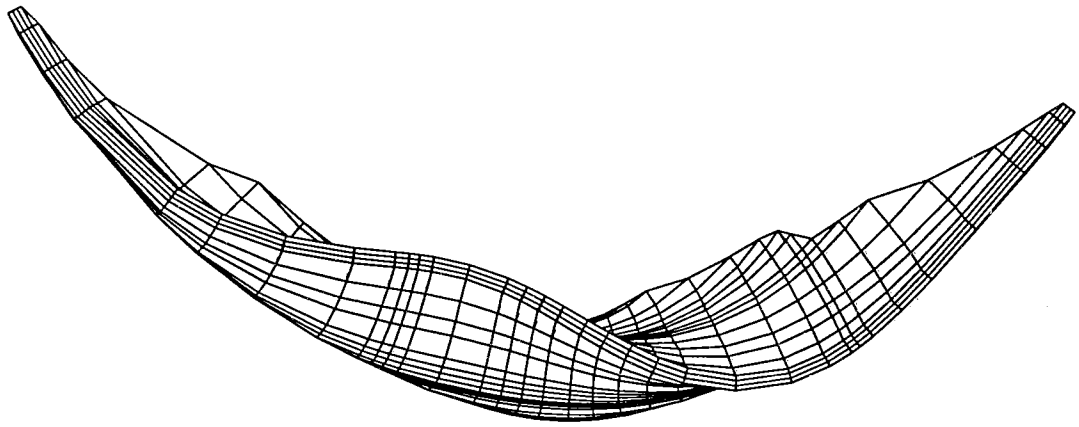
Undeformed Shape – ADAP Linear Model



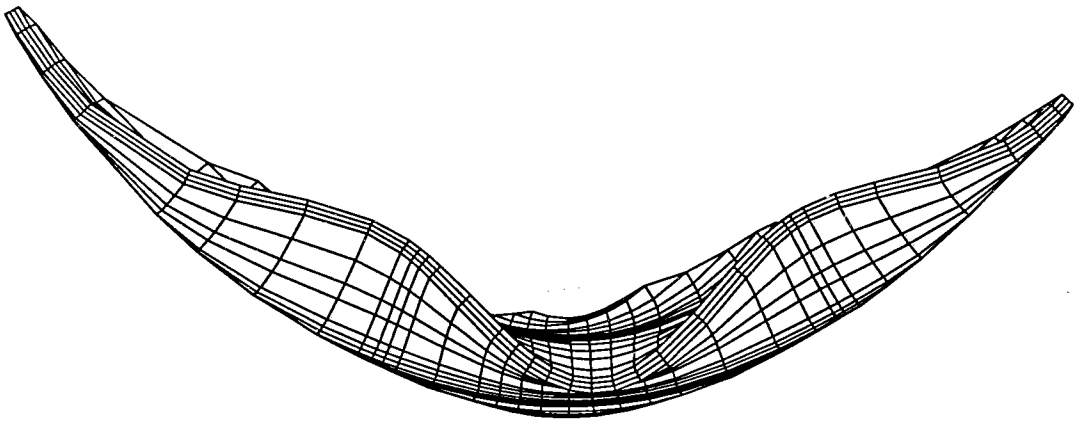
Mode 1 – 5.49 Hz, ADAP Linear Model

HARZA

Figure 5-11

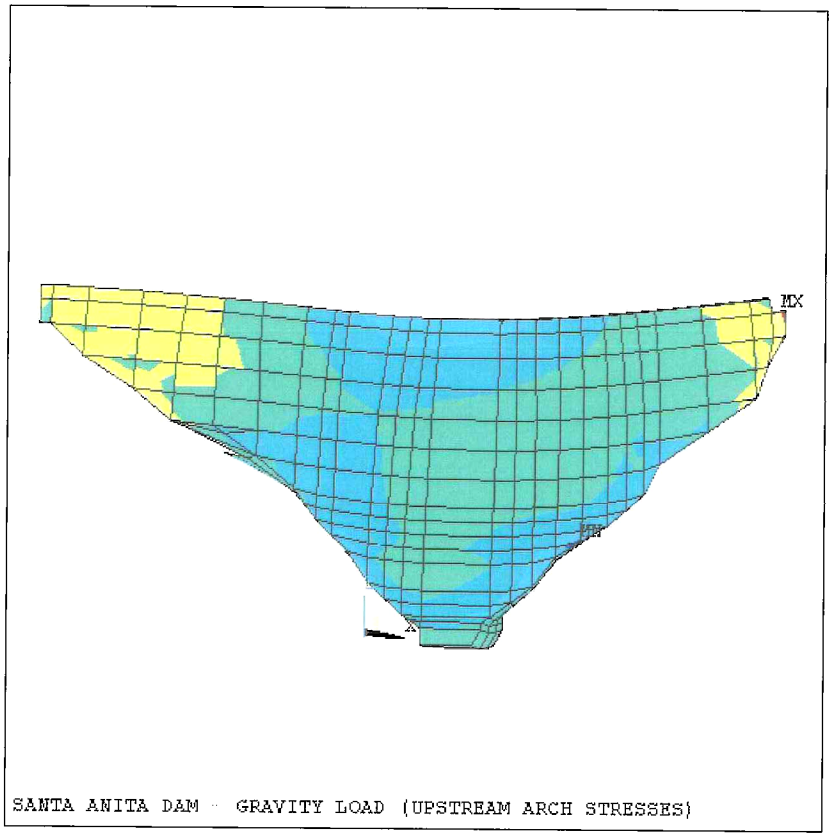


Mode 2 – 6.41 Hz, ADAP Linear Model



Mode 3 – 8.37 Hz, ADAP Linear Model

HARZA
Figure 5-12



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SMX =13138

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-6500
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6000
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11000
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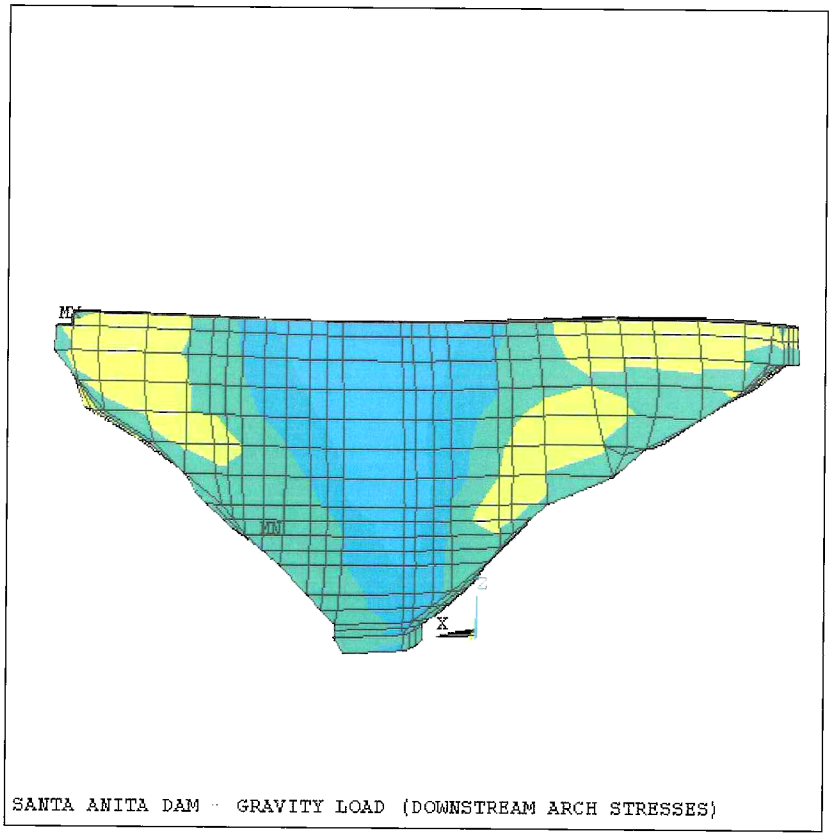
ALL VALUES
IN PSF

SANTA ANITA DAM - GRAVITY LOAD (UPSTREAM ARCH STRESSES)



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ALL VALUES
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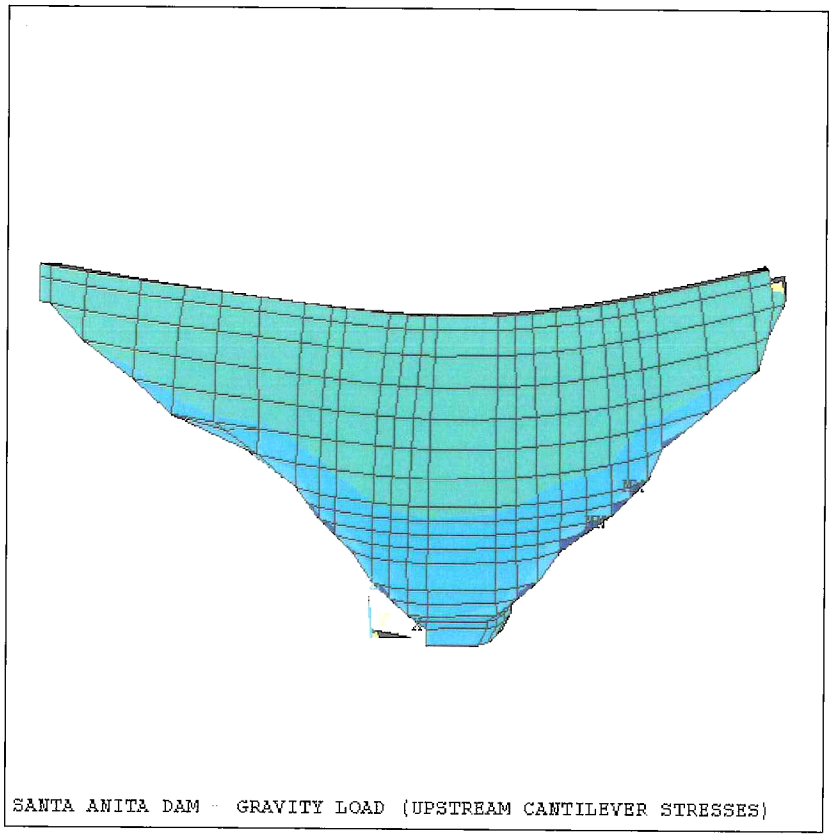
SANTA ANITA DAM - GRAVITY LOAD (DOWNSTREAM ARCH STRESSES)

HARZA
Figure 5-13



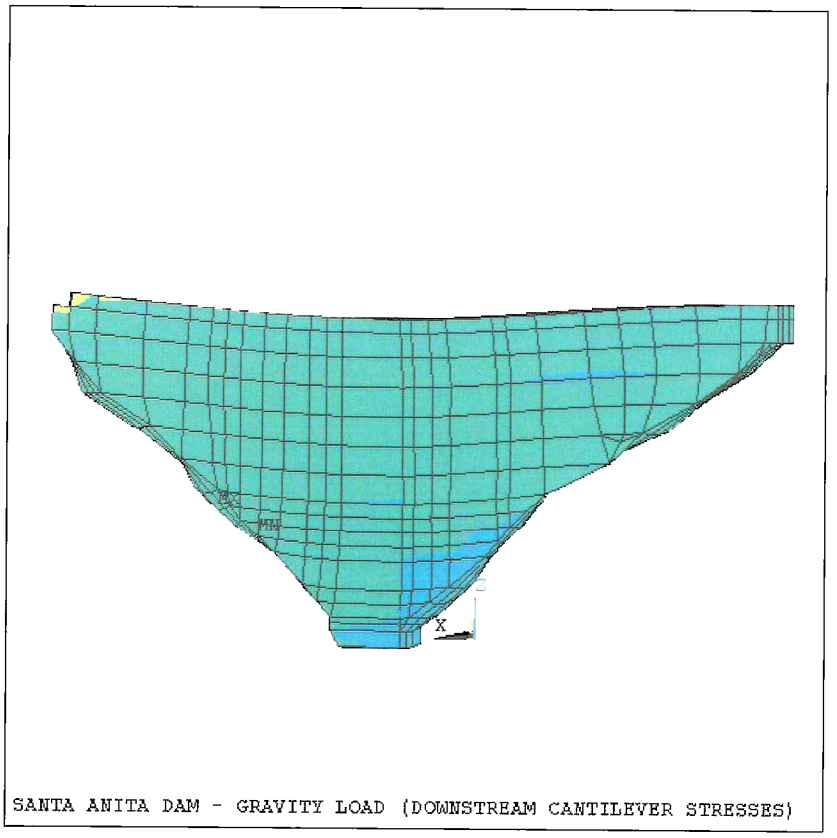


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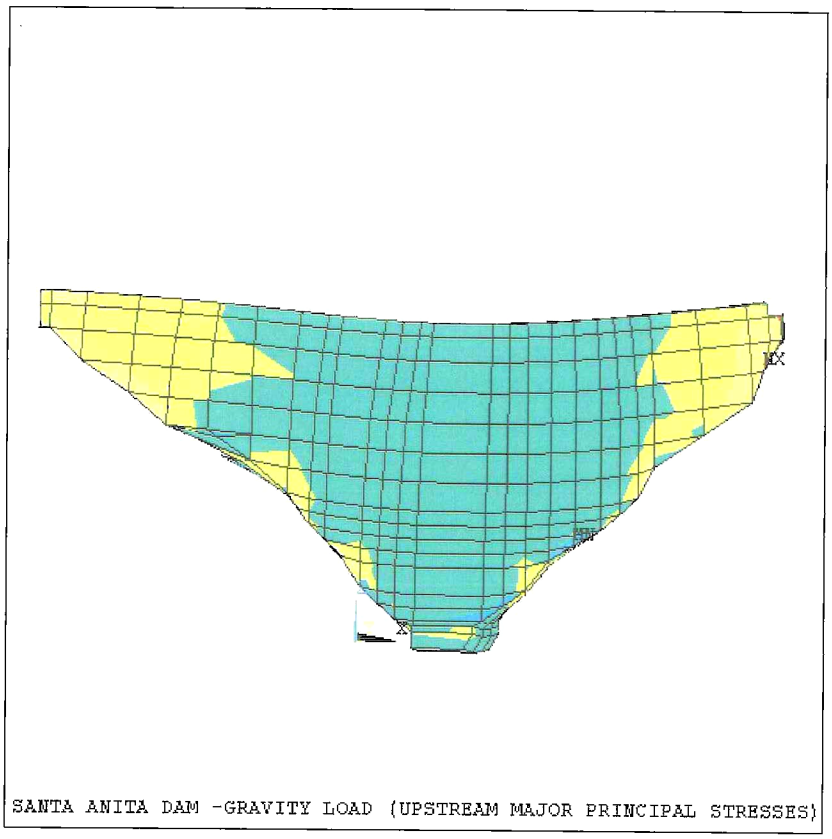
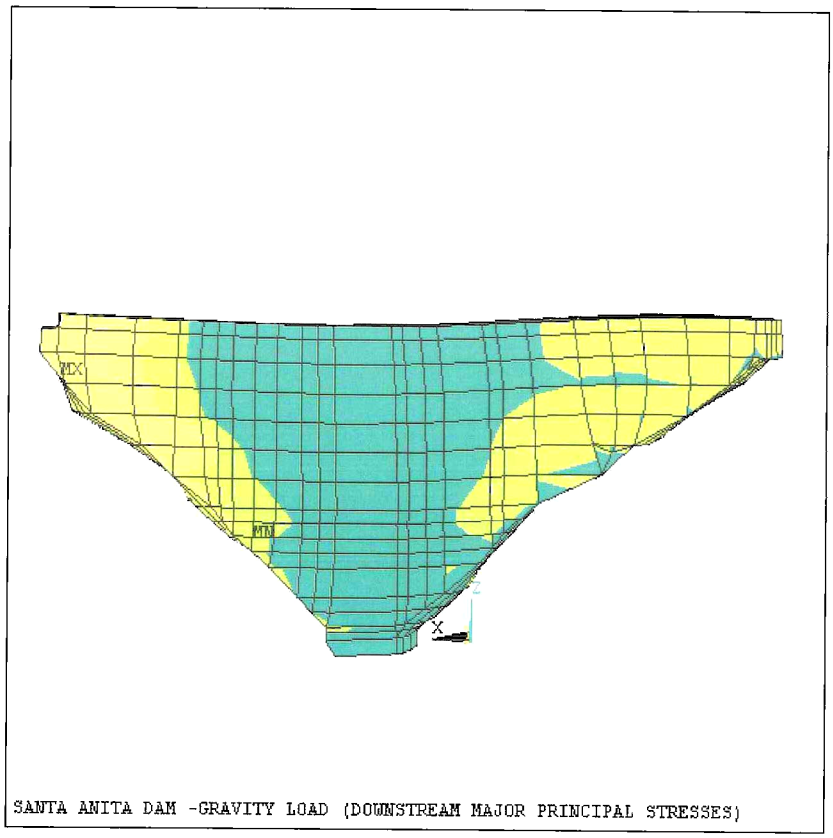
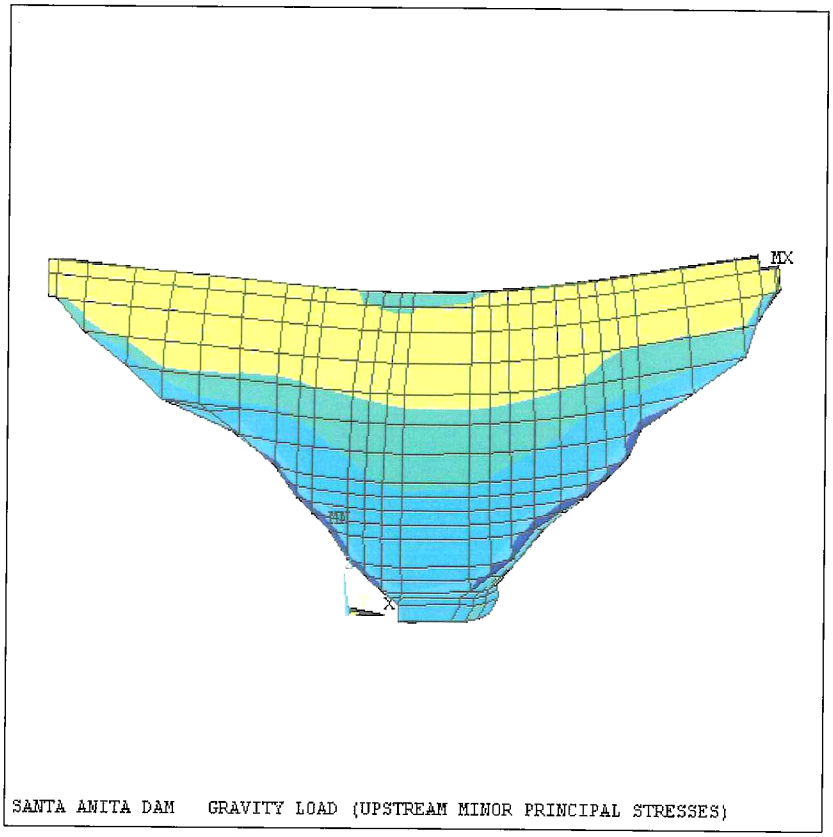
Figure 5-14

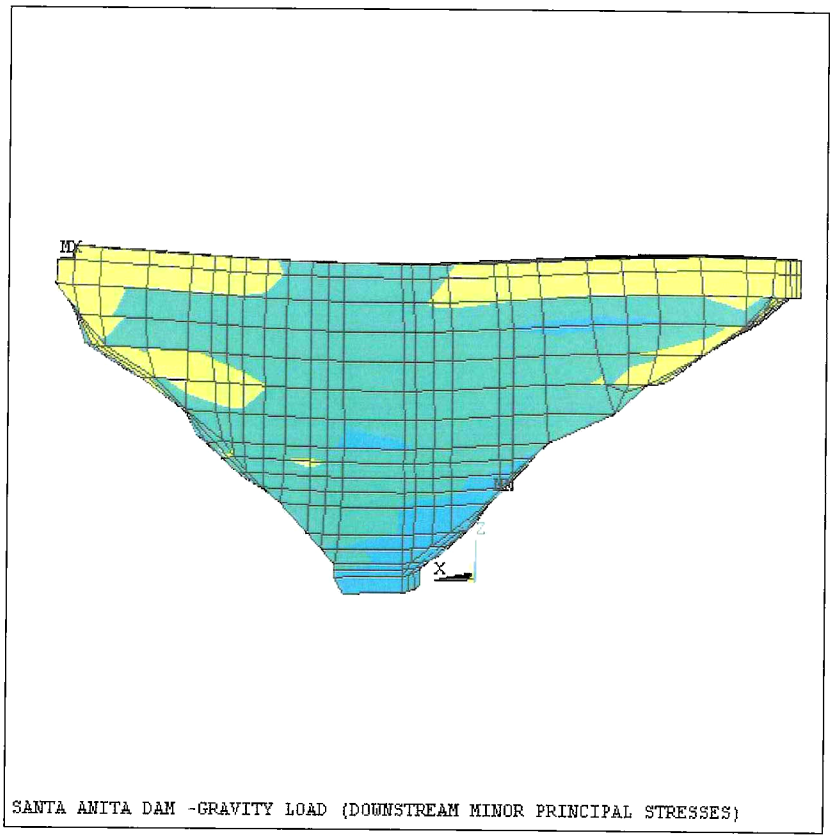




HARZA

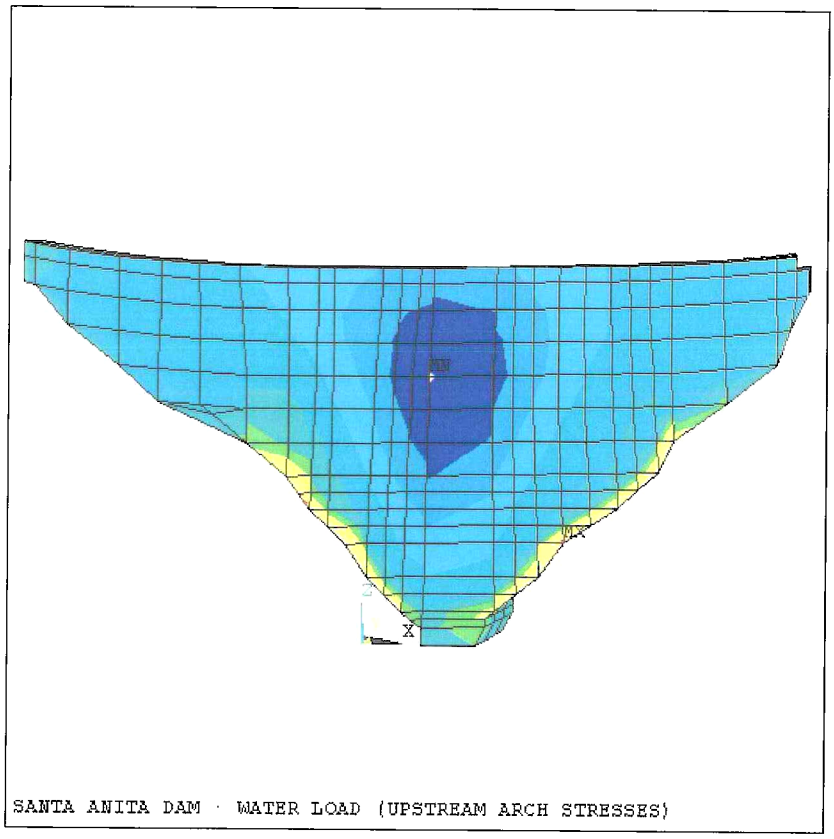
Figure 5-15

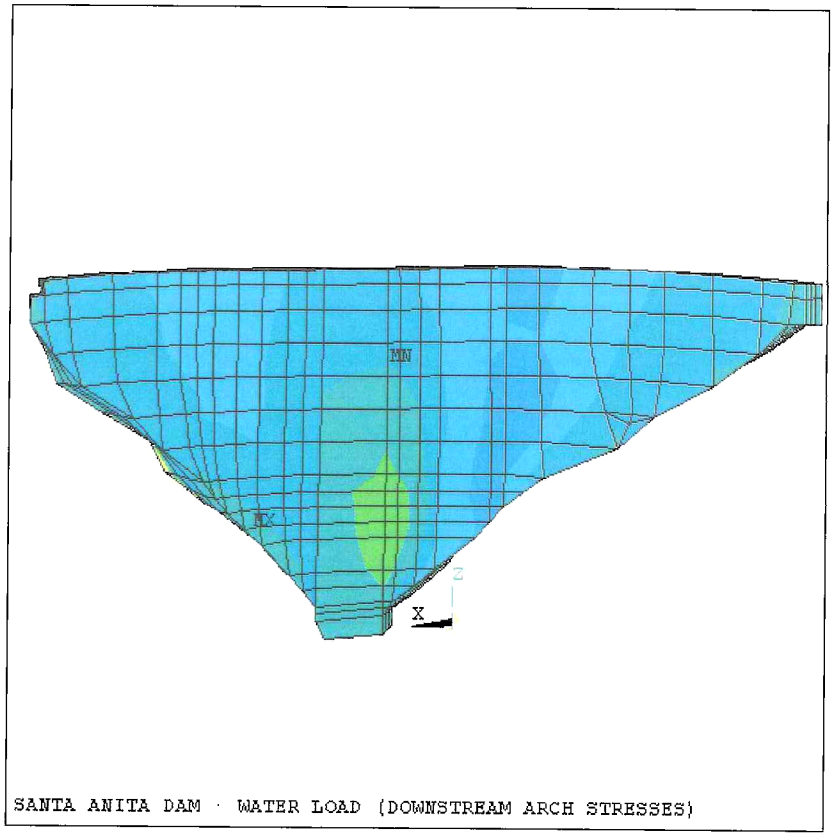




HARZA

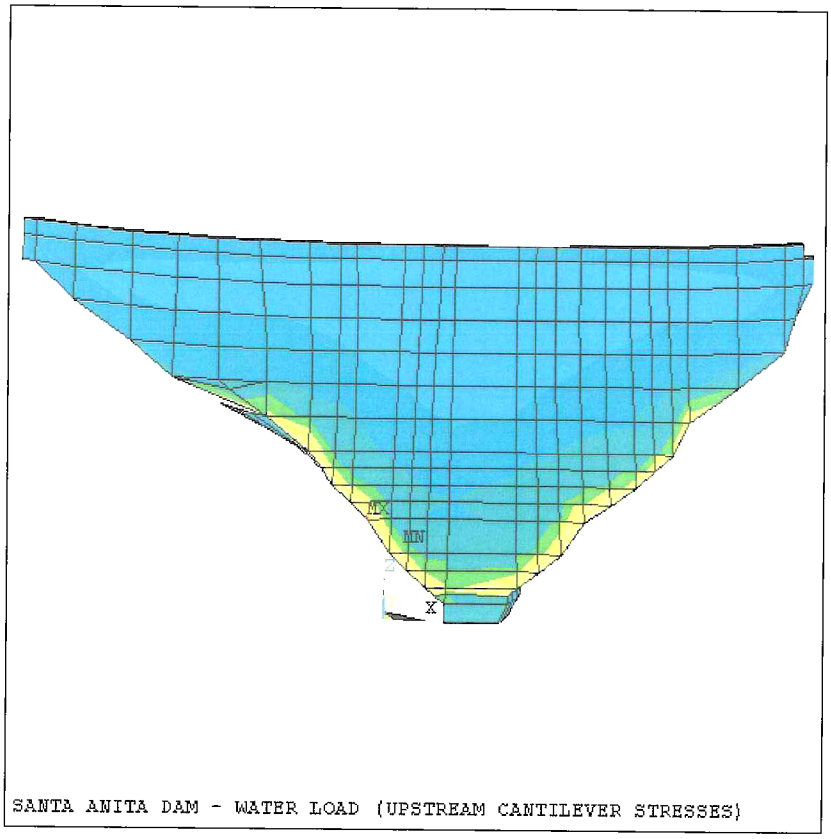
Figure 5-16

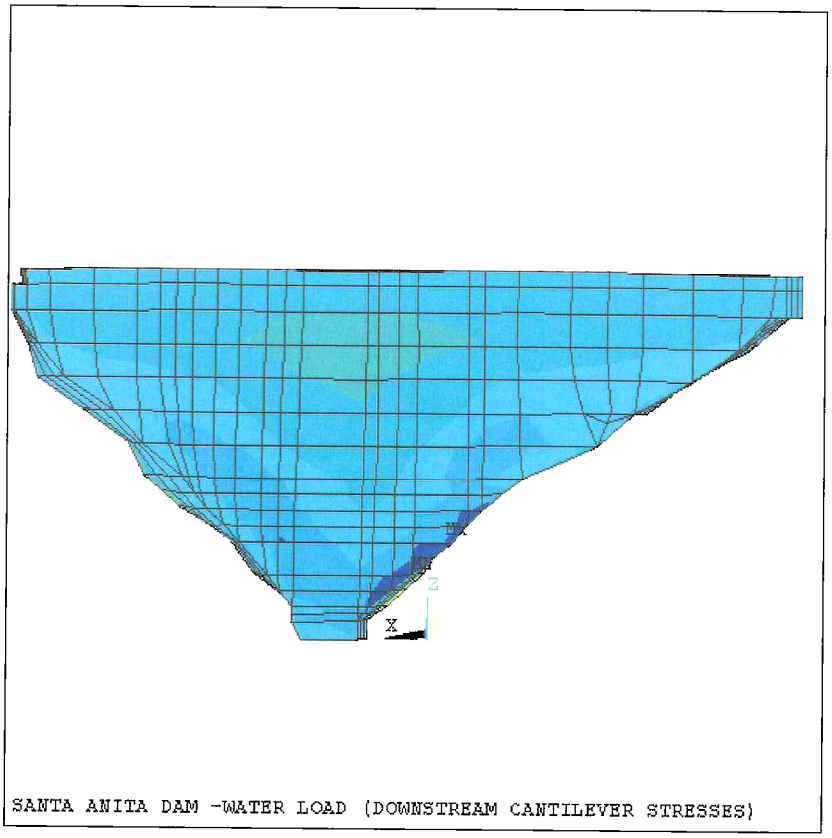




HARZA

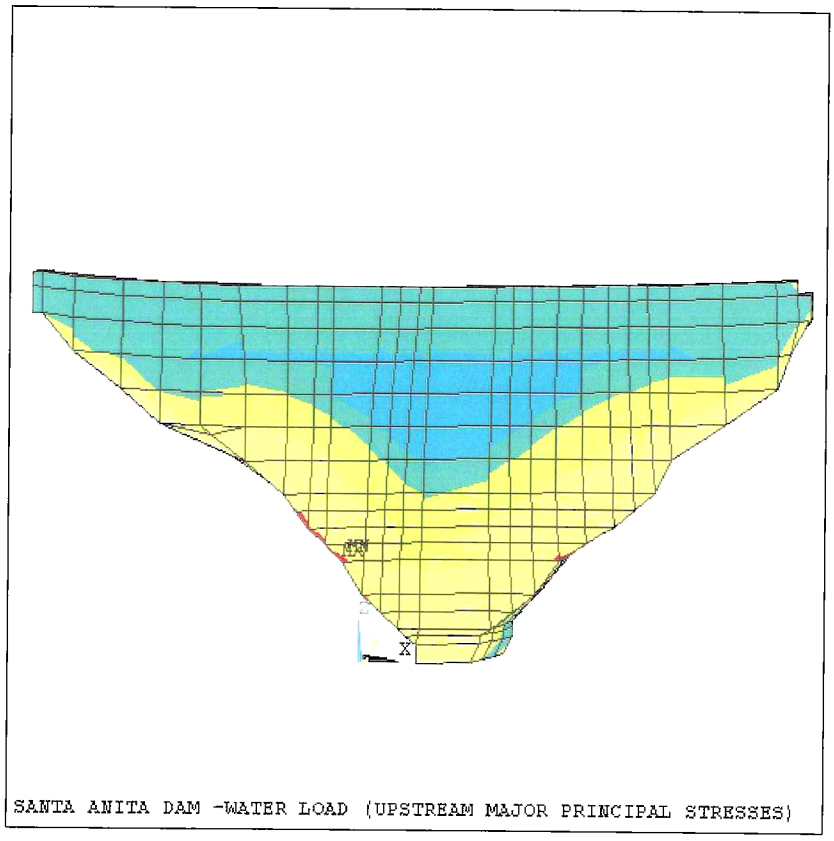
Figure 5-17



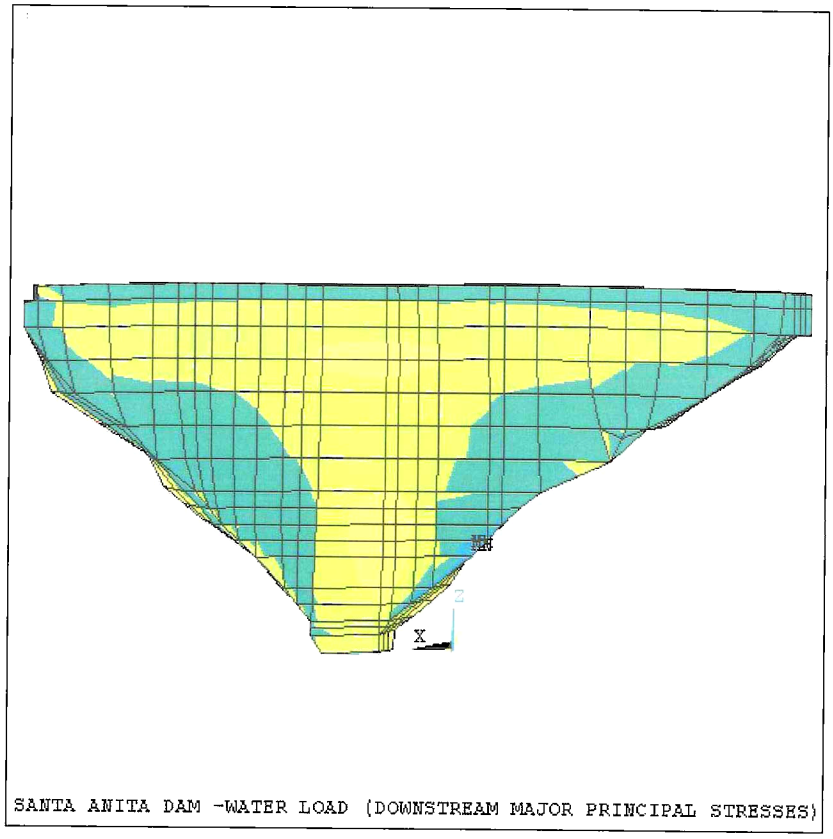


HARZA

Figure 5-18



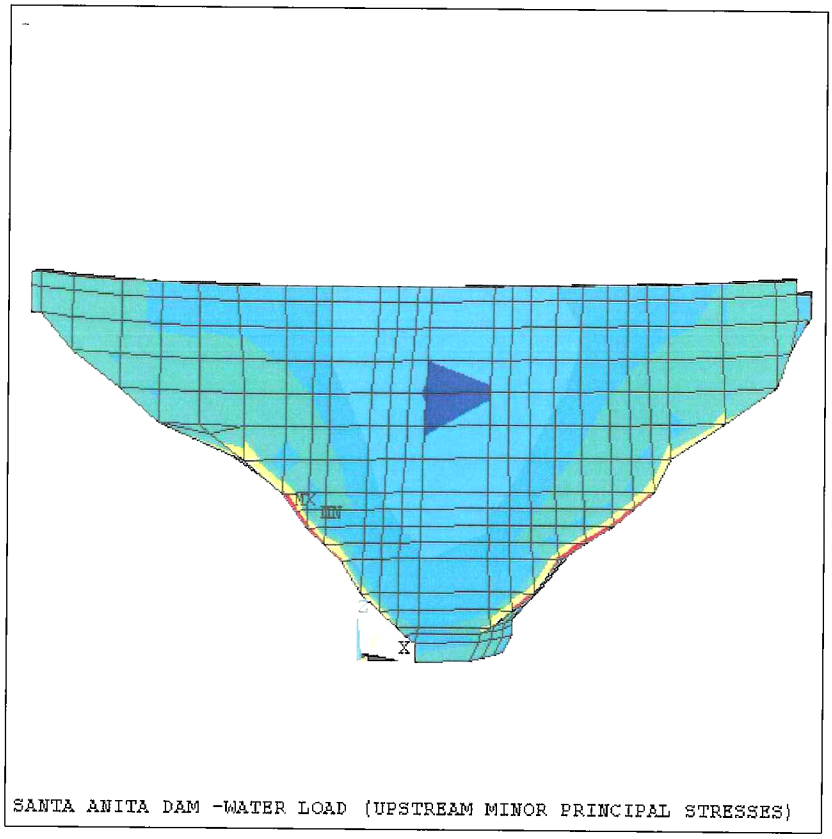
SANTA ANITA DAM -WATER LOAD (UPSTREAM MAJOR PRINCIPAL STRESSES)

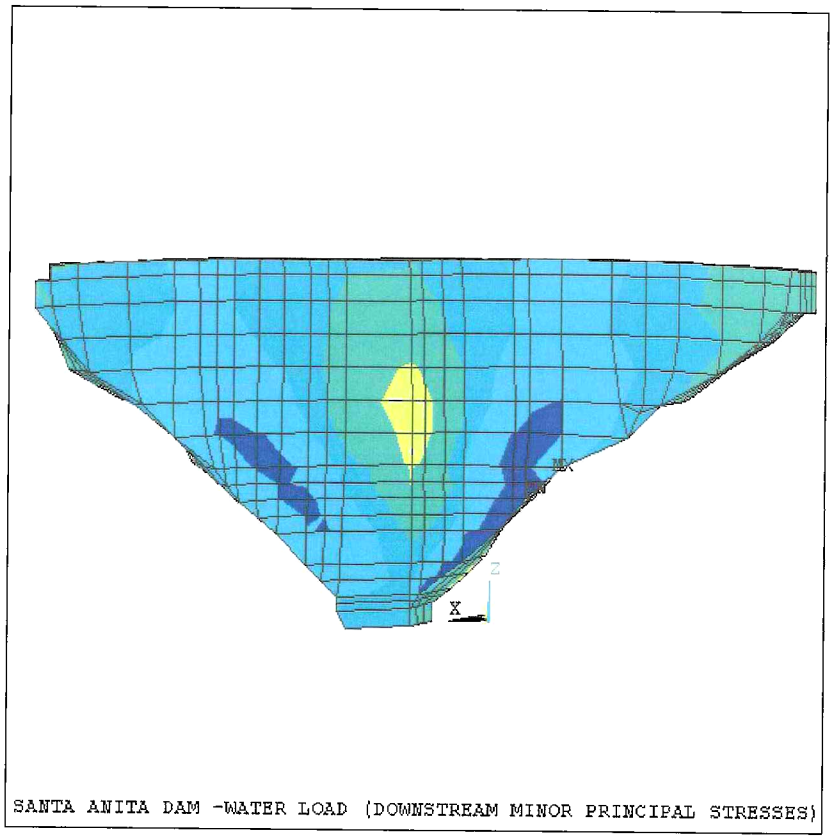


SANTA ANITA DAM -WATER LOAD (DOWNSTREAM MAJOR PRINCIPAL STRESSES)

HARZA

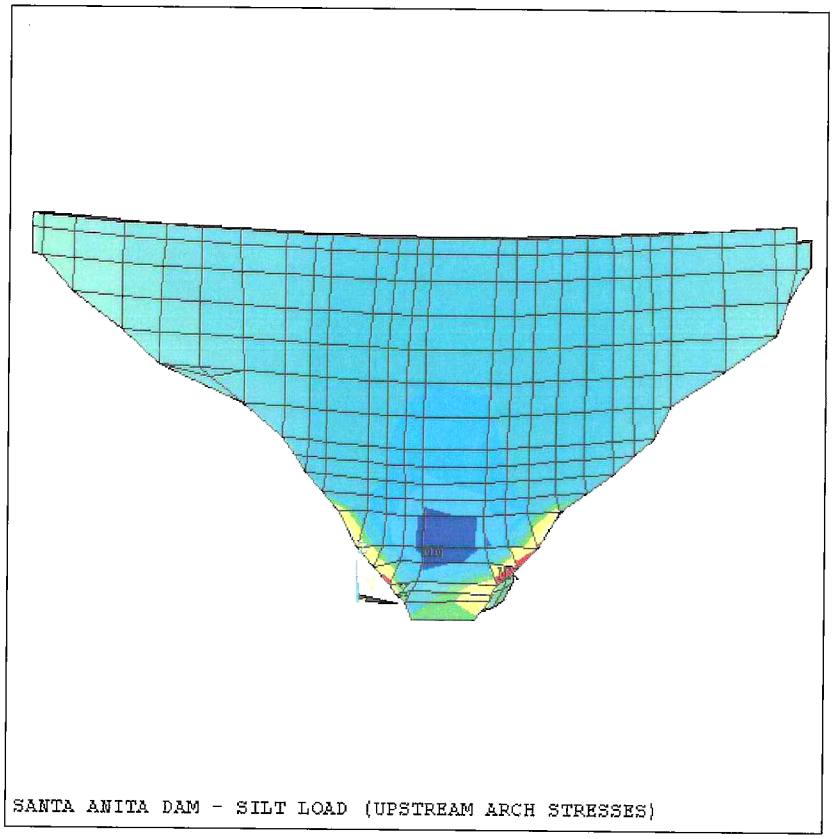
Figure 5-19

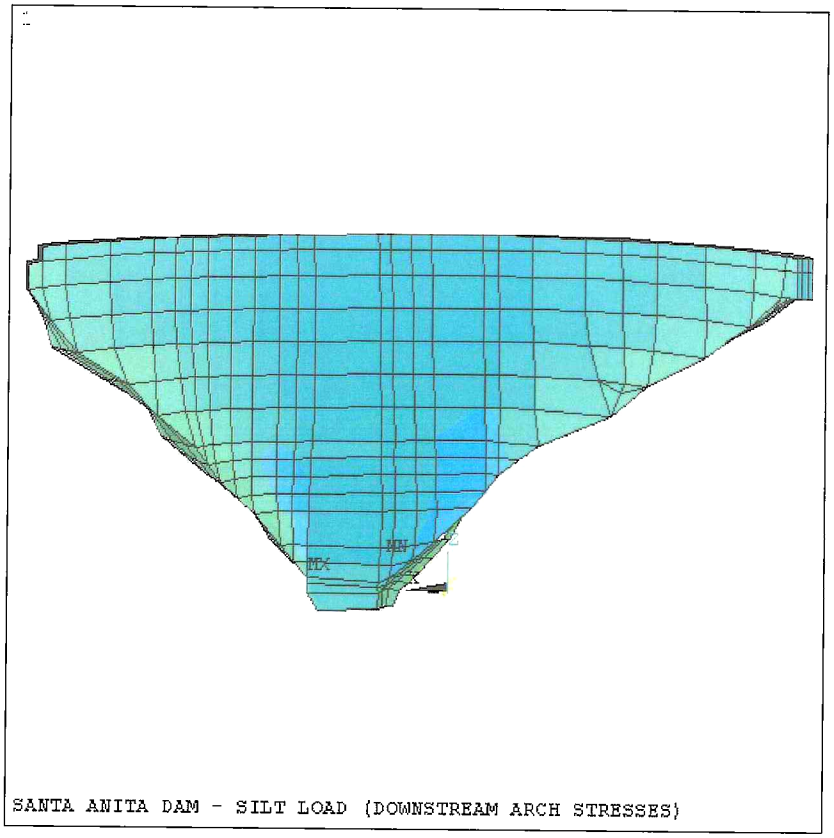




HARZA

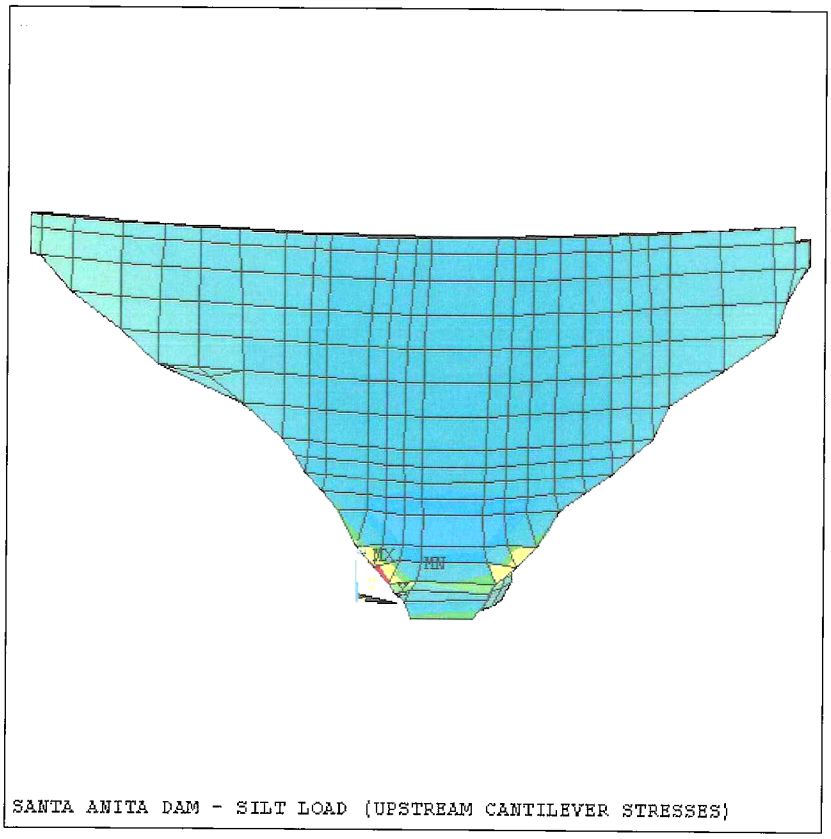
Figure 5-20

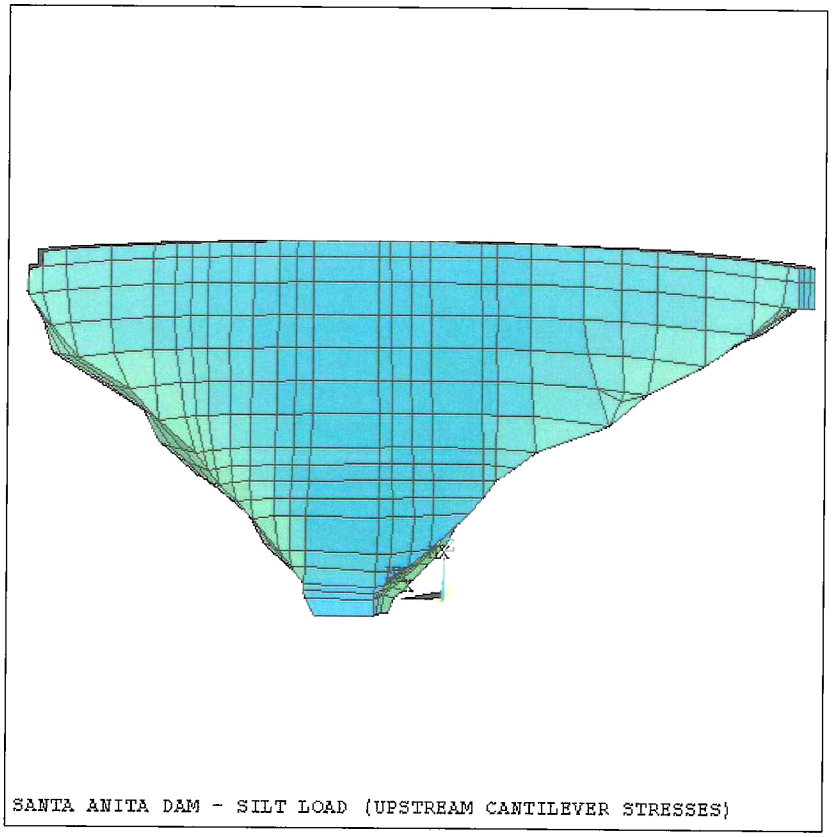




HARZA

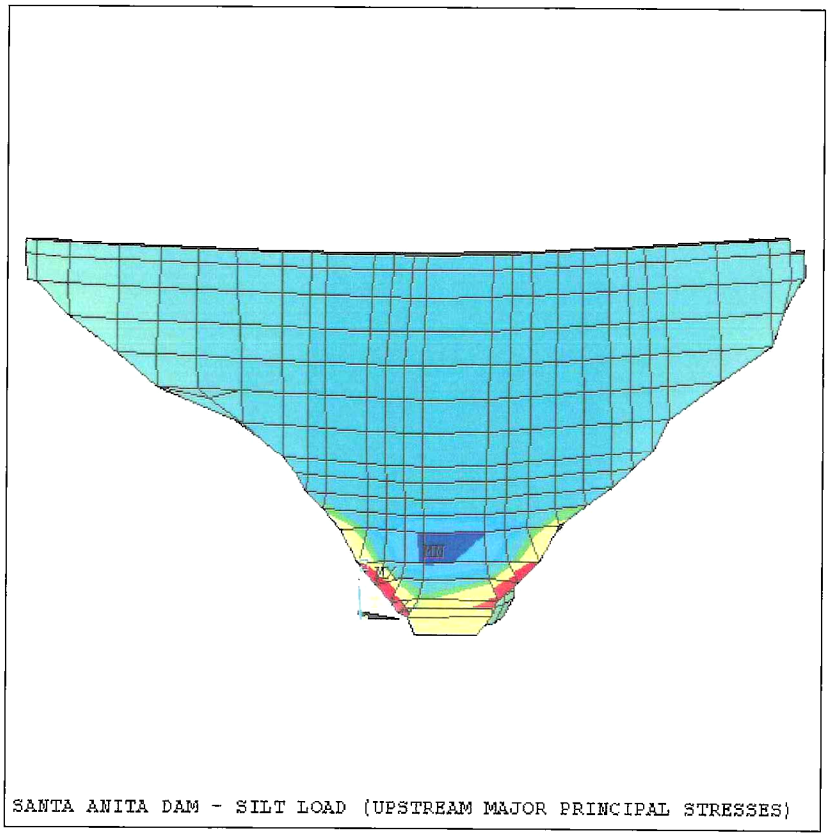
Figure 5-21



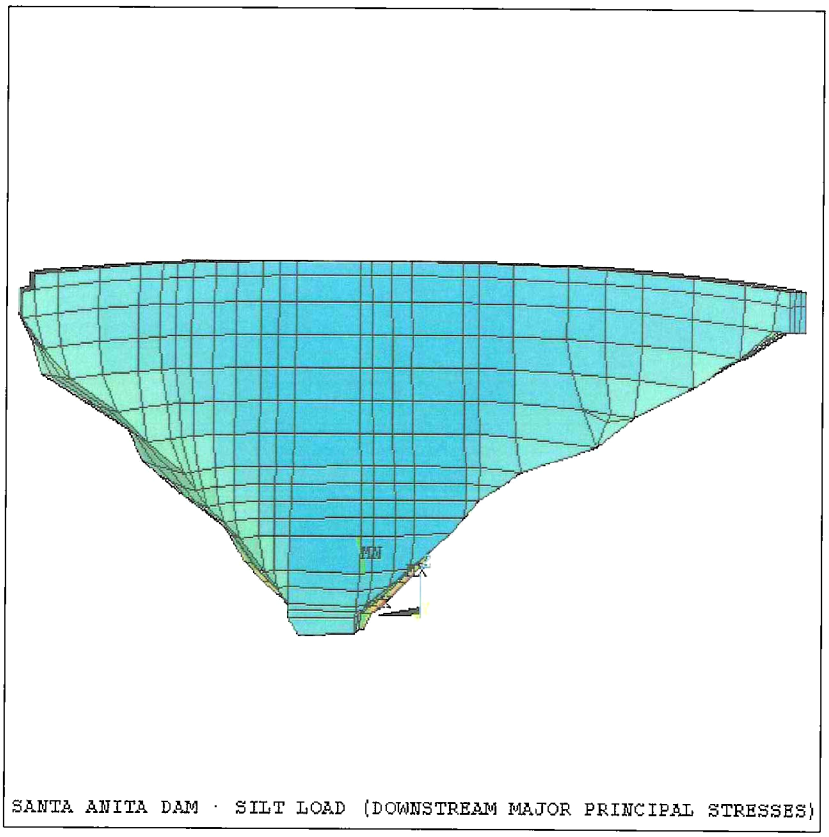


HARZA

Figure 5-22



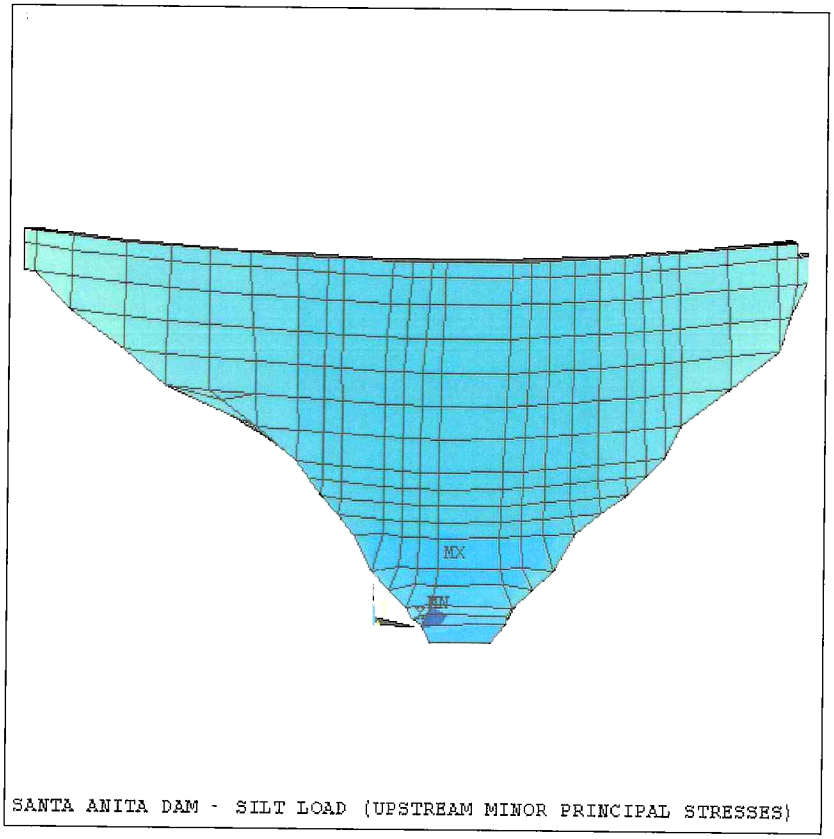
SANTA ANITA DAM - SILT LOAD (UPSTREAM MAJOR PRINCIPAL STRESSES)

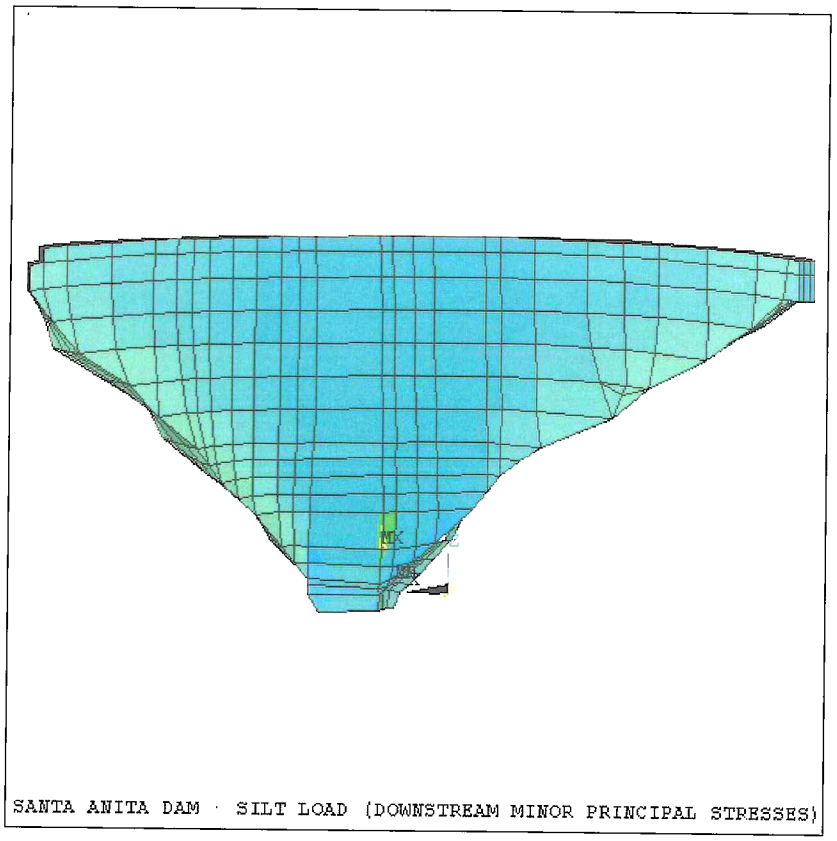


SANTA ANITA DAM - SILT LOAD (DOWNSTREAM MAJOR PRINCIPAL STRESSES)

HARZA

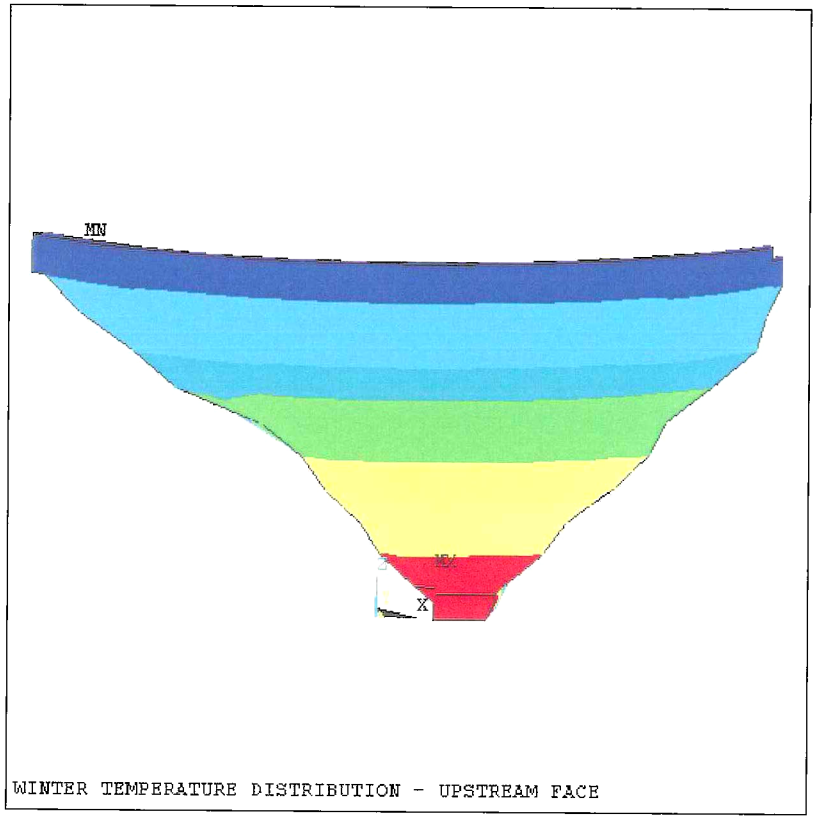
Figure 5-23



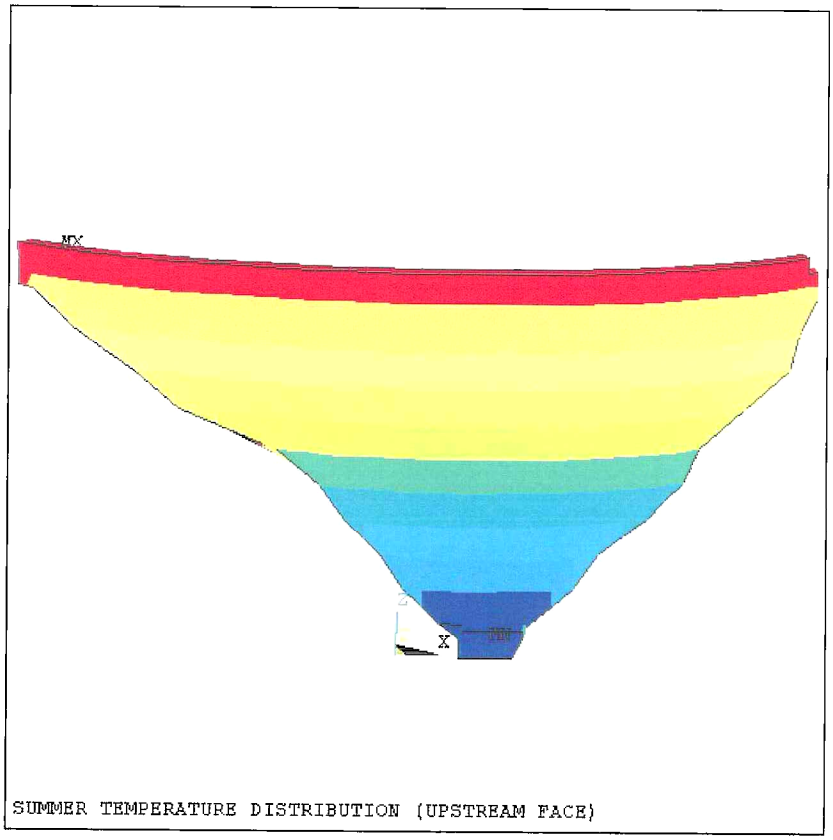


HARZA

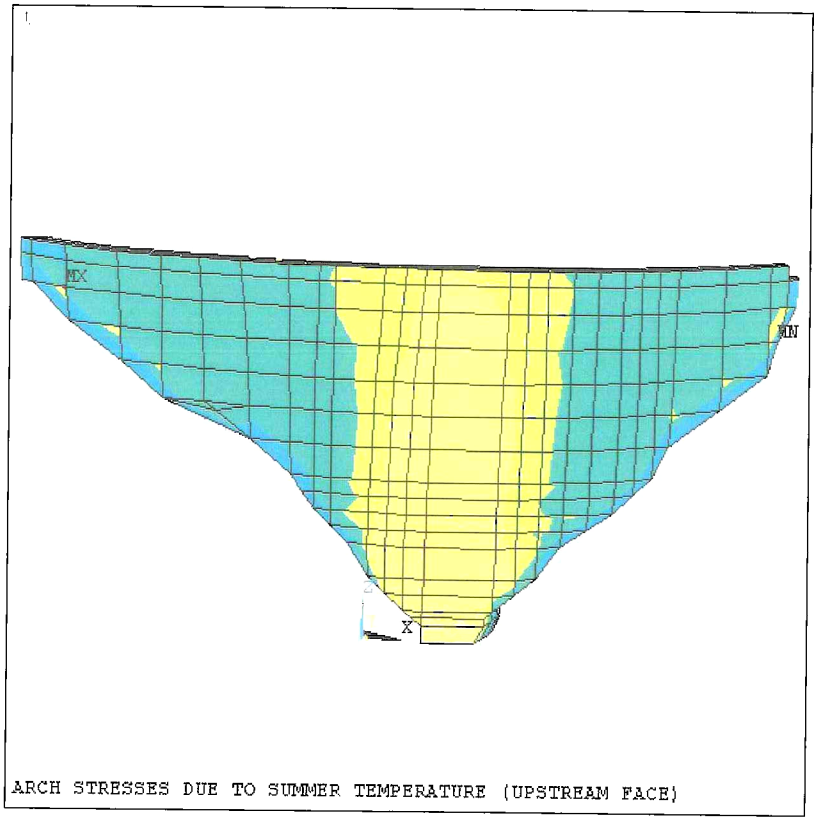
Figure 5-24



WINTER TEMPERATURE DISTRIBUTION - UPSTREAM FACE



SUMMER TEMPERATURE DISTRIBUTION (UPSTREAM FACE)



ANSYS 5 6
FEB 17 2000
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ELEMENT SOLUTION
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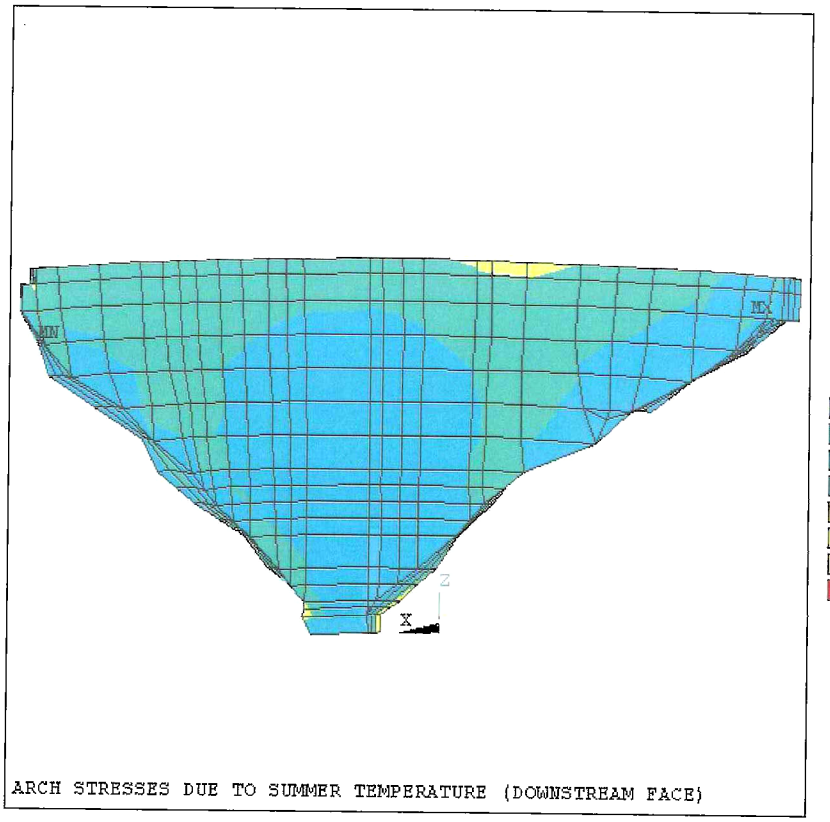
SUB =1
TIME=1
SX (NOAVG)
RSYS=0
PowerGraphics
EFACET=1
DMX =.054233
SMN =-178707
SMX =54841

-178707
-150000
-80000
-15000
10000
20000
30000
40000
55000

ALL VALUES IN PSF

ARCH STRESSES DUE TO SUMMER TEMPERATURE (UPSTREAM FACE)



ANSYS 5.6
FEB 17 2000
11:51:19
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SUB =1
TIME=1
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RSYS=0
PowerGraphics
EFACET=1
DMX =.054233
SMN =-178707
SMX =54841

-178707
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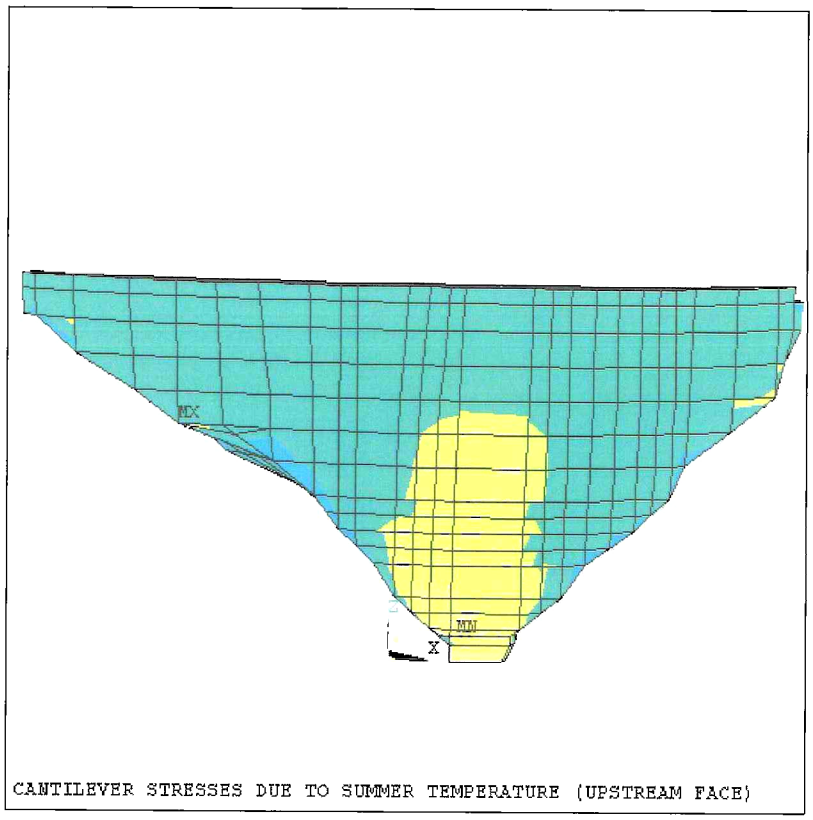
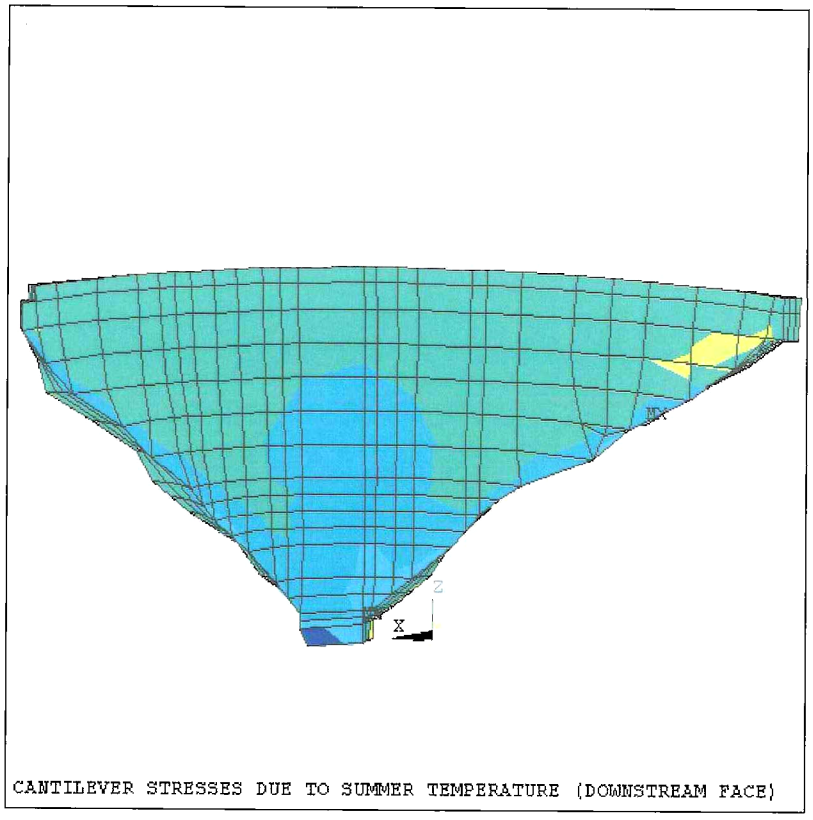
-80000
-15000
10000
20000
30000
40000
55000

ALL VALUES IN PSF

ARCH STRESSES DUE TO SUMMER TEMPERATURE (DOWNSTREAM FACE)



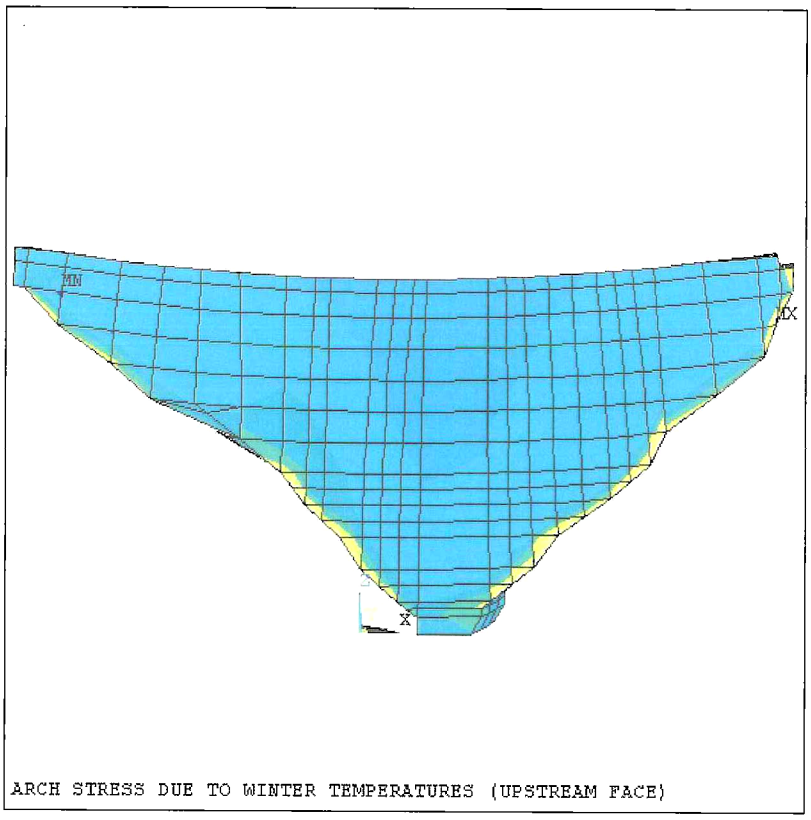
CANTILEVER STRESSES DUE TO SUMMER TEMPERATURE (UPSTREAM FACE)



CANTILEVER STRESSES DUE TO SUMMER TEMPERATURE (DOWNSTREAM FACE)

HARZA

Figure 5-27



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16:26:21
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TIME=1
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RSYS=0
PowerGraphics
EFACET=1
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SMX =227936

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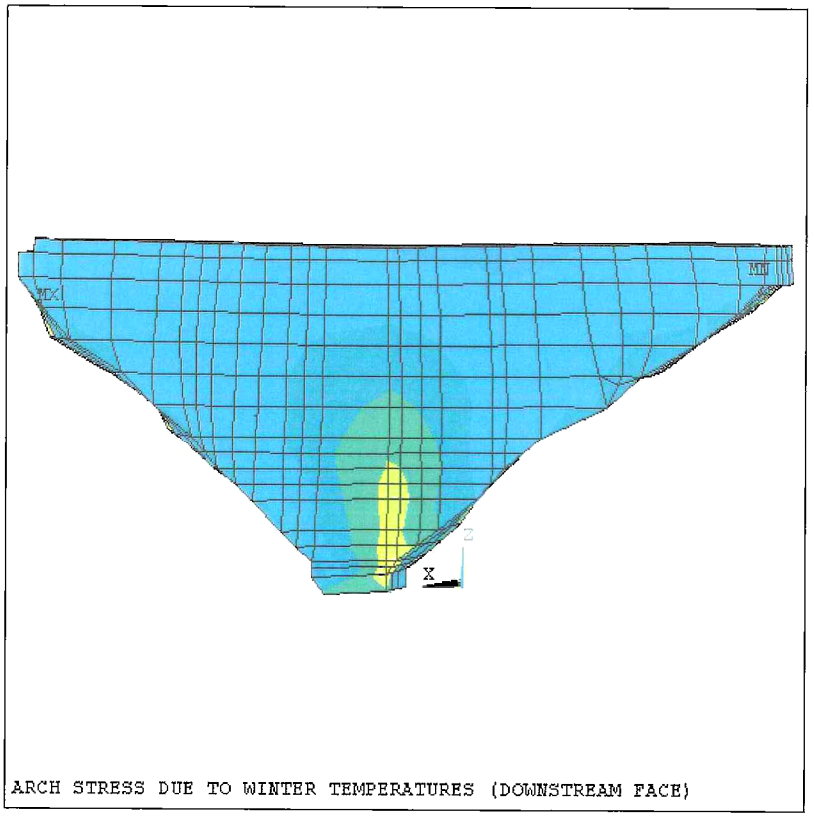
-2000
27000
56000
85000
114000
143000
172000
201000
230000

ALL VALUES IN PSF

ARCH STRESS DUE TO WINTER TEMPERATURES (UPSTREAM FACE)



ANSYS 5 6
MAR 30 2000
09:40:51
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TIME=1
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RSYS=0
PowerGraphics
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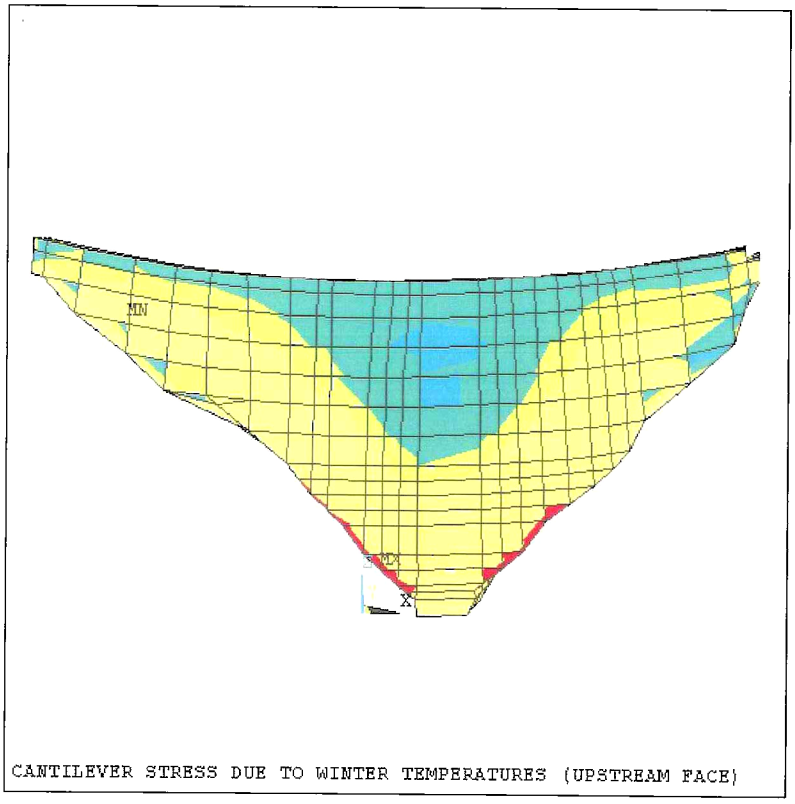
SMN =-59586
SMX =227936

-60000
-31000
-2000
27000
56000
85000
114000
143000
172000
201000
230000

ALL VALUES IN PSF

ARCH STRESS DUE TO WINTER TEMPERATURES (DOWNSTREAM FACE)

HARZA
Figure 5-28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10:54:06
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TIME=1
SZ (NOAVG)
RSYS=0
PowerGraphics
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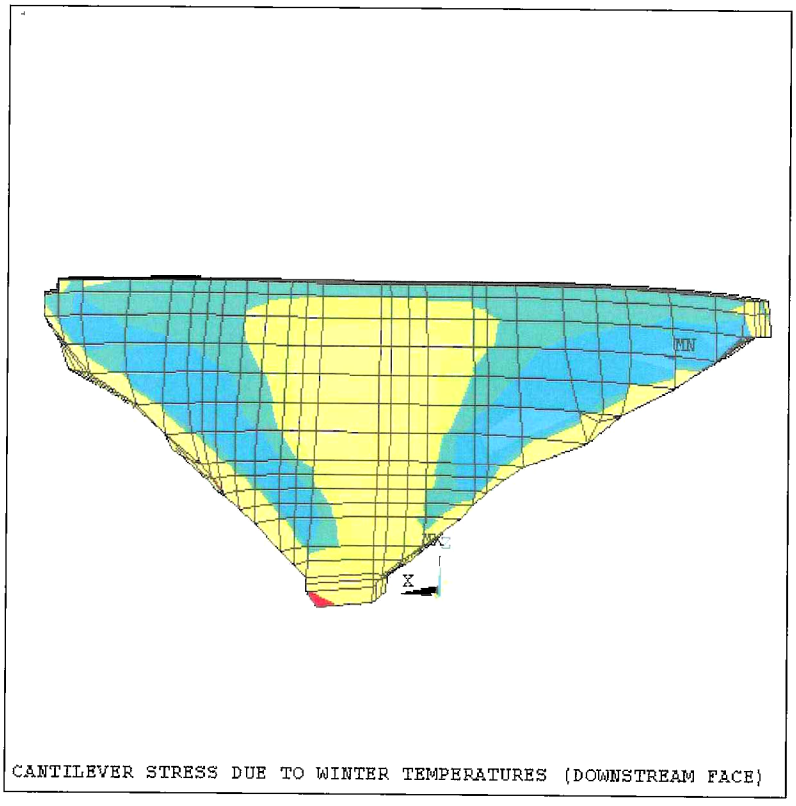
DMX =.049173
SMN =-38476
SMX =80905
-38476
-30000
-15000
-5000
1000
5000
15000
50000
80000
ALL VALUES IN PSF
MN
MX
CANTILEVER STRESS DUE TO WINTER TEMPERATURES (UPSTREAM FACE)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FEB 16 2000
10:50·58
ELEMENT SOLUTION
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TIME=1
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RSYS=0
PowerGraphics
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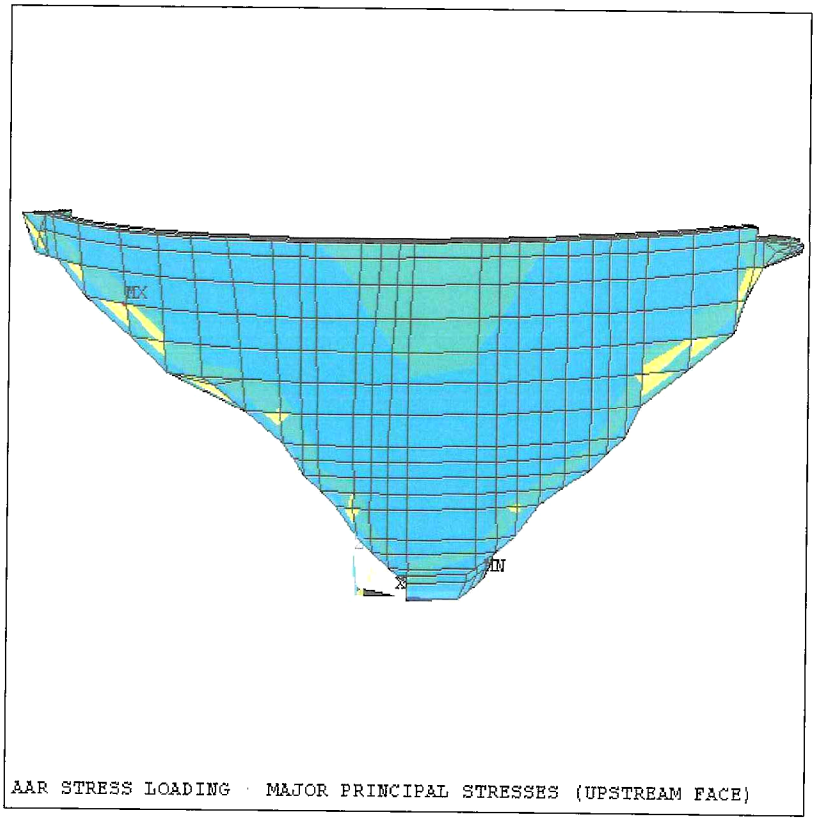
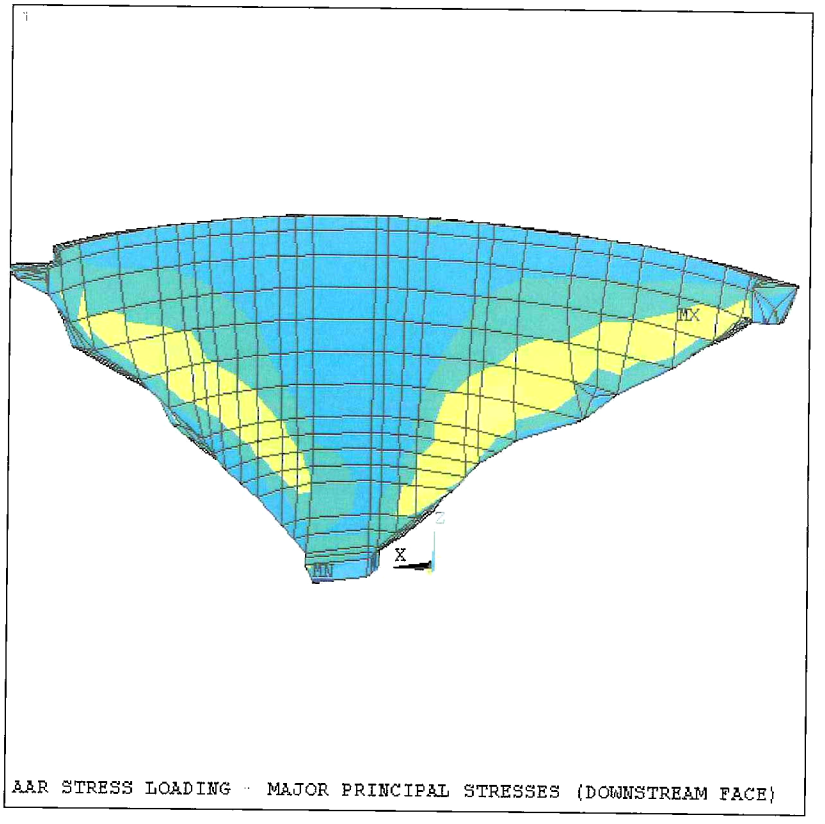
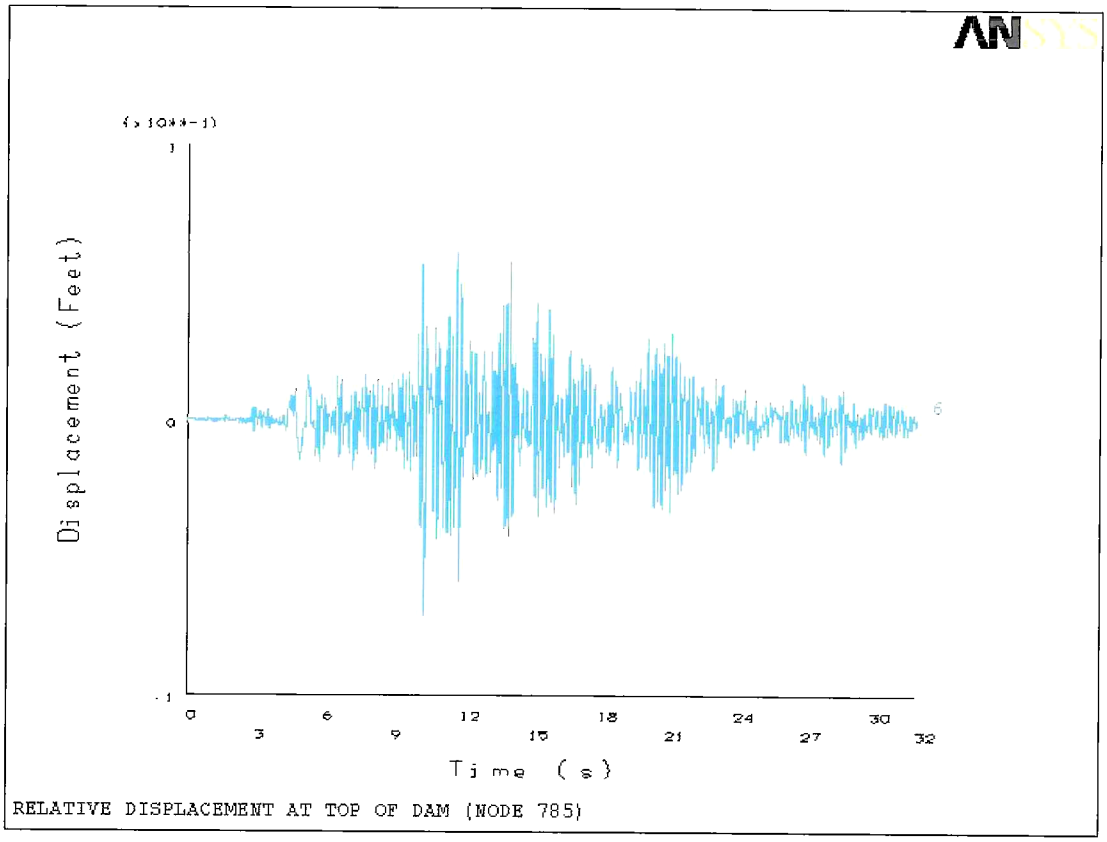
SMX =80905
-38476
-30000
-15000
-5000
1000
5000
15000
50000
80000
ALL VALUES IN PSF
MN
MX
CANTILEVER STRESS DUE TO WINTER TEMPERATURES (DOWNSTREAM FACE)



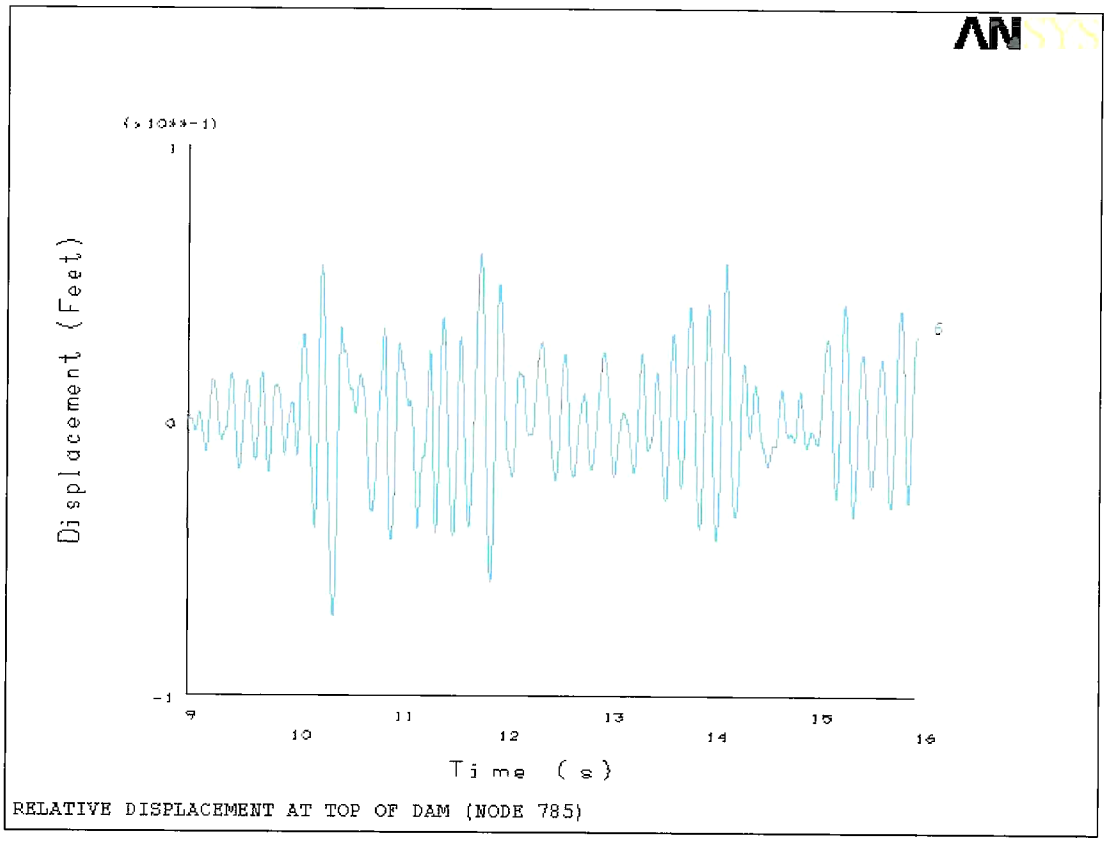


HARZA

Figure 5-30



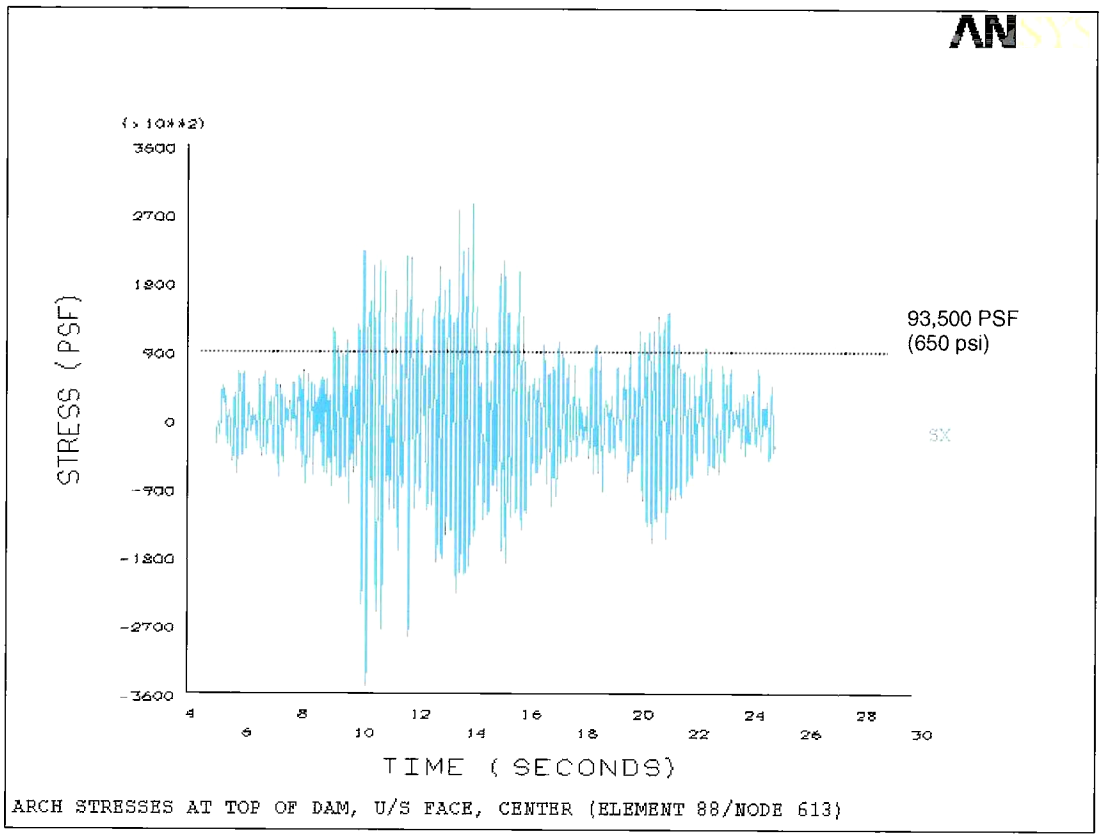
RELATIVE DISPLACEMENT AT TOP OF DAM (NODE 785)



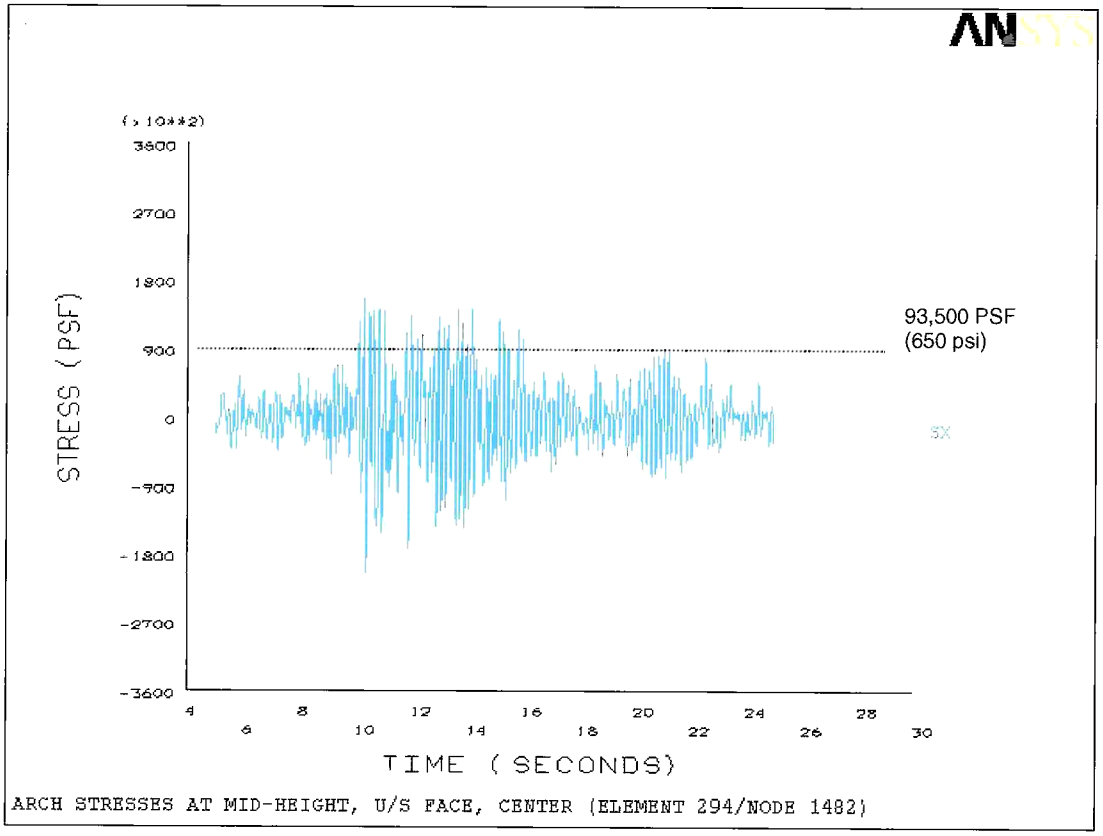
RELATIVE DISPLACEMENT AT TOP OF DAM (NODE 785)

HARZA

Figure 5-31



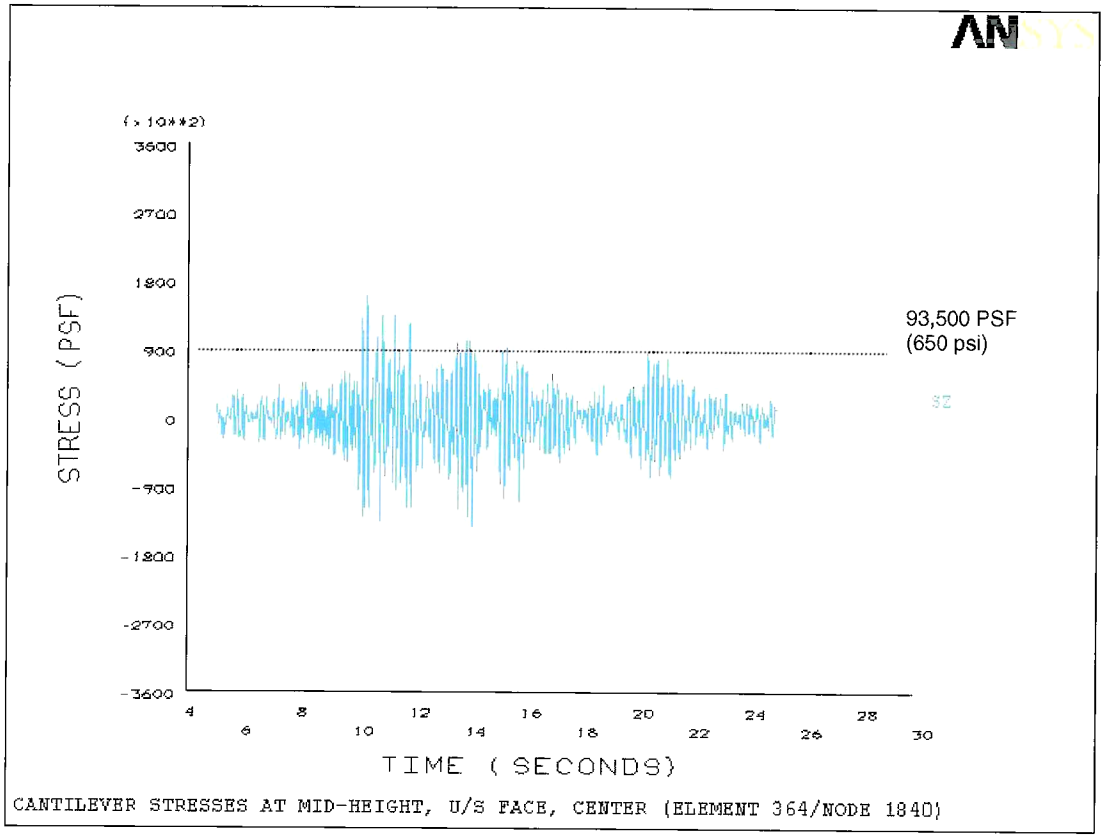
ARCH STRESSES AT TOP OF DAM, U/S FACE, CENTER (ELEMENT 88/NODE 613)



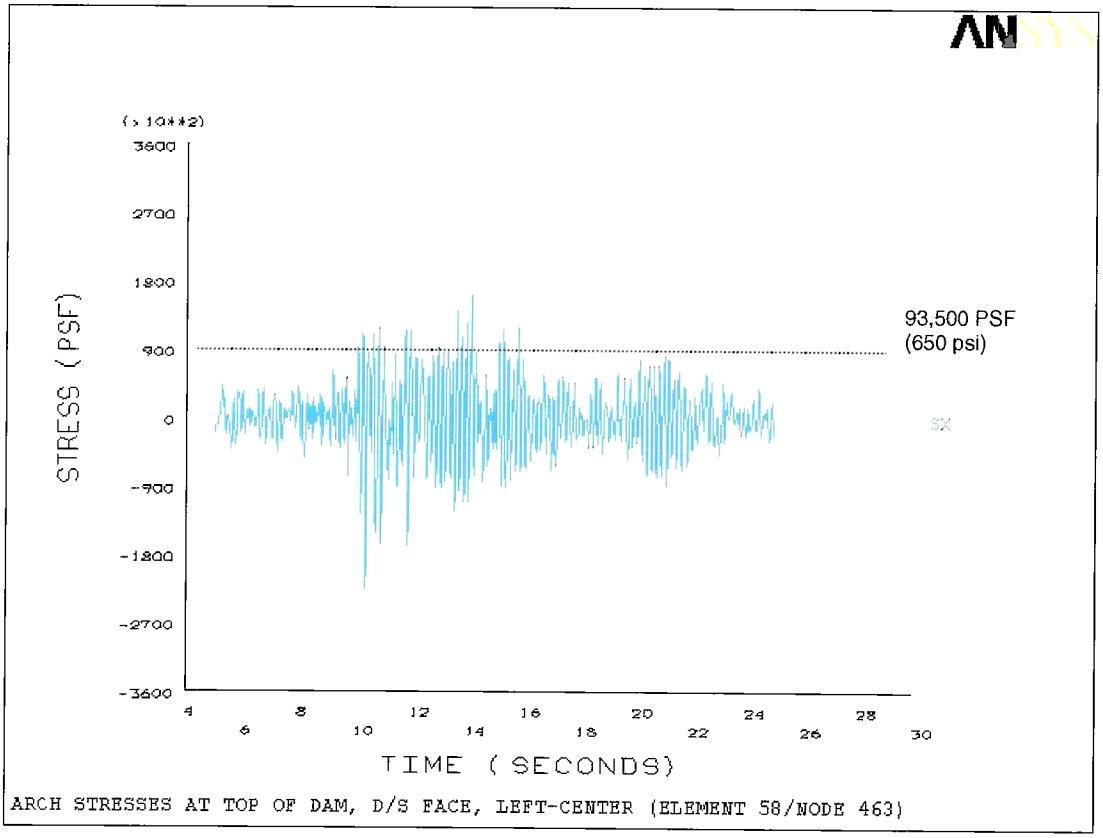
ARCH STRESSES AT MID-HEIGHT, U/S FACE, CENTER (ELEMENT 294/NODE 1482)

HARZA

Figure 5-32



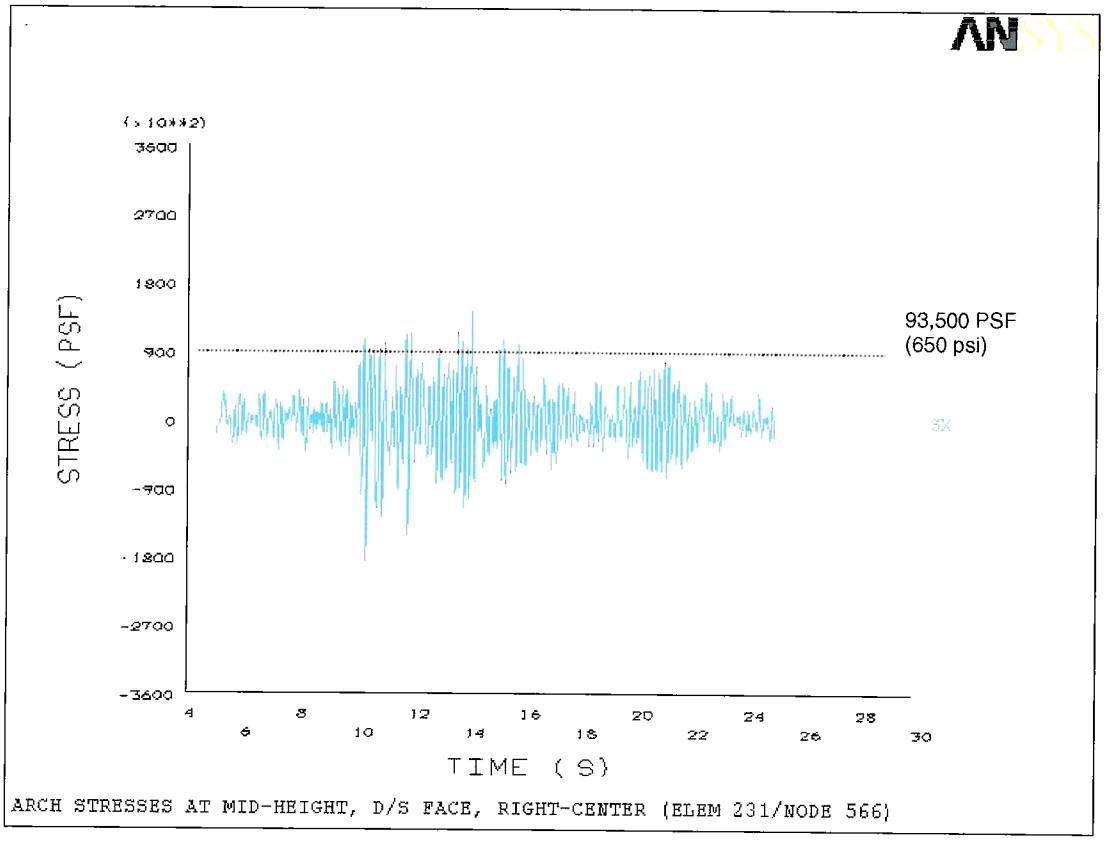
CANTILEVER STRESSES AT MID-HEIGHT, U/S FACE, CENTER (ELEMENT 364/NODE 1840)



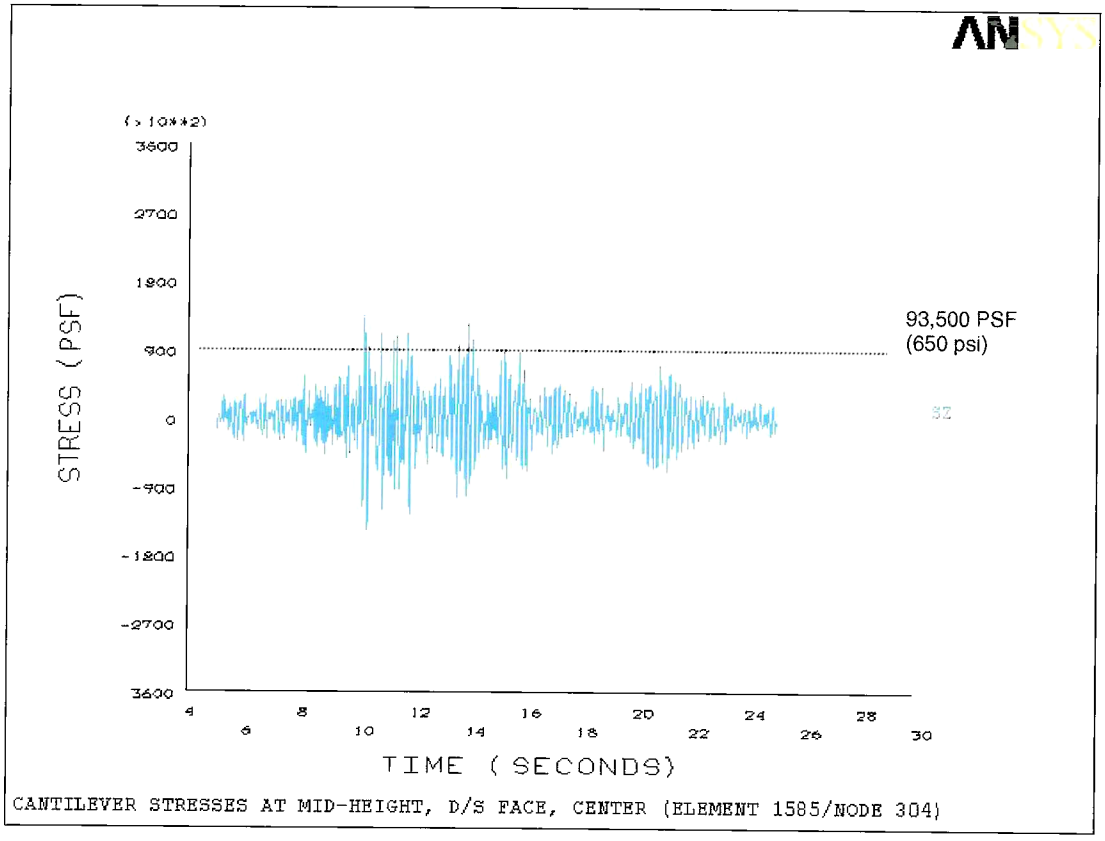
ARCH STRESSES AT TOP OF DAM, D/S FACE, LEFT-CENTER (ELEMENT 58/NODE 463)

HARZA

Figure 5-33



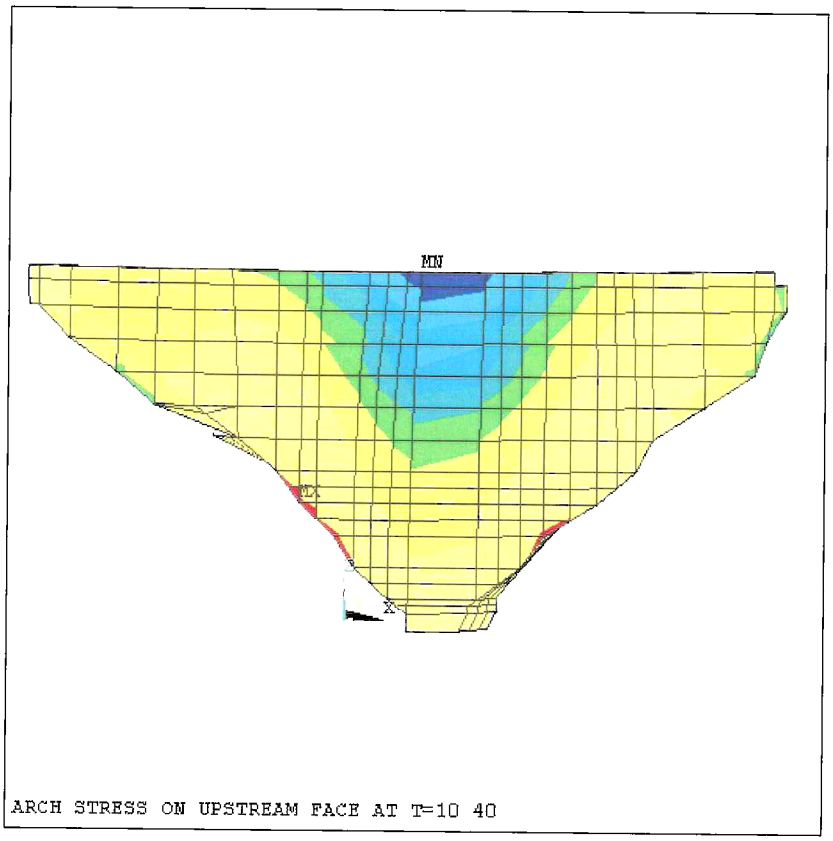
ARCH STRESSES AT MID-HEIGHT, D/S FACE, RIGHT-CENTER (ELEM 231/NODE 566)



CANTILEVER STRESSES AT MID-HEIGHT, D/S FACE, CENTER (ELEMENT 1585/NODE 304)

HARZA

Figure 5-34



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14:26:12
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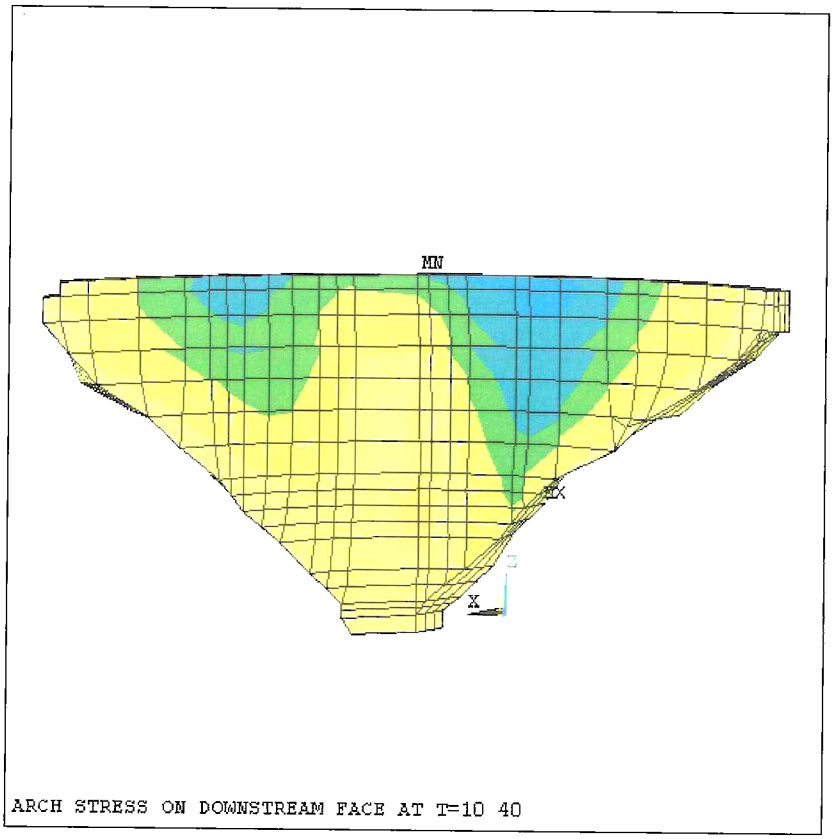
TIME=10 4
SX (NOAVG)
RSYS=0
PowerGraphics
EFACET=1
DMX =3.237
SMN =-350668
SMX =148870

-350668
-295164
-239660
-184156
-128651
-73147
-17643
37861
93365
148870

ALL VALUES IN PSF

ARCH STRESS ON UPSTREAM FACE AT T=10 40



ANSYS 5 6
FEB 10 2000
14:25:32
ELEMENT SOLUTION
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SUB =270
TIME=10 4
SX (NOAVG)
RSYS=0
PowerGraphics
EFACET=1
DMX =3.237
SMN =-350668
SMX =148870

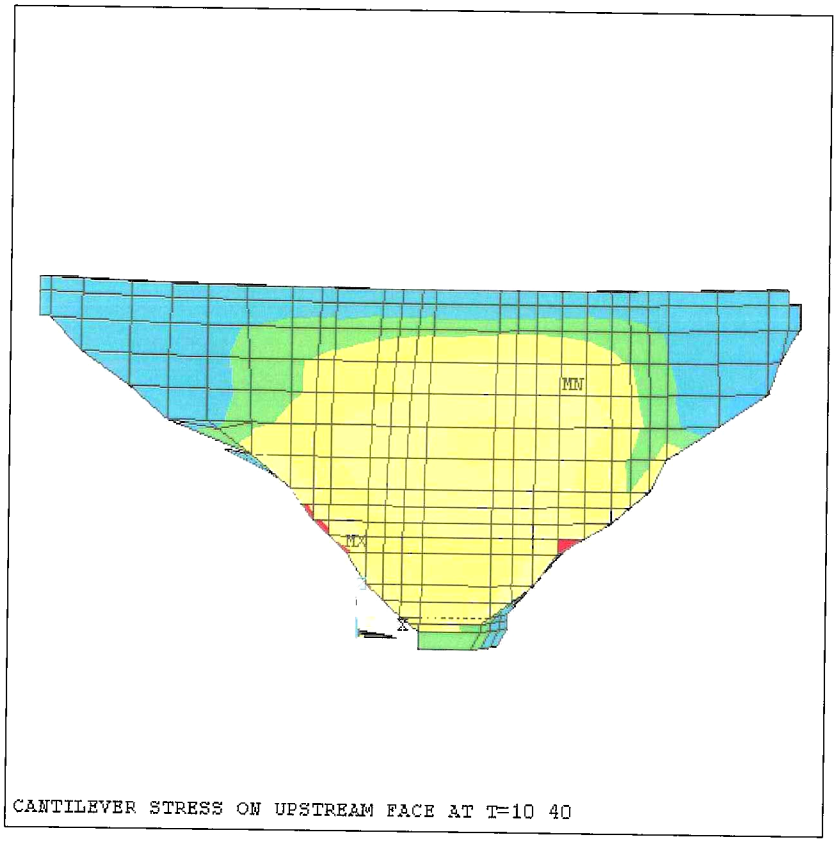
-350668
-295164
-239660
-184156
-128651
-73147
-17643
37861
93365
148870

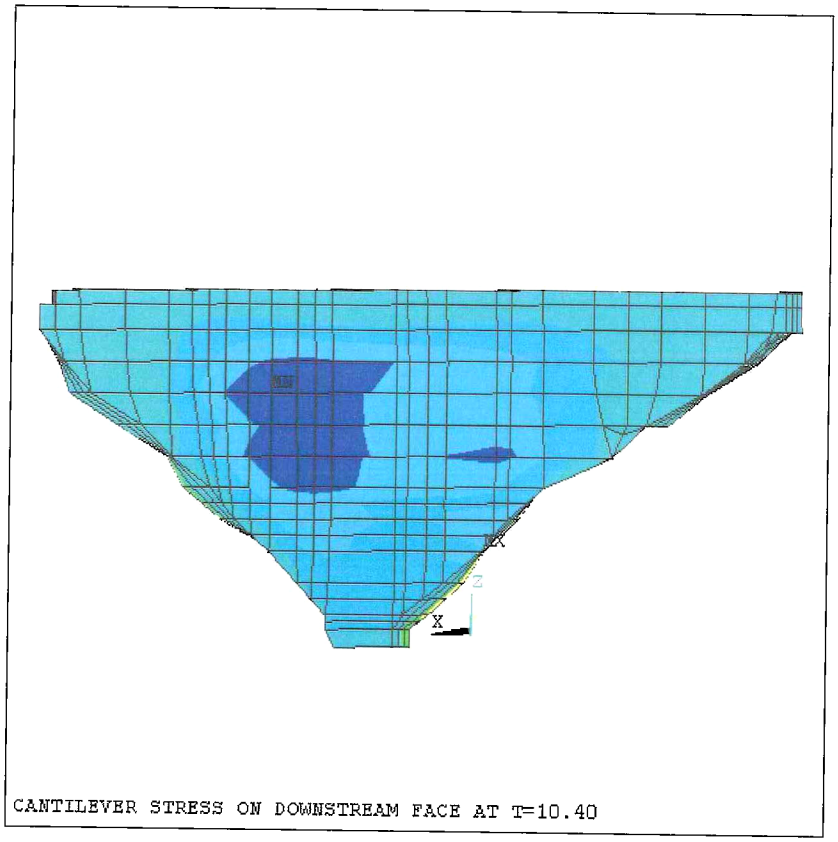
ALL VALUES IN PSF

ARCH STRESS ON DOWNSTREAM FACE AT T=10 40

HARZA

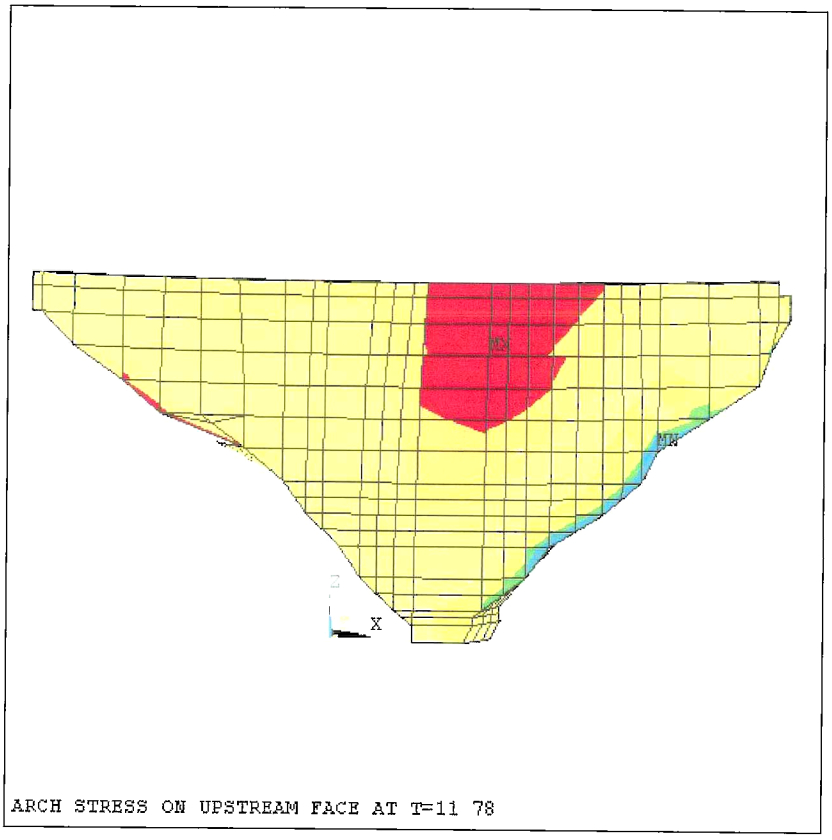
Figure 5-35





HARZA

Figure 5-36



ANSYS 5 6
FEB 10 2000
14:54:54
ELEMENT SOLUTION
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RSYS=0
PowerGraphics
EFACET=1
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SMN =-269184
SMX =123915
-269184
-225507
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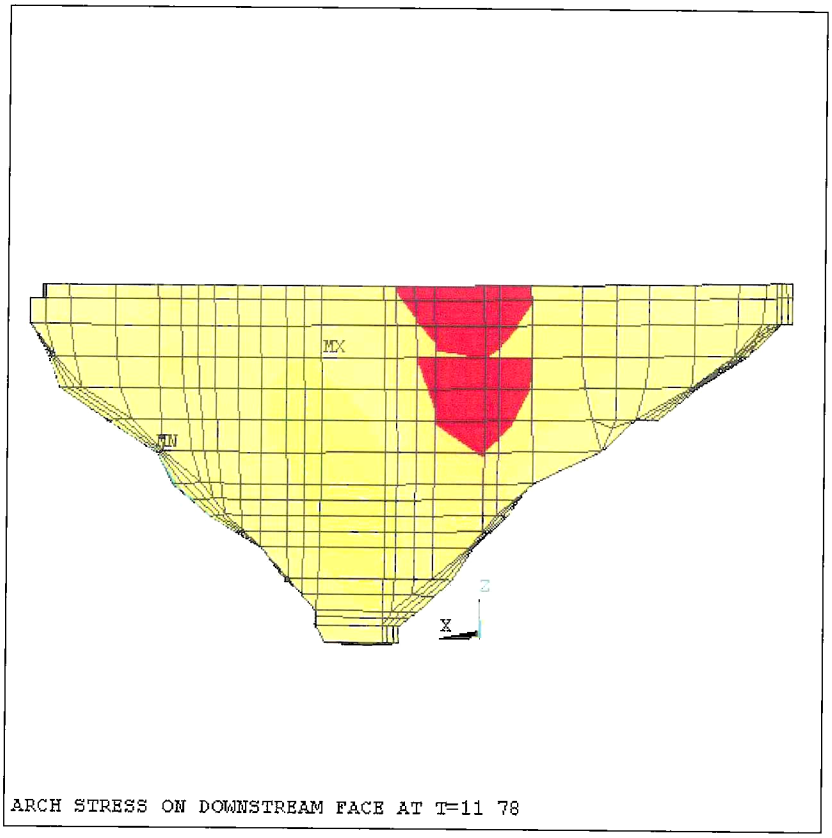
-138151
-94473
-50796
-7118
36560
80237
123915

ALL VALUES IN PSF

ARCH STRESS ON UPSTREAM FACE AT T=11 78



ANSYS 5 6
FEB 10 2000
14:50:40
ELEMENT SOLUTION
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SX (NOAVG)
RSYS=0
PowerGraphics
EFACET=1
DMX =1 508
SMN =-269184
SMX =123915
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-181829
-138151
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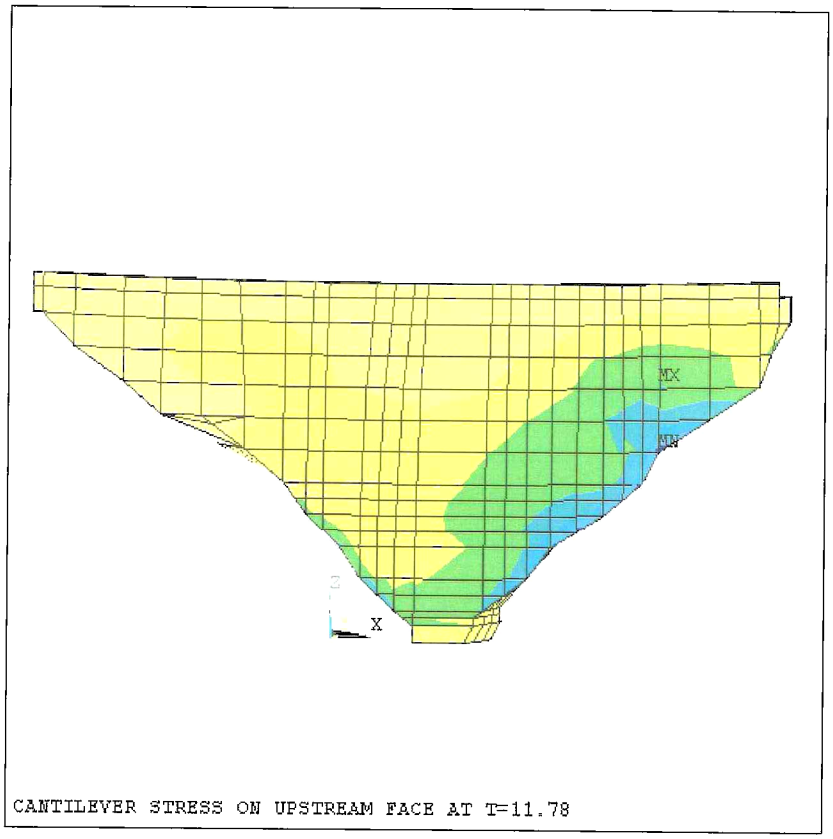
36560
80237
123915

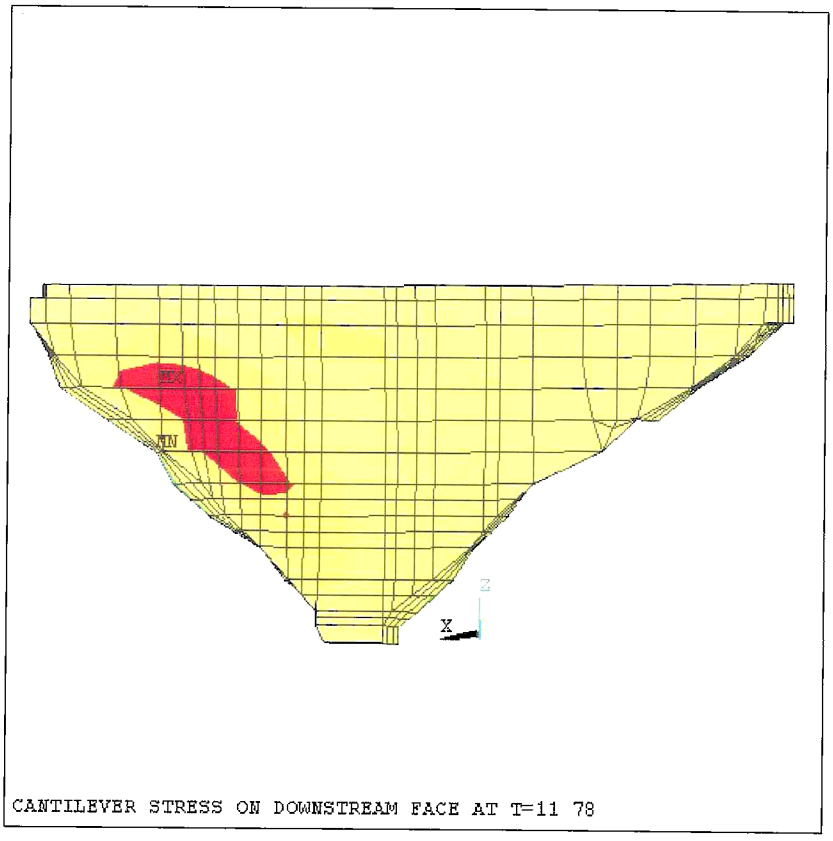
ALL VALUES IN PSF

ARCH STRESS ON DOWNSTREAM FACE AT T=11 78

HARZA

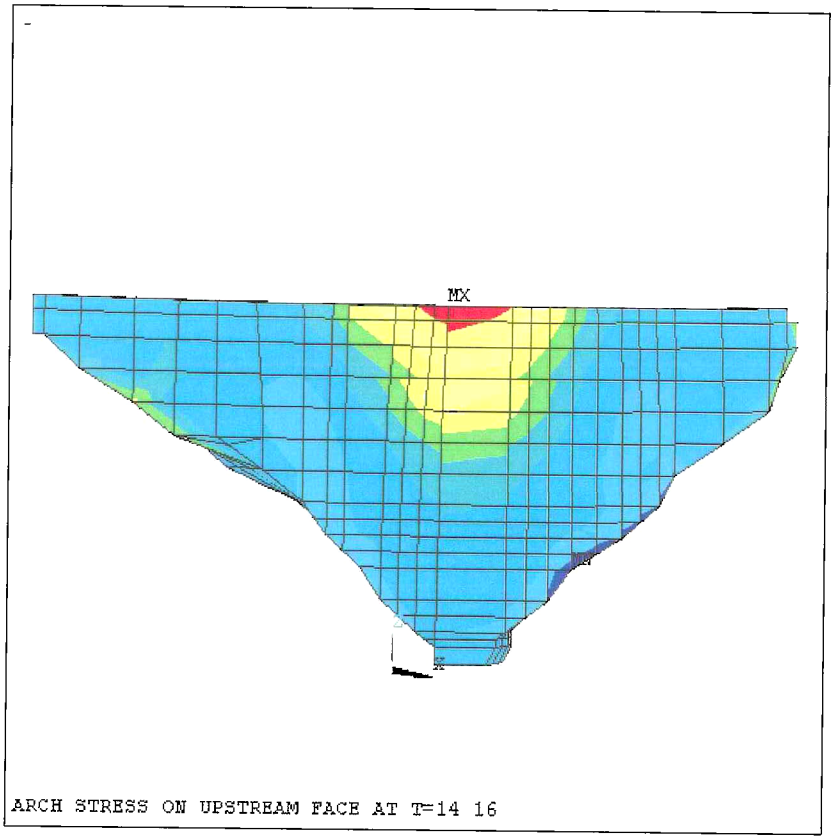
Figure 5-37





HARZA

Figure 5-38



ARCH STRESS ON UPSTREAM FACE AT T=14 16

ANSYS 5.6
FEB 10 2000
14:46:13
ELEMENT SOLUTION
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TIME=14.16
SX (NOAVG)
RSYS=0
PowerGraphics
EFACET=1
DMX =2.041
SMN =-98611
SMX =282567
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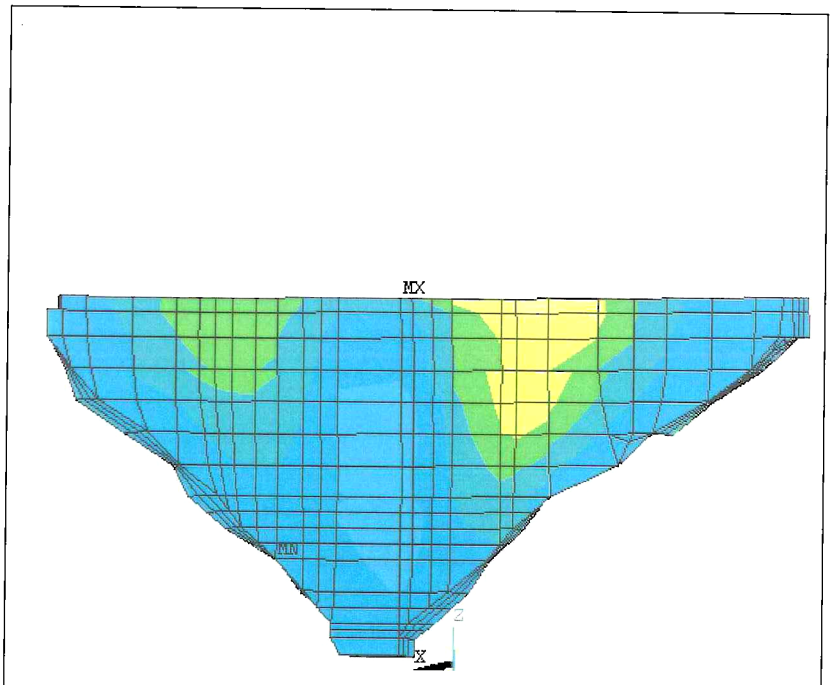
-56257
-13904
28449
70802
113155
155508
197861
240214
282567

ALL VALUES IN PSF



ARCH STRESS ON DOWNSTREAM FACE AT T=14 16

ANSYS 5 6
FEB 10 2000
14:49:28
ELEMENT SOLUTION
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SUB =458
TIME=14 16
SX (NOAVG)
RSYS=0
PowerGraphics
EFACET=1
DMX =2.041
SMN =-98611
SMX =282567
-98611
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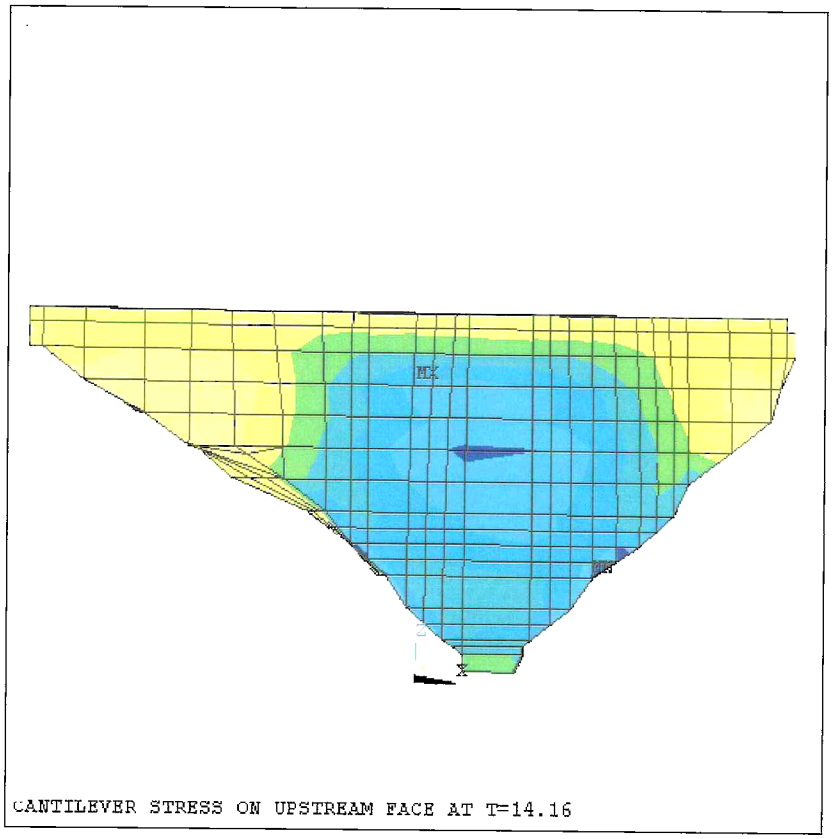
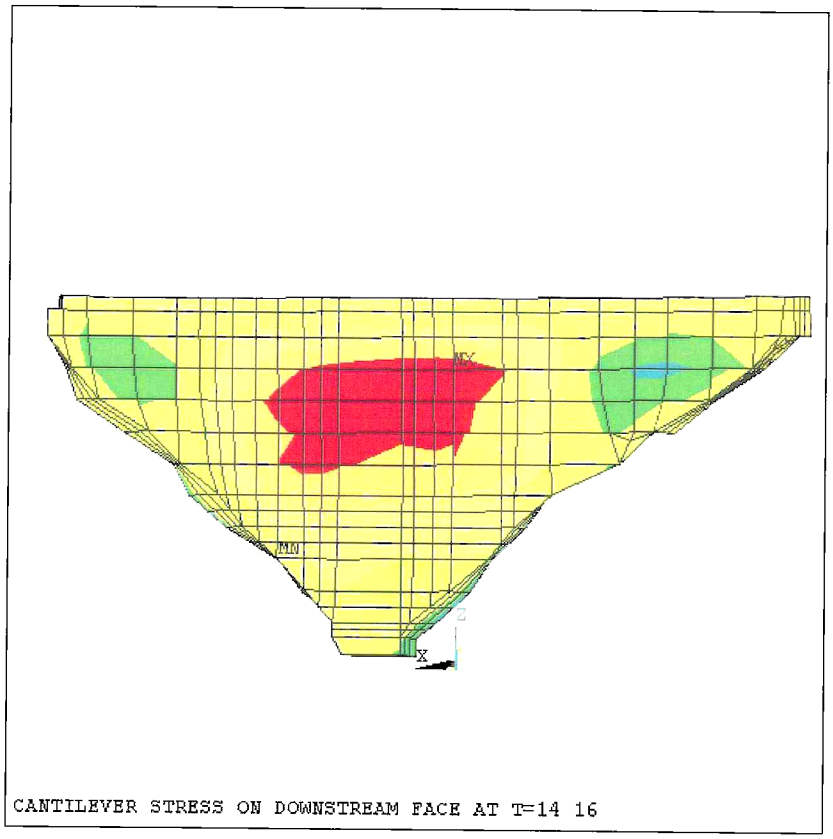
-13904
28449
70802
113155
155508
197861
240214
282567

ALL VALUES IN PSF

HARZA

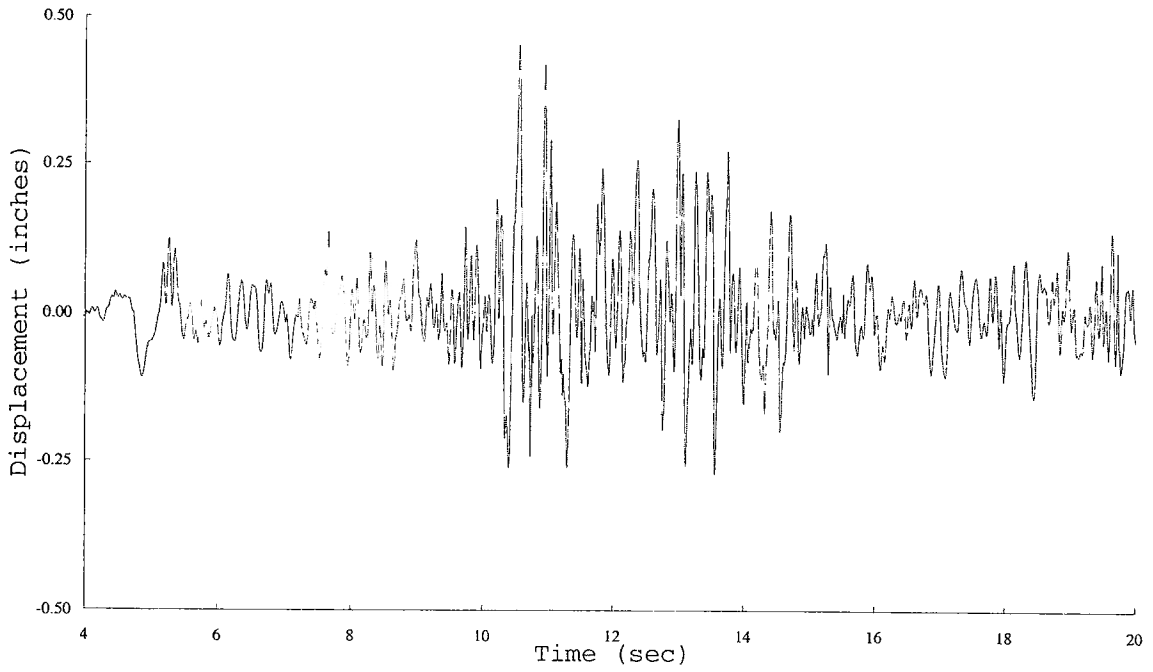
Figure 5-39



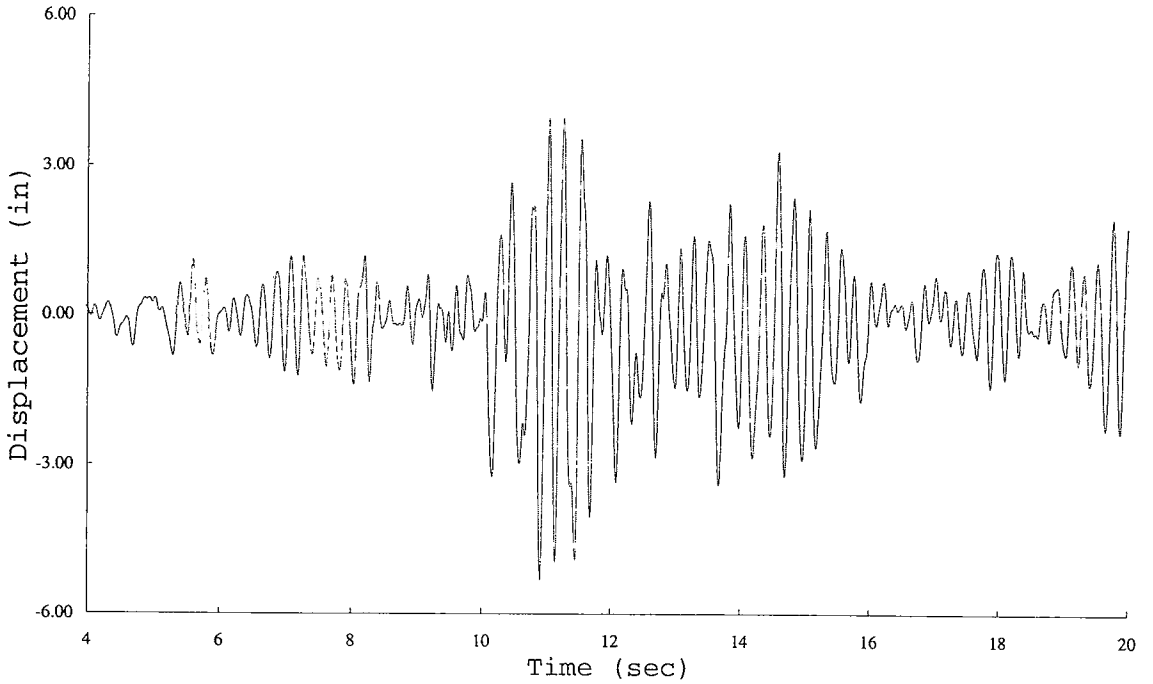


HARZA

Figure 5-40



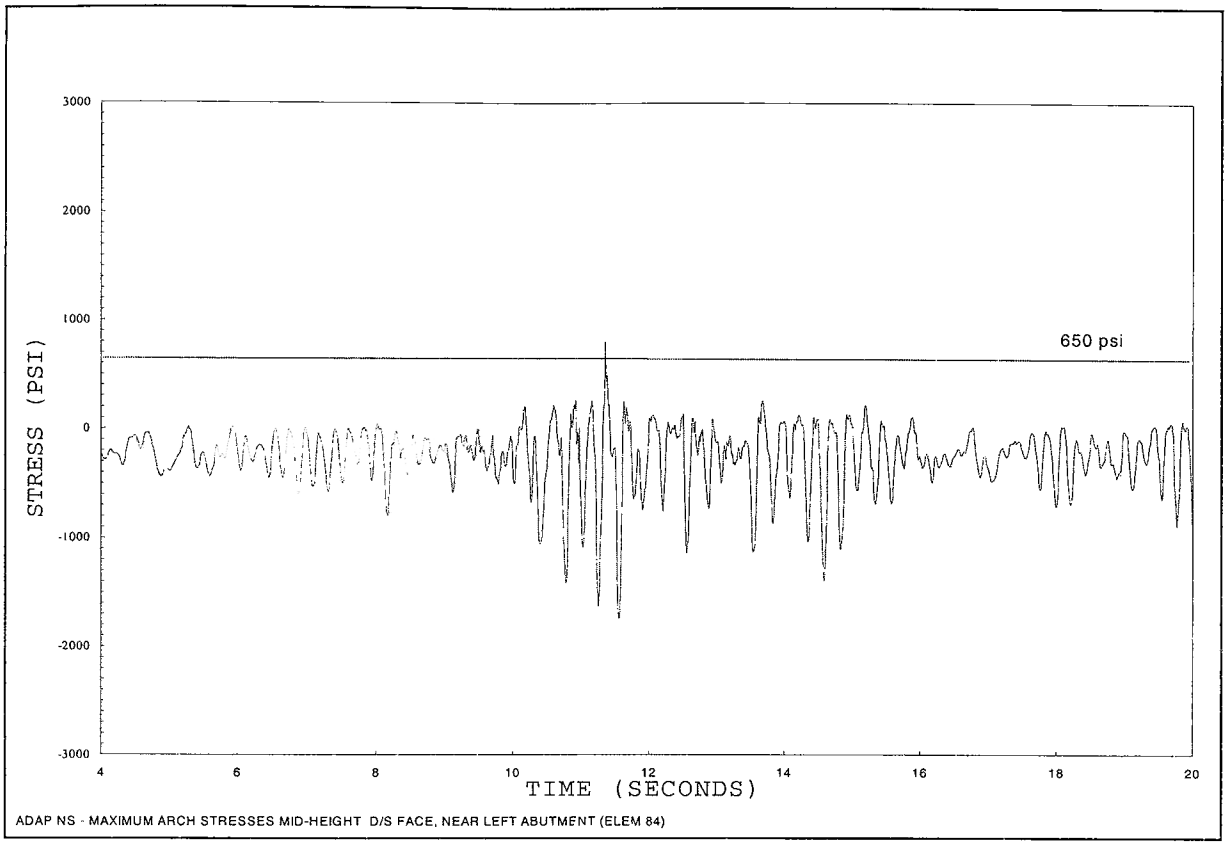
ADAP-NS - RELATIVE DISPLACEMENT AT TOP OF DAM (CROSS-CANYON)



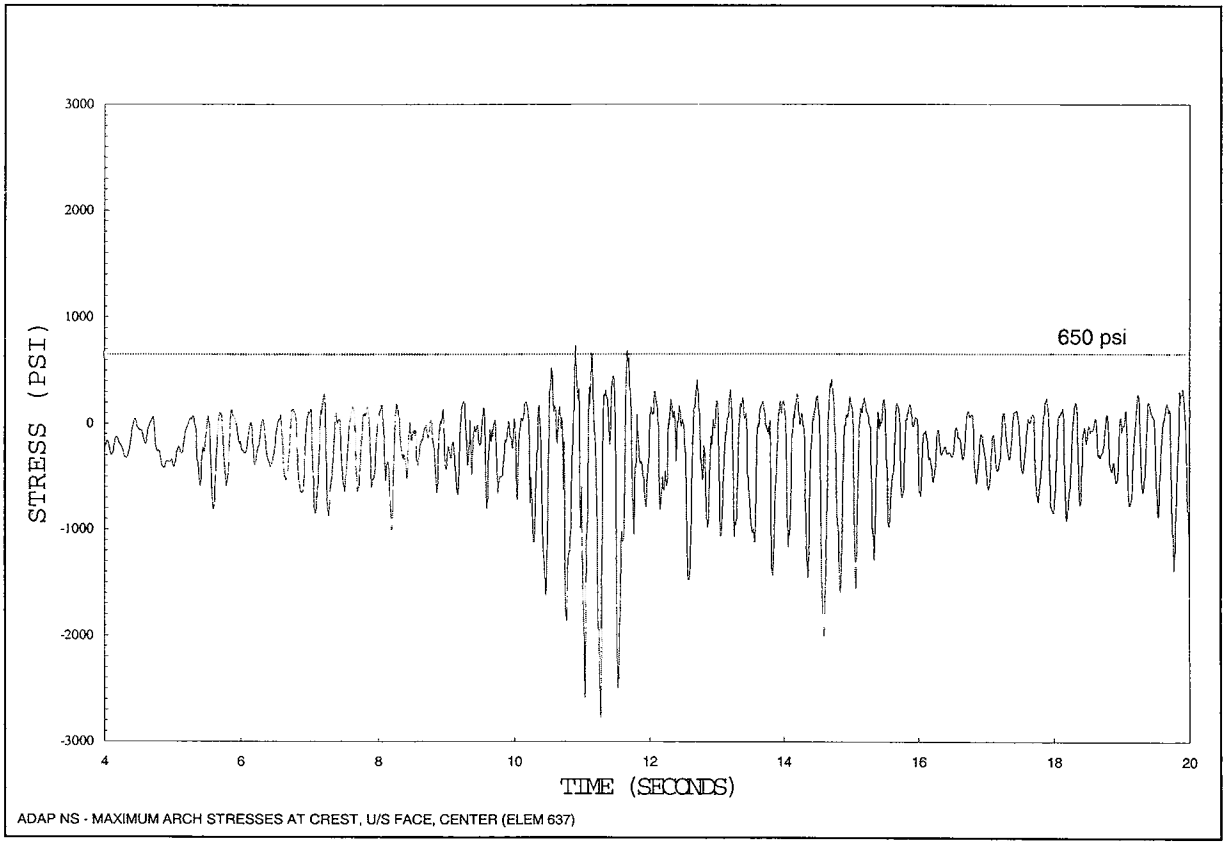
ADAP-NS - RELATIVE DISPLACEMENT AT TOP OF DAM (CANYON PARALLEL)

HARZA

Figure 5-41



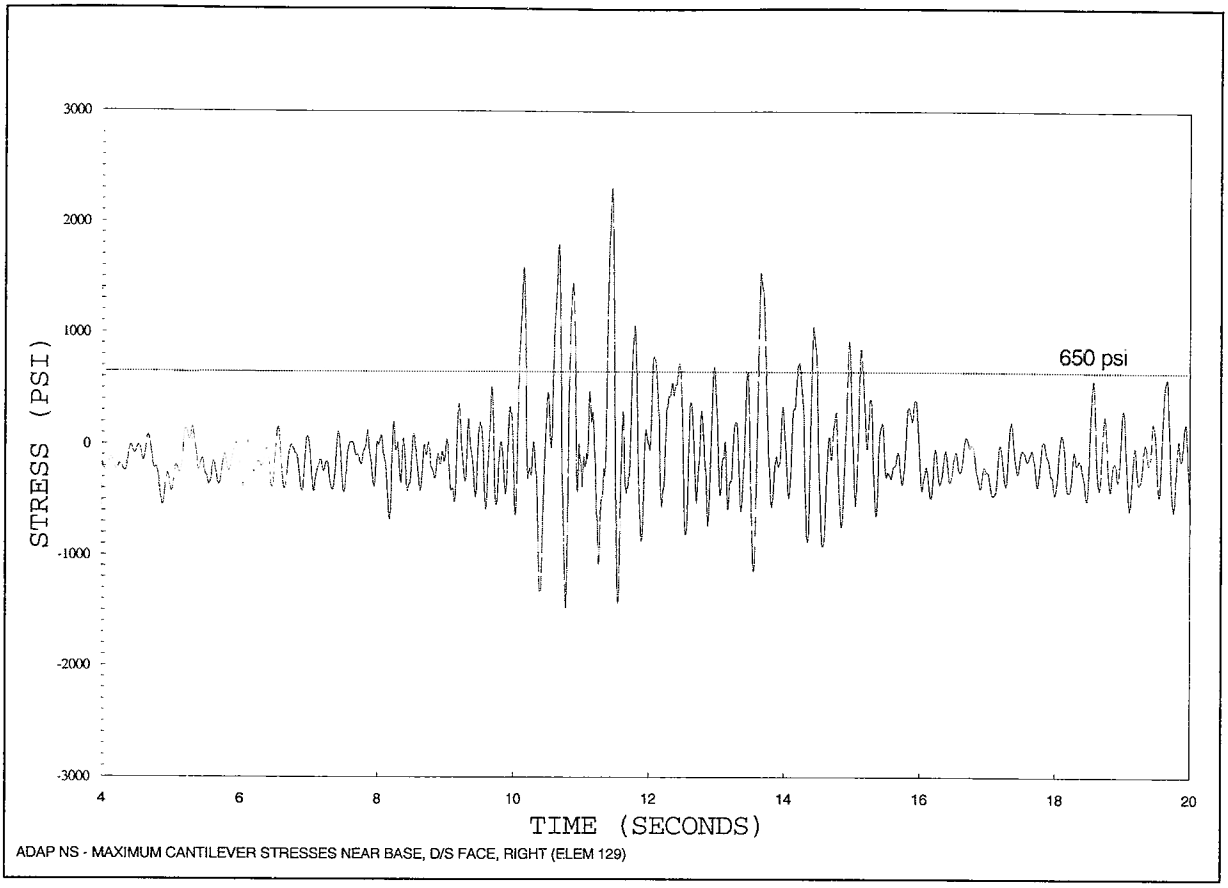
ADAP NS - MAXIMUM ARCH STRESSES MID-HEIGHT D/S FACE, NEAR LEFT ABUTMENT (ELEM 84)



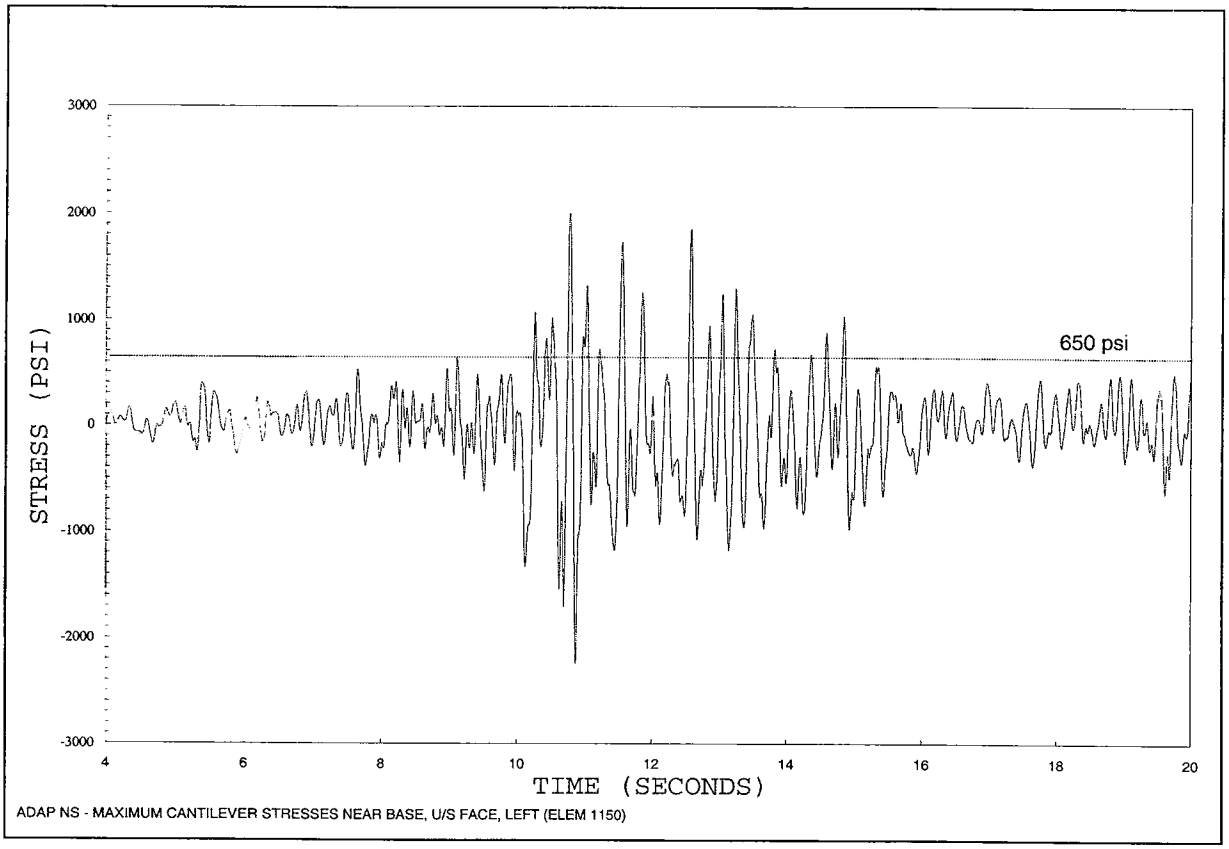
ADAP NS - MAXIMUM ARCH STRESSES AT CREST, U/S FACE, CENTER (ELEM 637)

HARZA

Figure 5-42



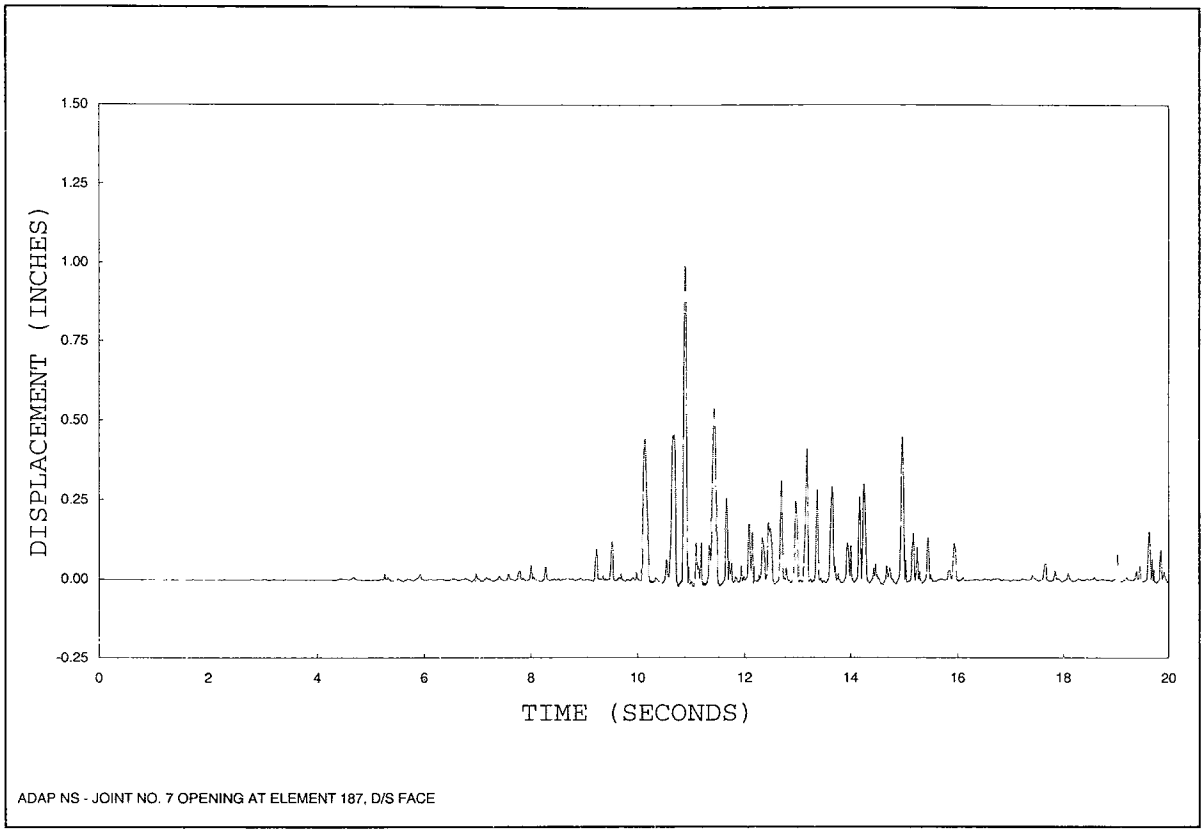
ADAP NS - MAXIMUM CANTILEVER STRESSES NEAR BASE, D/S FACE, RIGHT (ELEM 129)



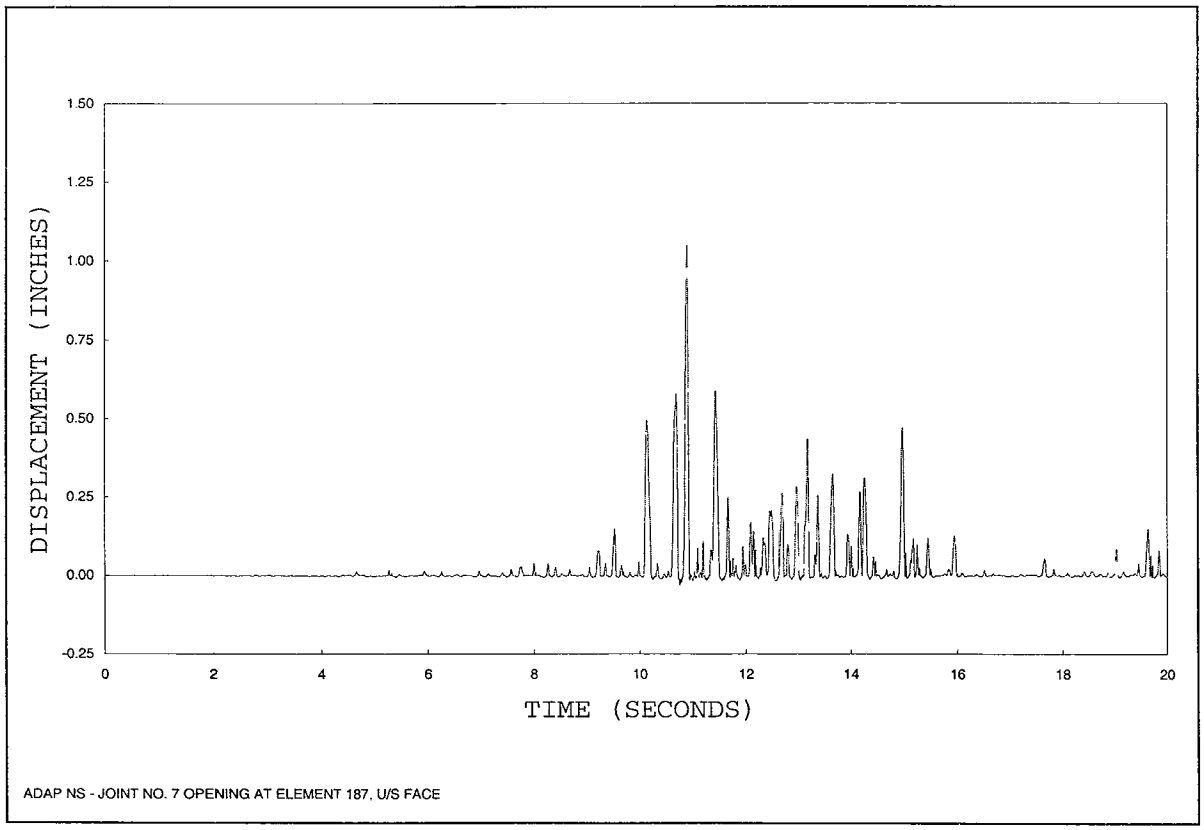
ADAP NS - MAXIMUM CANTILEVER STRESSES NEAR BASE, U/S FACE, LEFT (ELEM 1150)

HARZA

Figure 5-43



ADAP NS - JOINT NO. 7 OPENING AT ELEMENT 187, D/S FACE

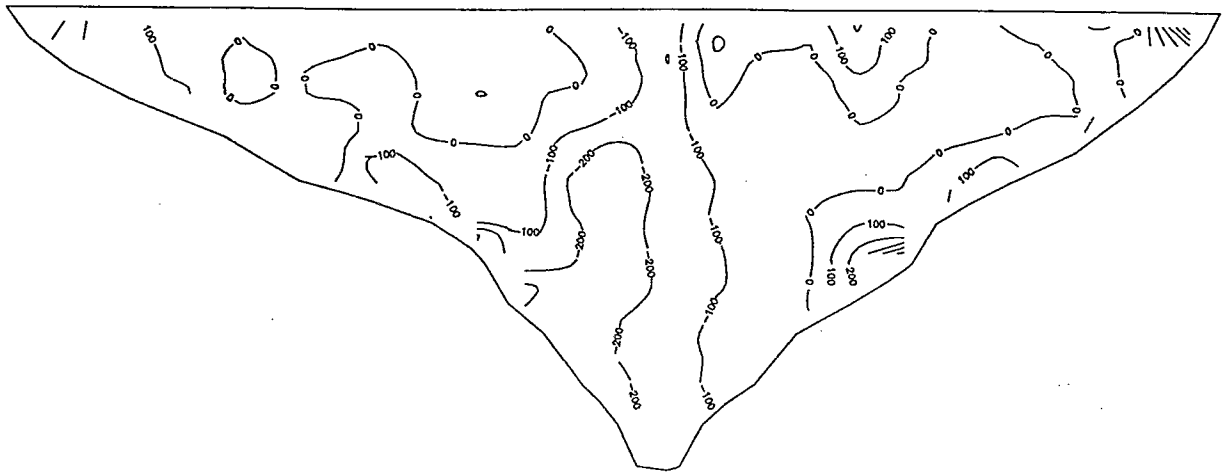


ADAP NS - JOINT NO. 7 OPENING AT ELEMENT 187, U/S FACE

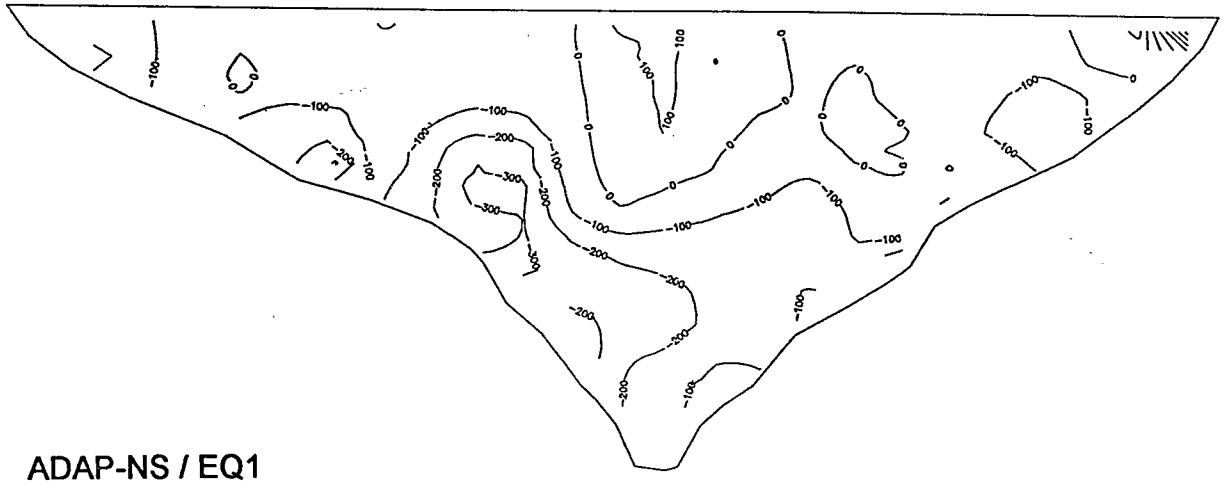
HARZA

Figure 5-44

US Arch Stress, Time=10.87



DS Arch Stress, Time=10.87

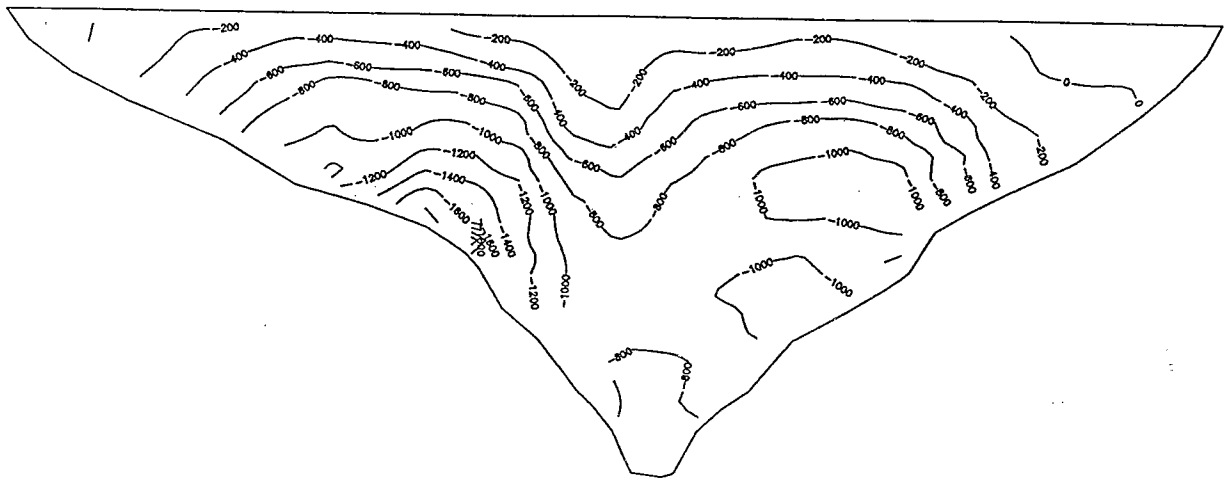


ADAP-NS / EQ1
Dynamic Analysis

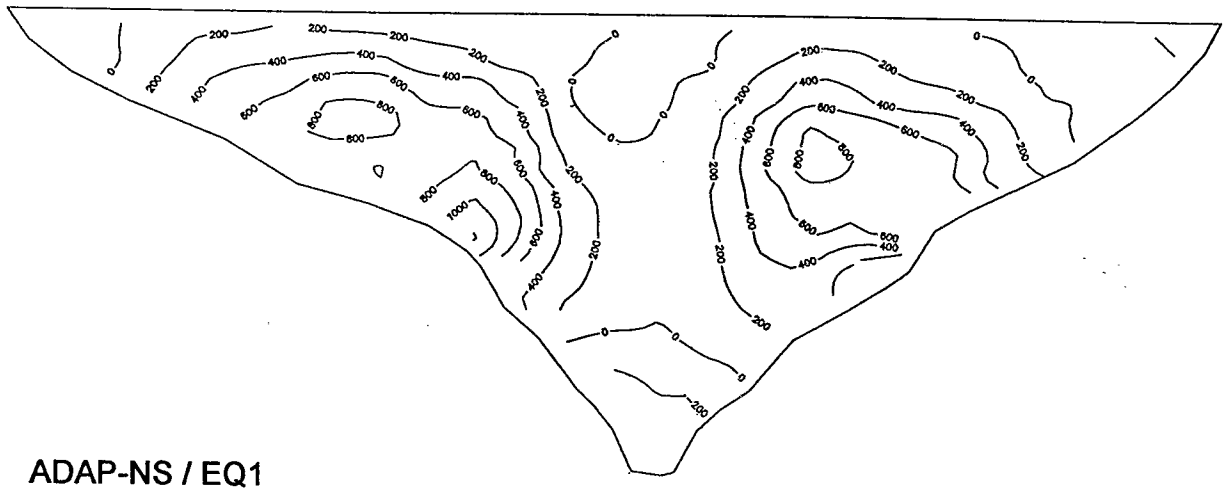
HARZA

Figure 5-45

US Cantilever Stress, Time=10.87



DS Cantilever Stress, Time=10.87

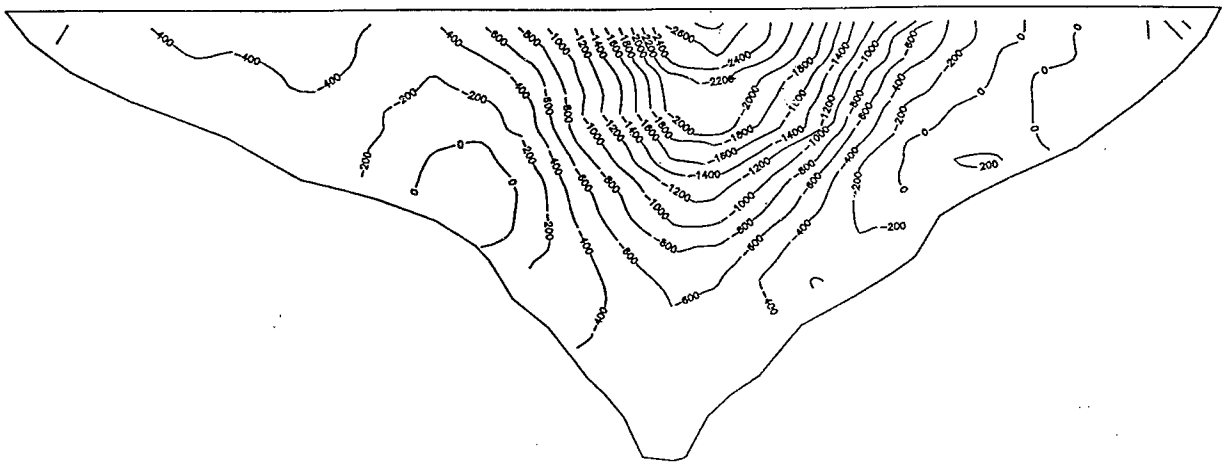


ADAP-NS / EQ1
Dynamic Analysis

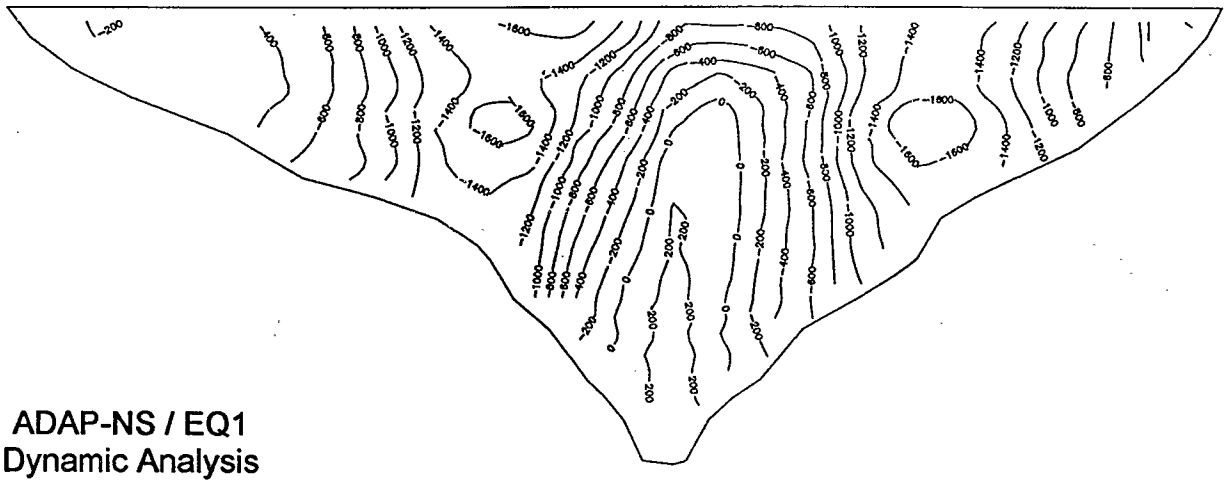
HARZA

Figure 5-46

US Arch Stress, Time=11.28



DS Arch Stress, Time=11.28

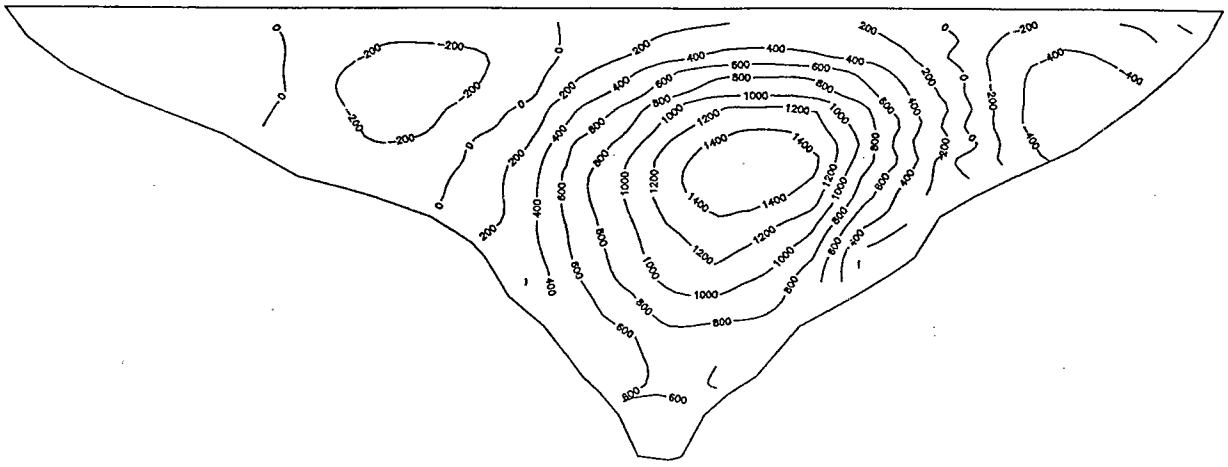


ADAP-NS / EQ1
Dynamic Analysis

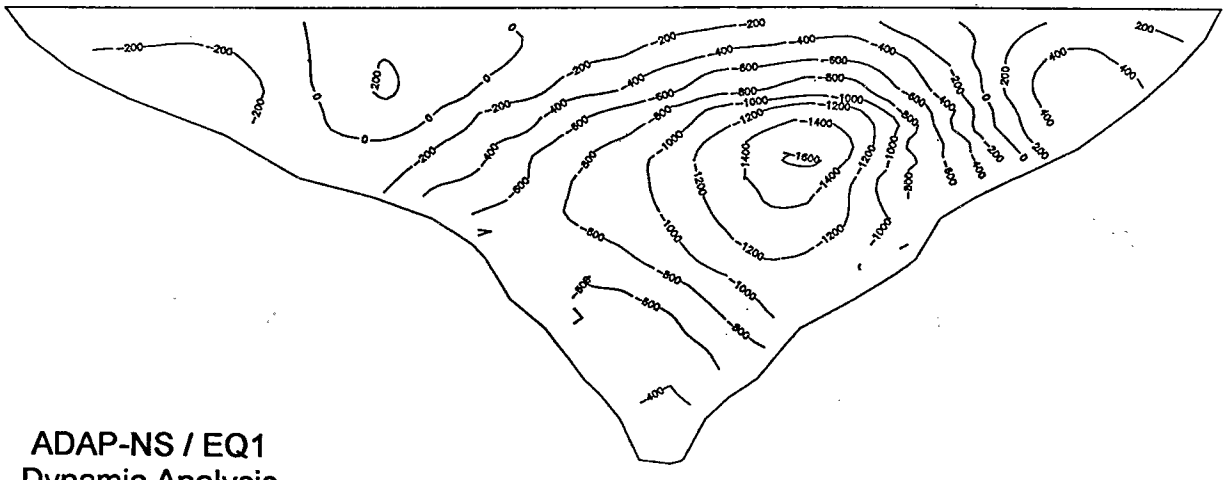
HARZA

Figure 5-47

US Cantilever Stress, Time=11.28



DS Cantilever Stress, Time=11.28

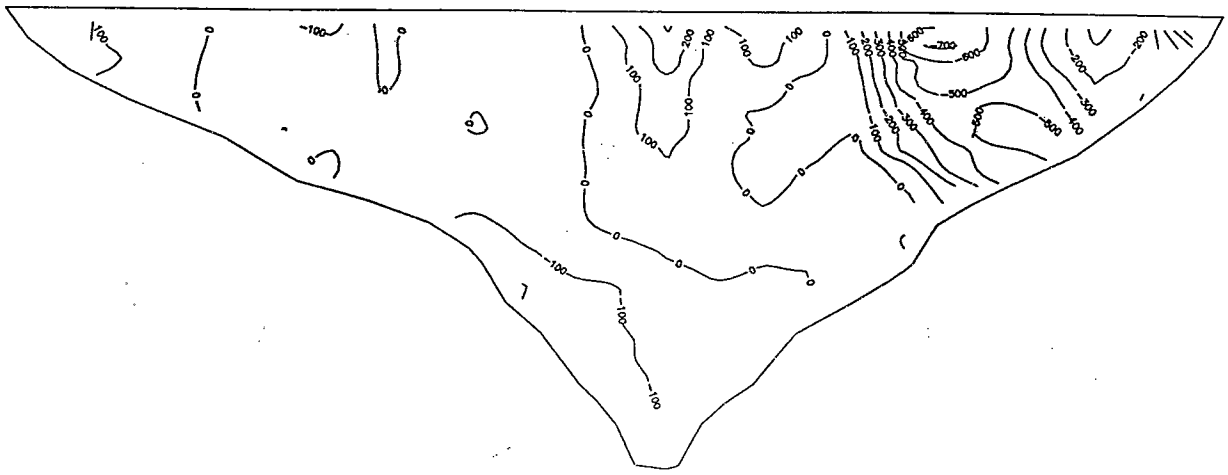


ADAP-NS / EQ1
Dynamic Analysis

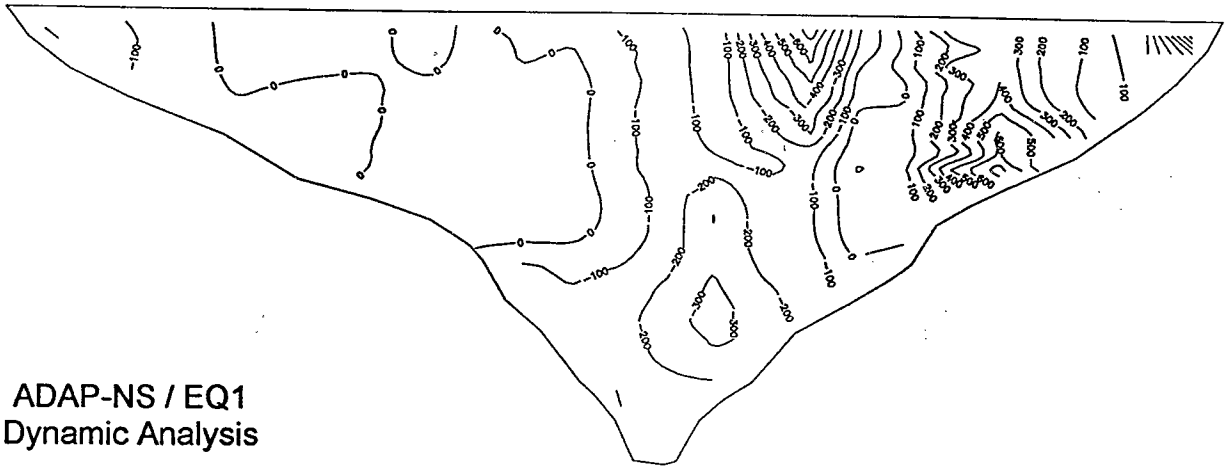
HARZA

Figure 5-48

US Arch Stress, Time=11.36



DS Arch Stress, Time=11.36

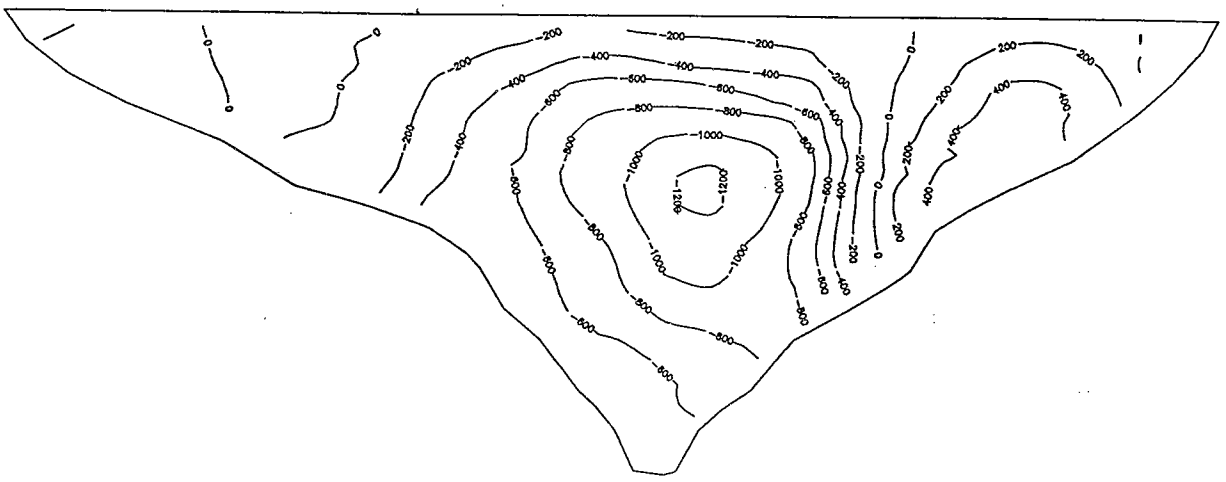


ADAP-NS / EQ1
Dynamic Analysis

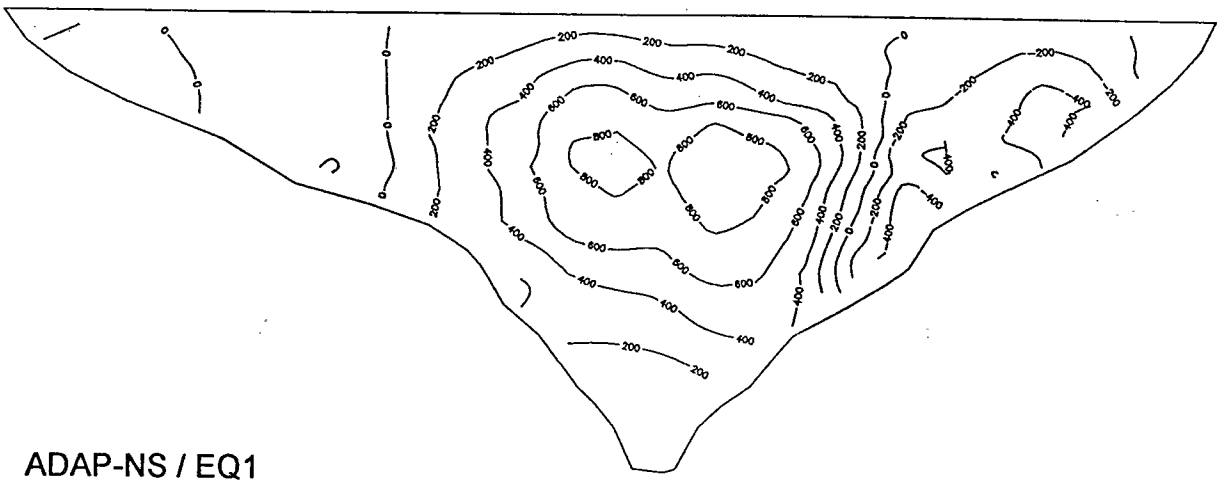
HARZA

Figure 5-49

US Cantilever Stress, Time=11.36



DS Cantilever Stress, Time=11.36

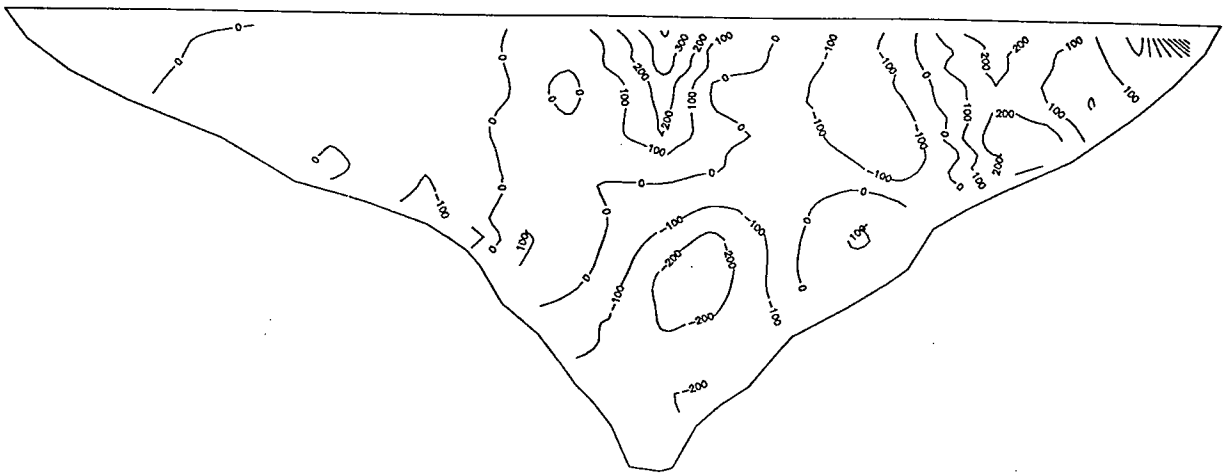


ADAP-NS / EQ1
Dynamic Analysis

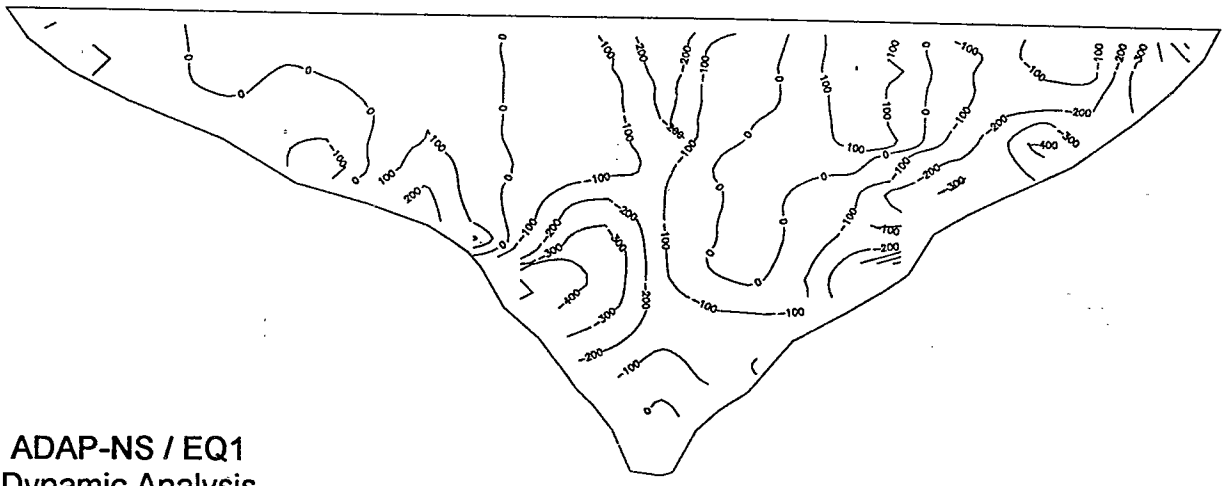
HARZA

Figure 5-50

US Arch Stress, Time=11.46



DS Arch Stress, Time=11.46

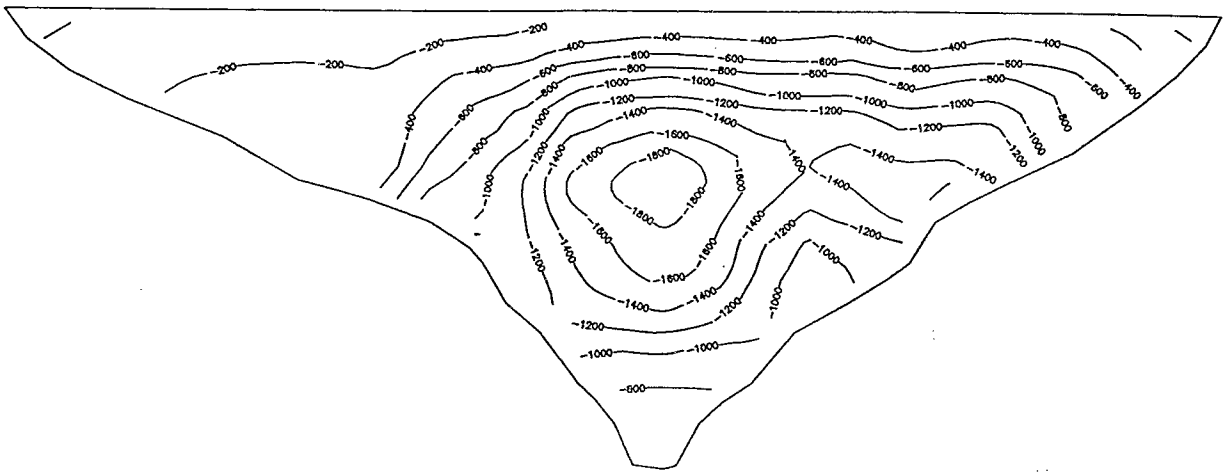


ADAP-NS / EQ1
Dynamic Analysis

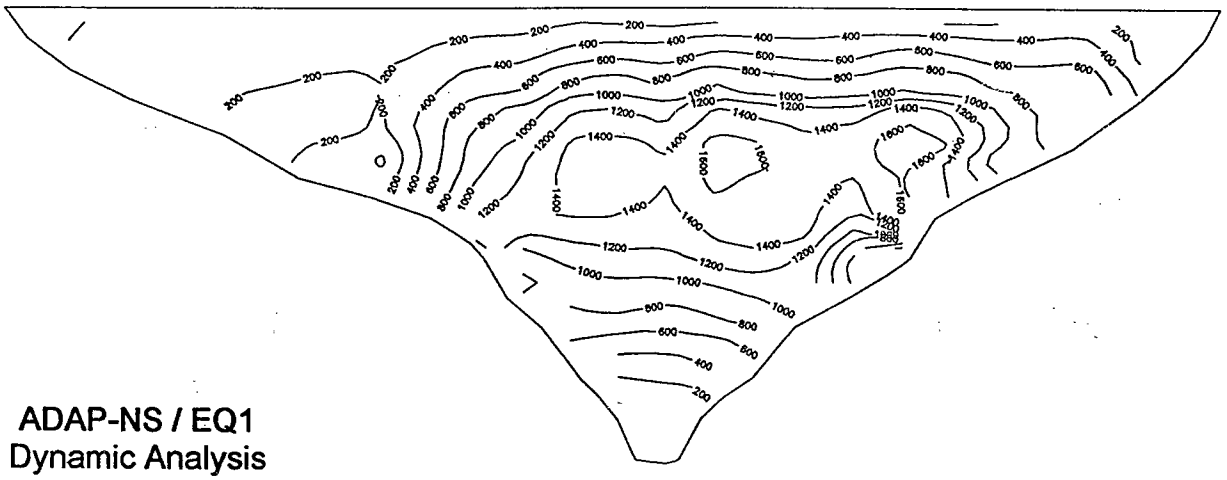
HARZA

Figure 5-51

US Cantilever Stress, Time=11.46



DS Cantilever Stress, Time=11 46



ADAP-NS / EQ1
Dynamic Analysis

HARZA

Figure 5-52

SECTION SIX

6.0 HYDROLOGIC / HYDRAULIC REANALYSIS

The purpose of this task is to address the capacity limitations, if any, of the existing spillway/outlet works and to reanalyze and reevaluate previous hydraulic studies. The reanalyses include identifying and performing necessary evaluations of previous hydraulic and hydrologic studies.

Based on the large size and high hazard classification for Santa Anita Dam, the Probable Maximum Flood (PMF) runoff resulting from the Probable Maximum Precipitation (PMP) has been recommended by the California Division of Safety of Dams to be the same as the spillway design flood (DSOD, 1980). It is County policy that all dams that meet the State of California's definition of a dam must be constructed to safely pass the PMF (County, 1991). A summary of the existing PMF studies is presented below. The County's Capital Flood is also presented because it is an objective that spillway modifications will reduce, or at least not increase, the Capital Flood outflow.

The existing outlet capabilities are reviewed and summarized herein. Harza has prepared an independent estimate of the outlet capabilities with a comparison to previous estimates. Using the estimated outlet capabilities, the PMF and the Capital Flood are routed through the reservoir and maximum water levels are determined. The reservoir routing results as presented in HEC-1 output are also included in Appendices G and H.

6.1 Description of Existing Outlet Facilities

Santa Anita Dam, constructed in 1924-27, is a 225-feet high variable radius arch dam and had an original reservoir capacity of 932 acre-feet. The dam has three designated spillways, plus outlet works and a dam crest that can be overtopped, all of which are described in the following paragraphs.

6.1.1 Main Spillway

The main spillway is a covered channel of varying section and slope located at the west (right) abutment. The main spillway is approximately 170-feet long, having no slope at its inlet and a slope of 6% at its outlet, with a steep intermediate slope between the two sections. The inlet, with a crest at El 1316, has a rounded entrance lip and sidewalls, and is approximately 27-feet wide by 12-feet high. The channel narrows to about 20-feet wide at a point where it enters a 10-foot high covered channel. The covered channel slopes steeply downward and narrows until it becomes a 10-foot wide by 10-foot high box culvert. Construction of spillway modifications was completed in 1947, which included removing bridge piers, rounding the edges of the reservoir inlet, and constructing a new single span spillway slab topped with a shelter house.

6.1.2 Auxiliary Spillway

The auxiliary spillway inlet, located near the center of the dam, is a semi-hemispherical concrete structure on the upstream dam face that discharges into a circular conduit of decreasing diameter passing through the dam. The semi-circular crest at El 1316 has a radius of 15-feet. The inlet is topped by a concrete and steel trashrack.

6.1.3 Crest Spillway

The crest spillway is located at El 1325 near the center of the dam, above the auxiliary spillway and outlet works. The crest spillway is formed by a 90-foot long gap in the parapet wall on the crest of the dam. The spillway has an extension platform projecting outward approximately 17 feet from the vertical downstream face of the dam, which forces overflow away from the dam and protects the outlet works and operating personnel below.

6.1.4 Outlet Works

The outlet works consist of several valves located on platforms on the downstream face of the dam, circular conduits through the dam, and three separate inlet risers on the upstream face of the dam. The inlet risers raise the level of the inlets to reduce the chance of plugging of the intakes by sedimentation that occurs during flood inflows. The outlet works components are summarized as follows:

- A 30-inch needle valve at El 1198 with upstream riser to El 1234 plus an 8-foot high trashrack to El 1242.
- A 30-inch needle valve at El 1177 with upstream riser to El 1232 plus a 20-foot high trashrack to El 1252.
- A 20-inch gate valve, with upstream riser to El 1195, and a 20-inch conduit through the dam leading to an 18-inch hollow cone valve at El 1161.
- A 72-inch square slide gate sluiceway, with inlet centerline at El 1183, which leads to a 5-foot diameter conduit through the dam and outlet with a centerline at El 1179.5. Construction of the sluiceway was completed in 1944. The sluiceway is intended for sediment and debris removal.
- An 18-inch gate valve at El 1112.75 that is inoperable due to sedimentation.

The combined capacity of the three flood operation valves has previously been estimated to range from 550 cfs to 633 cfs with a water surface at El 1316. Deterioration or debris plugging would result in reduced outflow capacity. The valves are assumed to be inoperative during the PMF and Capital Flood because of potential plugging with debris and sedimentation, and downstream hydraulic conditions. Even if the valves could be operated during extreme floods such as the PMF and the Capital Flood, they would have a negligible effect on the peak water level.

In 1968-69, a 9-foot diameter tunnel was constructed in the east (left) abutment for the purposes of removing reservoir sedimentation. The tunnel was sealed in 1971.

6.1.5 Dam Crest

Reservoir outflows exceeding the spillway and outlet valve capacity would flow over the dam. The dam crest is 10-feet wide at the crest level at El 1325. There is a 3-foot high parapet wall on the upstream side of the dam with a top at El 1328. The crest length of the dam is 612 feet.

6.2 Existing Outlet Capability

The outlet capability is one of the primary factors that determines the maximum water level in the reservoir behind Santa Anita Dam during a flood. If the reservoir volume is small in relation to the inflow flood volume, the outlet capability directly determines the maximum reservoir water level.

The outlet rating curve is the sum of outlet capabilities of each component with outflow capability during the design flood. For Santa Anita Dam, the PMF and Capital Flood outlet capability includes the main spillway, auxiliary spillway, crest spillway, and dam crest (over the parapet wall). An outlet rating curve has previously appeared in the Phase I Safety Inspection Report (DSOD, 1980). However, Harza has independently derived an outlet rating curve as described in the following paragraphs.

6.2.1 Main Spillway

The main spillway is a complicated hydraulic structure of varying cross-section and slope that can include subcritical and supercritical flows, open channel and pressurized flows, and hydraulic controls at different sections for different flows. Physical hydraulic model studies of the original main spillway (with piers and poor inlet hydraulics) and a modified (with unconstructed siphon) main spillway are available (LACFCD, 1947), but neither closely depict the spillway in its current condition. The Phase I Inspection Report (DSOD, 1980, Table 5-4) included a rating curve that differed significantly from the hydraulic model studies. In an attempt to model in detail the main spillway as it currently exists, Harza used the HEC-RAS model (USACOE, 1998) to develop an updated rating curve.

Results of the original main spillway hydraulic model, the HEC-RAS model, and the DSOD main spillway rating curve are presented on Figure 6-1. Because the HEC-RAS model should not show lower flows than the physical model study, it had to be rejected. The final rating curve assumed that at reservoir levels between El 1316 and about El 1328, flow in the main spillway will be controlled as a broad-crested weir at the reservoir inlet. For water levels at El 1329 or greater, the equation for an orifice under low head applies. Above El 1331, it appears that the flow is controlled by inlet to the 10-foot by 10-foot box culvert, and the final rating curve takes the shape of the model study rating curve.

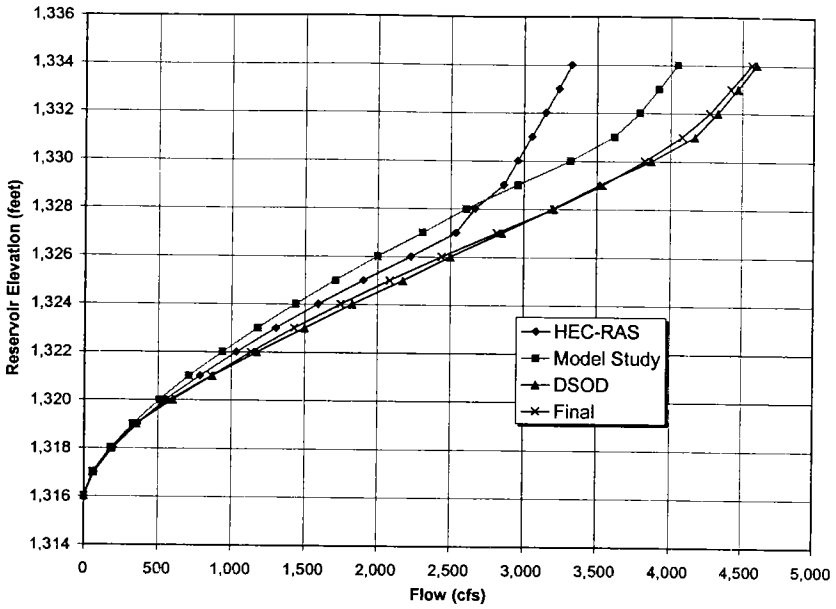


Figure 6-1: Comparison of Rating Curves for the Main Spillway

6.2.2 Auxiliary Spillway

The auxiliary spillway can potentially have hydraulic controls at three locations, (1) at the crest of the weir, (2) at the 7-foot diameter entrance to the conduit through the dam, or (3) at the 5-foot diameter exit of the conduit through the dam. The weir crest length was estimated to be reduced by about 15% due to trashrack bars and piers. The conduit through the dam slopes downward at an angle of 23.4 degrees. An analysis of the three potential hydraulic controls indicated that the weir crest controls until the depth on the crest is greater than 3 feet (water surface level greater than El 1319), at which point the 5-foot diameter exit orifice begins to control. The 7-foot diameter conduit entrance does not act as a hydraulic control. The 5-foot diameter exit opening is assumed to have an orifice coefficient of 0.95.

6.2.3 Crest Spillway

The 90-foot wide crest spillway, with a breadth of 27-feet (including the dam crest and extension), functions as a broad-crested weir. The concrete arch dam portion of the crest spillway is assumed to be horizontal. Drawings indicate that the crest extension slopes downward at an angle of about 3.3 degrees, which is a slope of about 6%. However, the dam crest is wide enough that the crest spillway will function as a broad-crested weir throughout the potential range of flow. The coefficient for a broad-crested weir is 2.65. This assumes that the upstream edge of the crest spillway has not been rounded, an improvement that has been suggested in the hydraulic model study reports. The crest spillway results presented in Table 6-1 below agree well with results from the model study (LACFCD, 1947, Fig. 1), but are significantly lower than the results presented in the Phase I Safety Inspection Report (DSOD, 1980, Table 5-4). The DSOD results imply a spillway coefficient as high as 3.67, which is more appropriate for a nearly ogee-shaped crest.

6.2.4 Dam Crest

The dam crest is 10-feet wide, which would cause flow over the dam to function as a broad-crested weir. However, the parapet wall (El 1328) on the upstream face of the dam, which functions as a sharp-crested weir, controls flow over the dam. A sharp-crested weir coefficient of 3.3 has been used to estimate flow over the dam crest. Based on the as-constructed drawing of Santa Anita Dam (No. 55-D38.1, Jan. 1948), a crest arc length of 562 feet was measured between the left abutment and the raised area on the right abutment that is generally blocked by the generator house on top of the main spillway. Subtracting 90 feet for the crest spillway leaves an effective overflow length for the parapet wall of 472 feet. Flow included in the flow over parapet wall column in Table 6-1 does not include crest spillway flow.

A summary of the existing outlet capability of Santa Anita Dam is presented in Table 6-1. The outflow capacities in cubic feet per second (cfs) are identified for various reservoir water surface levels in the following table.

Table 6-1: Santa Anita Dam Existing Outflow Rating Table

Water Surface Elevation (feet)	Main Spillway (cfs)	Auxiliary Spillway (cfs)	Crest Spillway (cfs)	Flow over parapet wall (cfs)	Total Outflow (cfs)
1,316	0	0	0	0	0
1,317	66	107	0	0	173
1,318	187	323	0	0	510
1,319	351	666	0	0	1,017
1,320	580	799	0	0	1,379
1,321	863	813	0	0	1,676
1,322	1,134	826	0	0	1,961
1,323	1,429	840	0	0	2,269
1,324	1,746	853	0	0	2,599
1,325	2,084	866	0	0	2,950
1,326	2,440	879	239	0	3,558
1,327	2,816	892	675	0	4,382
1,328	3,208	904	1,239	0	5,352
1,329	3,540	916	1,908	1,558	7,922
1,330	3,824	929	2,667	4,406	11,825
1,331	4,082	941	3,505	8,094	16,622
1,332	4,274	952	4,417	12,461	22,104
1,333	4,421	964	5,397	17,414	28,196
1,334	4,567	976	6,440	22,892	34,874

The total outflow rating curve from Table 6-1 as used in the flood routing is graphically presented on Figure 6-2. The Harza rating curve is very similar to the rating curve presented in the Phase I Inspection Report (DSOD, 1980), the primary difference being in the rating curve for the crest spillway.

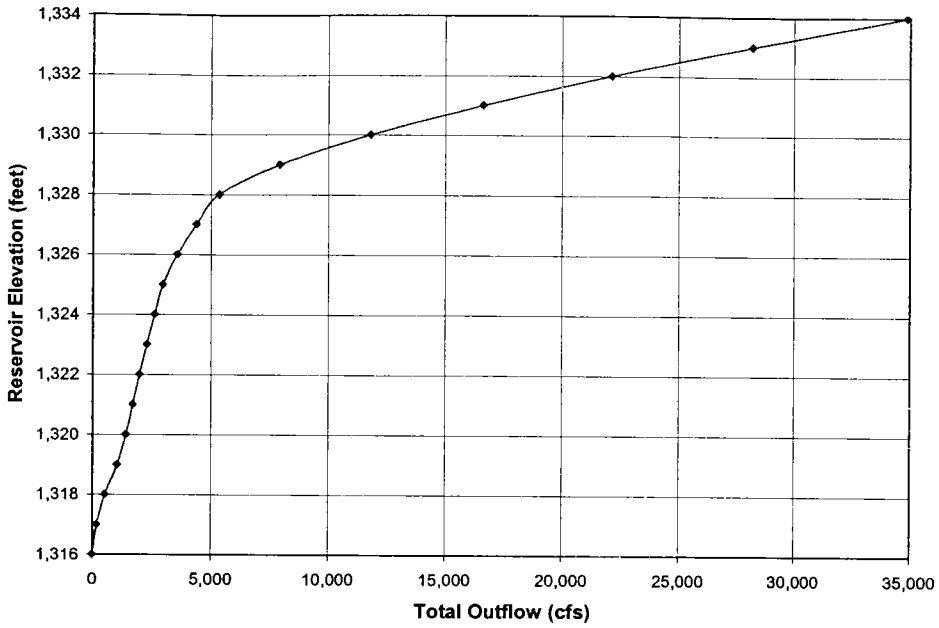


Figure 6-2: Total Outflow Rating Curve for Existing Santa Anita Dam

6.3 Inflow Floods

6.3.1 Probable Maximum Flood

The County has been developing PMFs for Santa Anita Dam since at least 1976. Two previous estimates of the PMF were based on Probable Maximum Precipitation (PMP) estimates developed from Hydrometeorological Report (HMR) No. 36 (U.S. Weather Bureau, 1961). The peak PMF inflow developed by Los Angeles County was 12,800 cfs (LACFCD, April 14, 1976), while the peak PMF inflows developed by the DSOD were 14,892 cfs for the 72-hour general storm and 24,927 cfs for the 6-hour local storm (DSOD, 1980). However, HMR 36 has now been superseded by HMR 58 (National Weather Service, 1998), and HMR 59 (National Weather Service, 1999).

PMP estimates from HMR 58/59 have been compared to PMP estimates from HMR 36 throughout California. Results were compared for a 24-hour duration and 10 square mile area. The areas of greatest increase of HMR 58/59 results compared to HMR 36 were generally confined to orographic regions. Results showed localized increases between HMR 36 and HMR 58/59 of up to 24 inches in 24 hours in the San Gabriel and San Bernadino Mountains.

The County has developed a revised PMF based on the PMP from HMR 58/59. As shown on Figure 6-3, the revised peak of the PMF inflow hydrograph is 22,100 cfs (County, 2000).

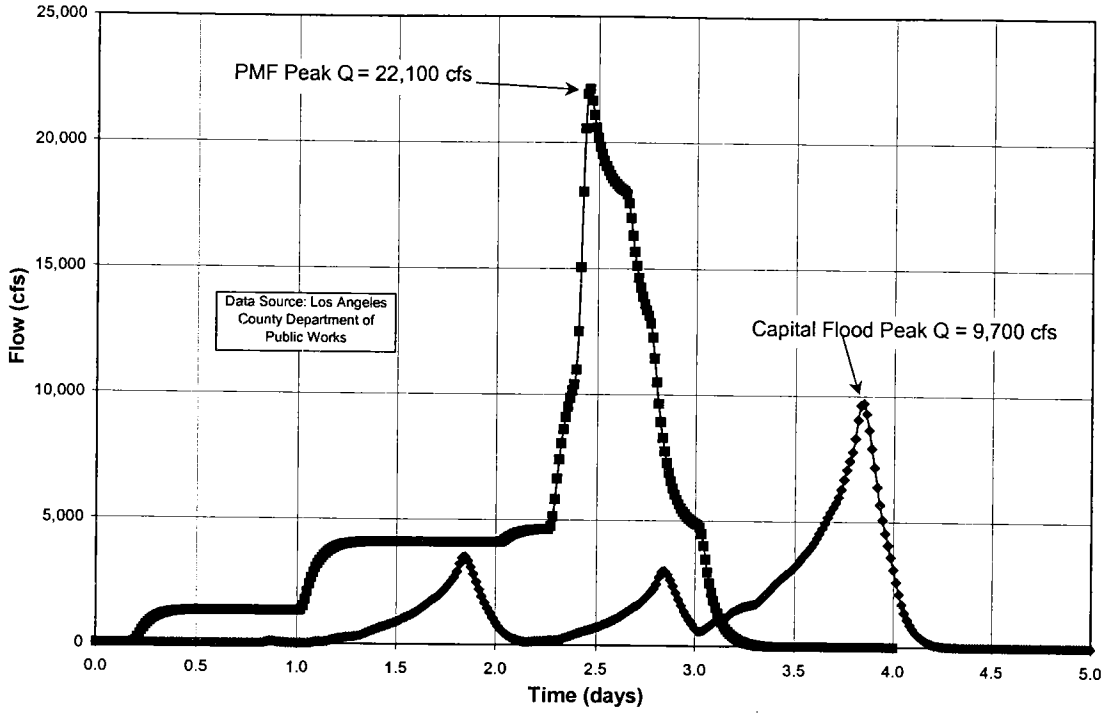


Figure 6-3: Probable Maximum Flood and Capital Flood Inflow Hydrographs

6.3.2 Capital Flood

The Capital Flood is runoff from a 50-year frequency design storm falling on a saturated watershed (County, 1991). The Capital Flood is generally applied in the design of open channels, closed conduits, bridges, and dams and debris basins not under State of California jurisdiction, that are constructed in or intercept flood waters from natural watercourses having watershed areas greater than 100 acres. The objective is that modifications to Santa Anita Dam will not increase the outflow from the Capital Flood in comparison to the existing conditions. The storm occurs over a period of four days, with the maximum rainfall falling on the fourth day. As shown on Figure 6-3, the peak of the Capital Flood inflow hydrograph is 9,700 cfs. The County developed the Capital Flood (County, 2000).

6.4 Reservoir Flood Routing

Reservoir flood routing was performed using a standard hydrologic analysis computer program, the HEC-1 Flood Hydrograph Package (COE, 1990). As a result of a previous seismic stability evaluation, the DSOD currently limits the maximum normal water level to El 1280, but allows temporary impoundment during storm periods. The initial water level for the flood routing was El 1280. Reservoir elevation-storage data was taken from Plate 5-6 of the Phase I Inspection Report (DSOD, 1980). Although the reservoir elevation-storage

data may be somewhat inaccurate due to sedimentation, results show that storage is a negligible factor in reducing outflow.

Complete HEC-1 output for routing of the PMF is included in Appendix G. The peak inflow and peak outflow are essentially equal at 22,100 cfs. The peak reservoir water level for the PMF is at El 1332.0.

Complete HEC-1 output for routing of the Capital Flood is included in Appendix H. The peak inflow of 9,700 cfs is essentially the same as the peak outflow. The peak reservoir water level for the Capital Flood is at El 1329.5.

6.5 Spill Trajectory

Spill trajectory is a consideration for determining impact zones and areas of potential scour. Spill impact zones are more of a consideration following resolution of spillway modifications, but example spill trajectories are presented on Figure 6-4. The trajectory of spill will vary with water level in the reservoir, flow rates in the spillways, and the type of spillway. The spill trajectory methodology was as presented in "Design of Small Dams" (USBR, 1987).

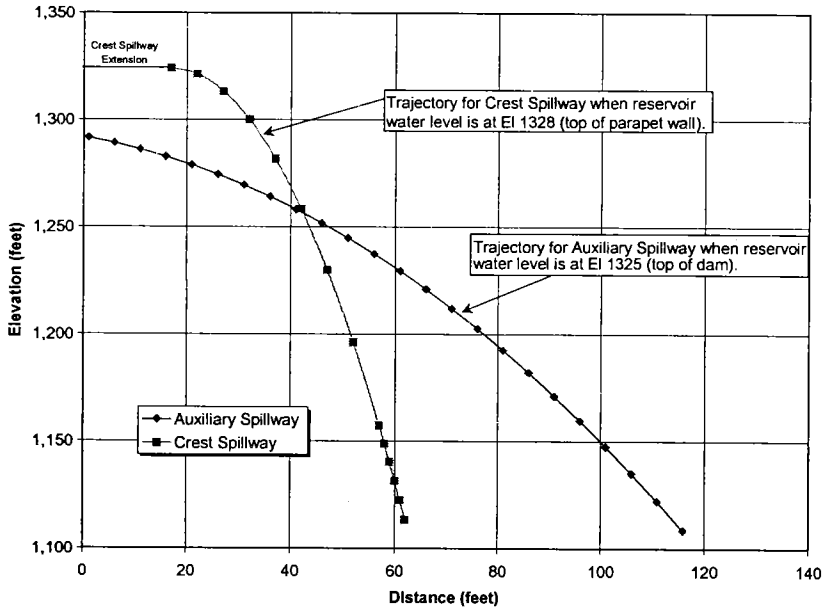


Figure 6-4: Spill Trajectory Distance from Downstream Dam Face

SECTION SEVEN

7.0 CONCLUSIONS

Evaluations of the dam's performance and stability have shown that the dam, as it currently exists, could potentially be rendered unsafe during the occurrence of a Maximum Credible Earthquake (MCE). The results from the hydrologic reanalysis also show that the existing spillway and outlet works will not pass the Probable Maximum Flood (PMF) without overtopping. Continued deterioration of concrete due to alkali-aggregate reaction (AAR) is, optimistically, uncertain. Summaries of the evaluations are given below.

The EQ1 ground motion record used to simulate the MCE for the seismic reanalysis is a Richter magnitude 7.25 ±0.25 event on the Sierra Madre Fault. Due to the very close proximity to the fault it may generate a peak horizontal ground acceleration (pga) of 1.45g, which is more than twice the horizontal pga of 0.7g from the ground motion record used in the 1977 analysis. Linear and non-linear seismic analyses using EQ1 show that the tensile stresses in the dam exceed the maximum permissible tensile strength values; more specifically, the stresses exceed the maximum permissible tensile strength of 650 psi for about seventeen cycles during the MCE in the non-linear (ADAP-NS) model and approximately over 30 cycles in the linear (ANSYS) model. These findings suggest that the occurrence of the MCE could potentially render the dam unsafe for operation in its current structural condition.

The current magnitude of the runoff resulting from the PMF, as determined by Los Angeles County Department of Public Works, is 22,100 cfs. HEC-1 output shows that the parapet wall of the dam would be overtopped by four feet during the PMF. Overtopping could potentially cause erosion of bedrock in the first 120 feet of the downstream abutments.

Recent laboratory studies of the concrete have shown the presence of AAR in all concrete samples taken from the concrete coring in November 1999. Studies of the current survey data also show that the dam has moved upstream approximately one and one-half inches during the past 30 years. In agreement with concrete experts, Harza has concluded that observed upstream movements are due to AAR related expansion and that "the cumulative effect is quantifiable." Thus, seismic retrofit alternatives should address concerns about AAR-related expansion, which could potentially affect the integrity of a retrofit.

As shown by the results of the seismic analyses, hydrologic studies, and AAR studies, risk factors exist that could potentially threaten the stability of the dam. Harza recommends that remedial action alternatives be discussed and evaluated to seismically retrofit and increase spillway capacity of the existing dam.

SECTION EIGHT

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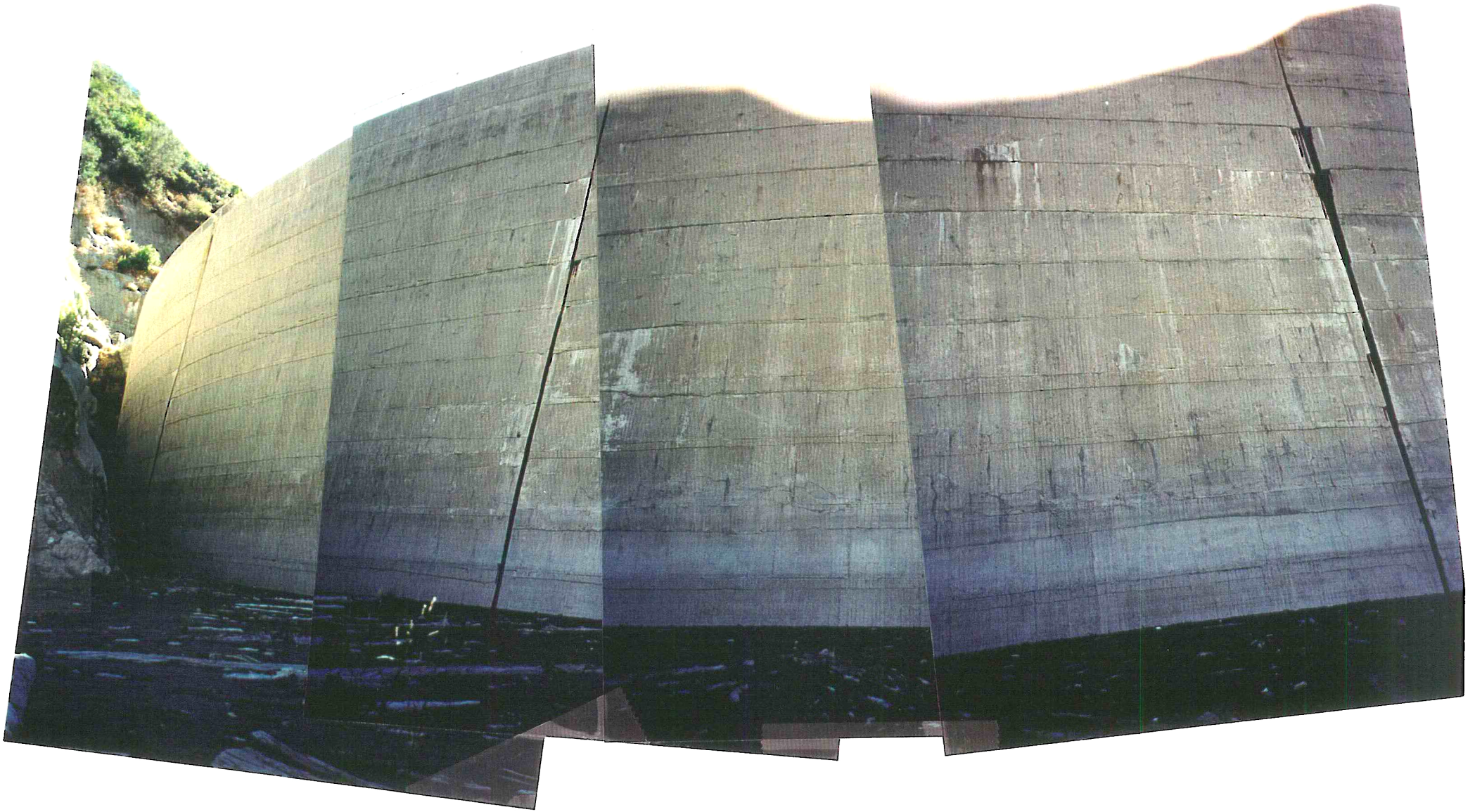
APPENDIX A

Photographs from Field Reconnaissance

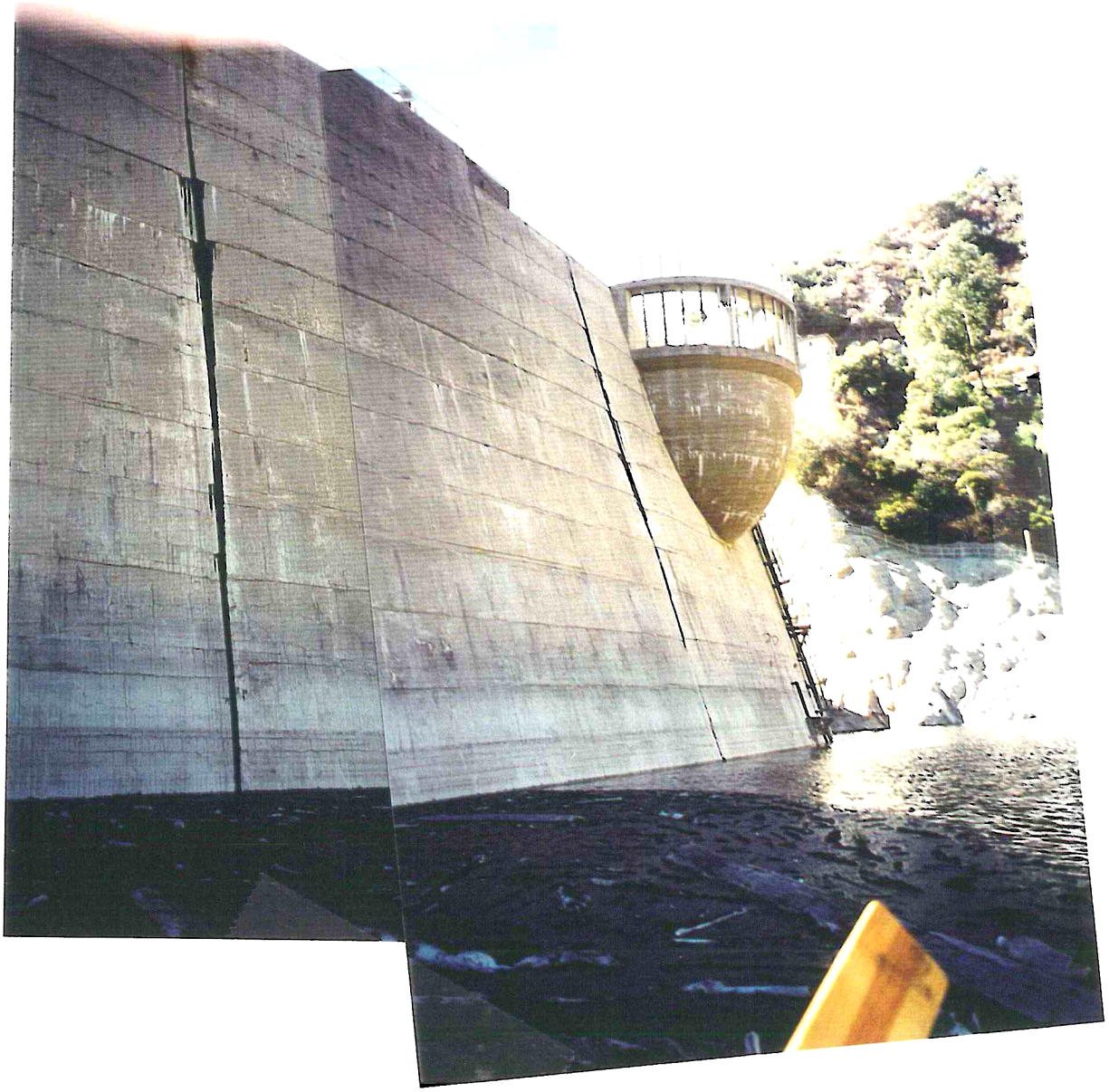
Upstream View of Dam
Showing Right Abutment

Upstream View of Dam
Showing Left Abutment

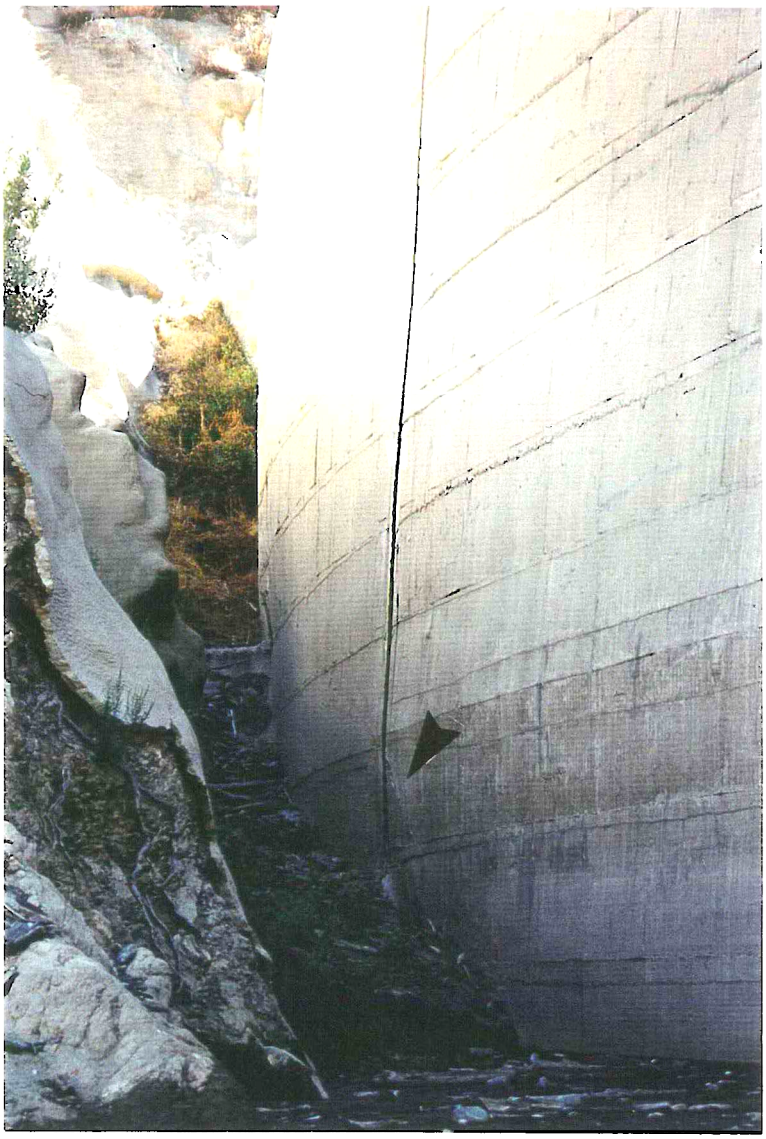


Upstream Face, Reservoir
El. 1267

Detail of Right Abutment and
Main Spillway Intake
(Upstream)

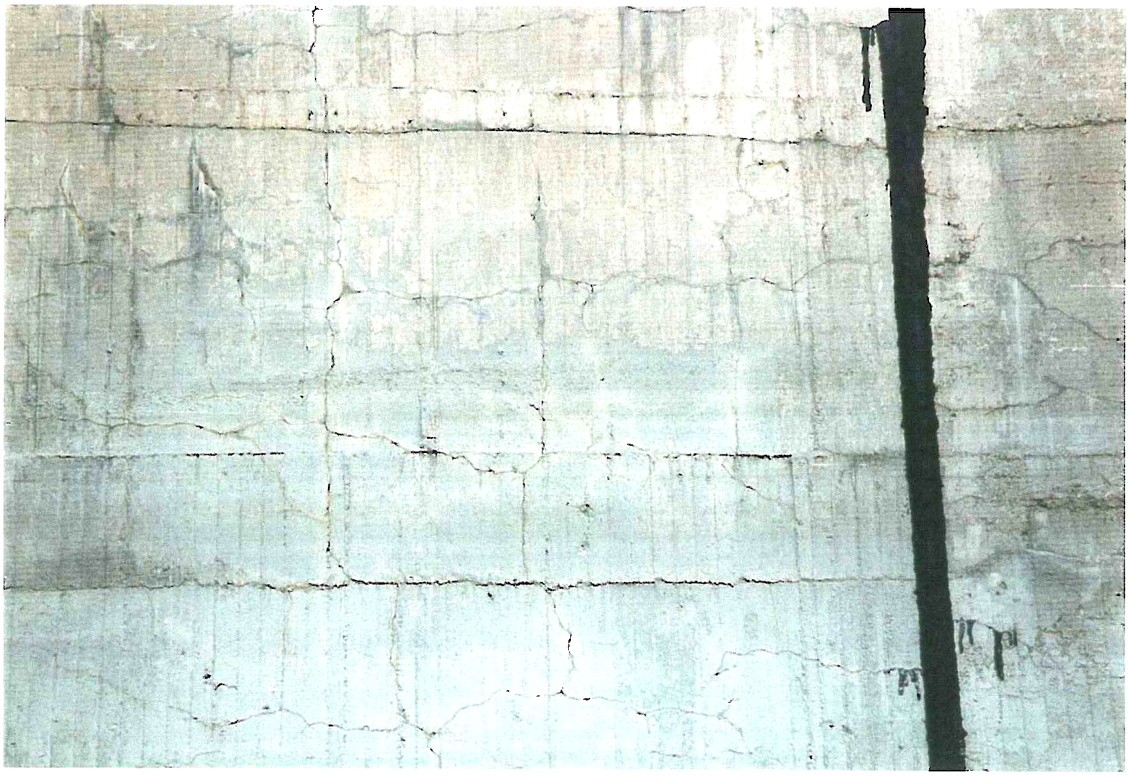


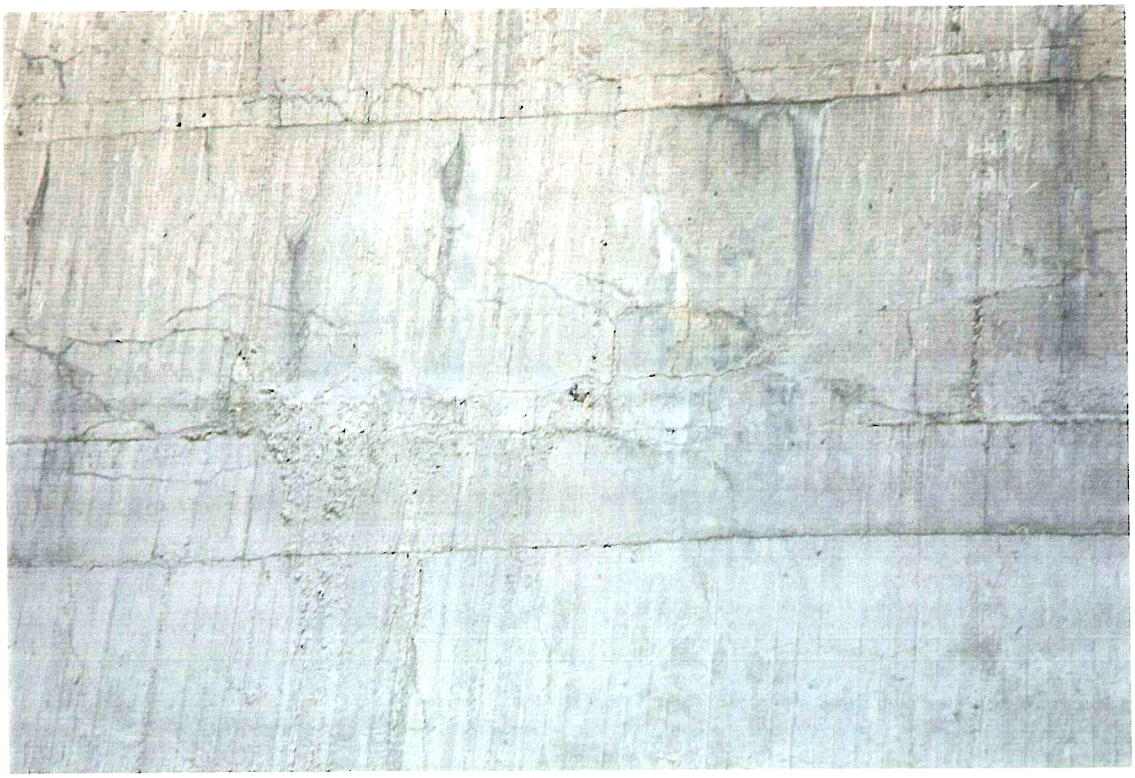
Upstream Face,
Facing Auxiliary Spillway and
Right Abutment



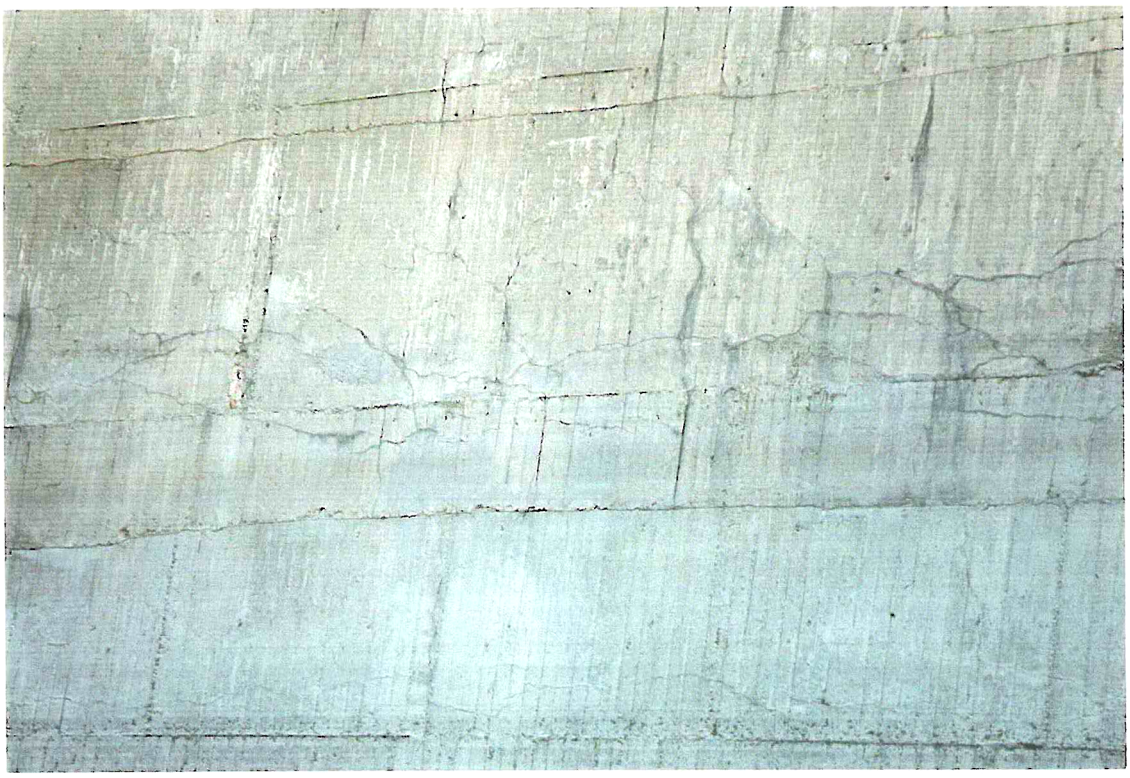
Upstream Face, Detail
Of Crack Near Joint #8
(Left)

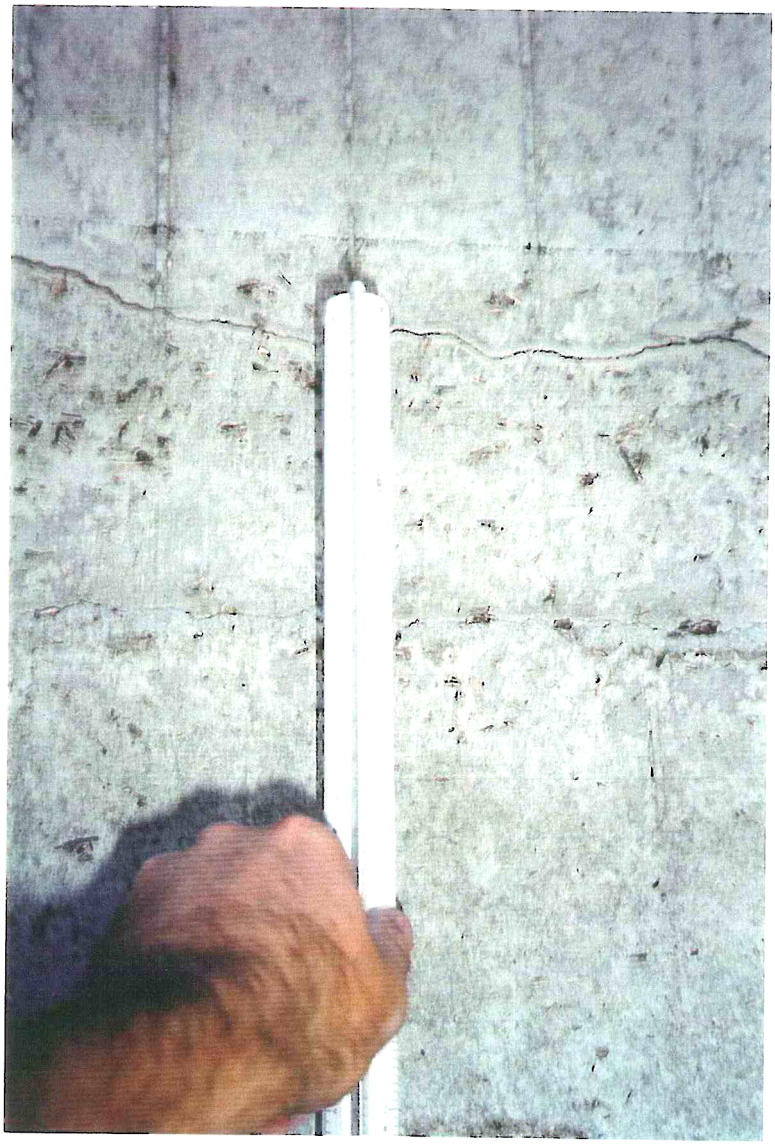
Upstream Face, Detail of Map
Cracking Near Joint #7,
El 1275 (Below)



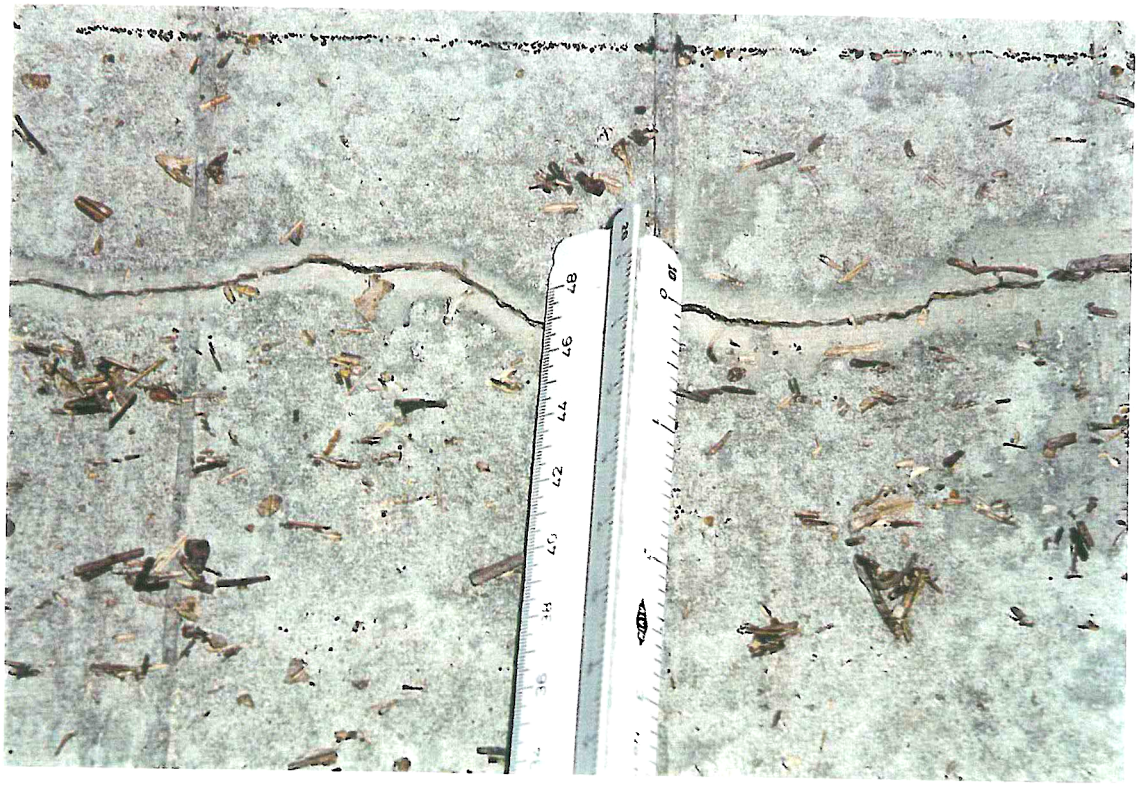


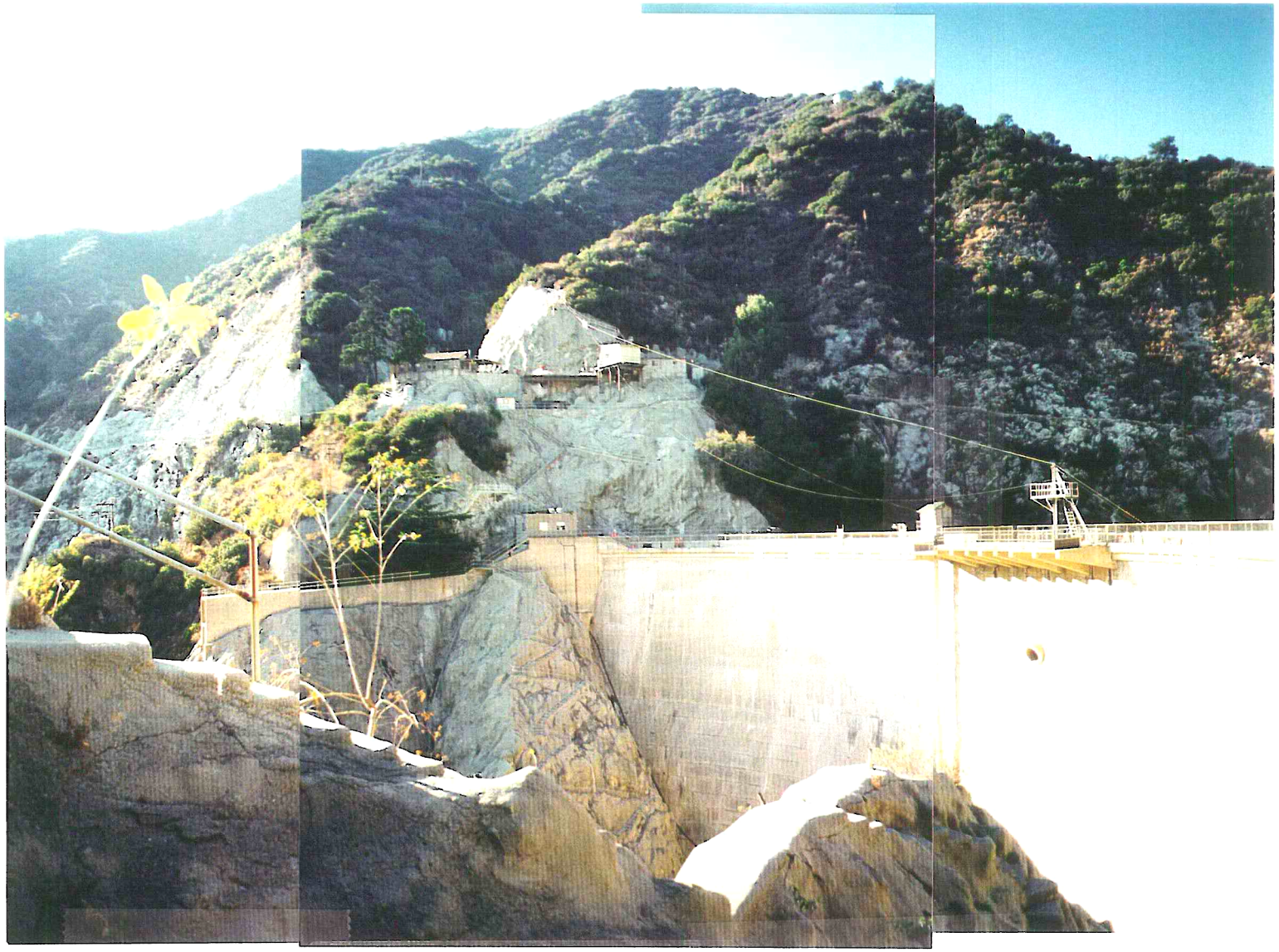
Upstream Face, Examples of
(Above and Below)
Cracking Near Joint #7, El. 1275





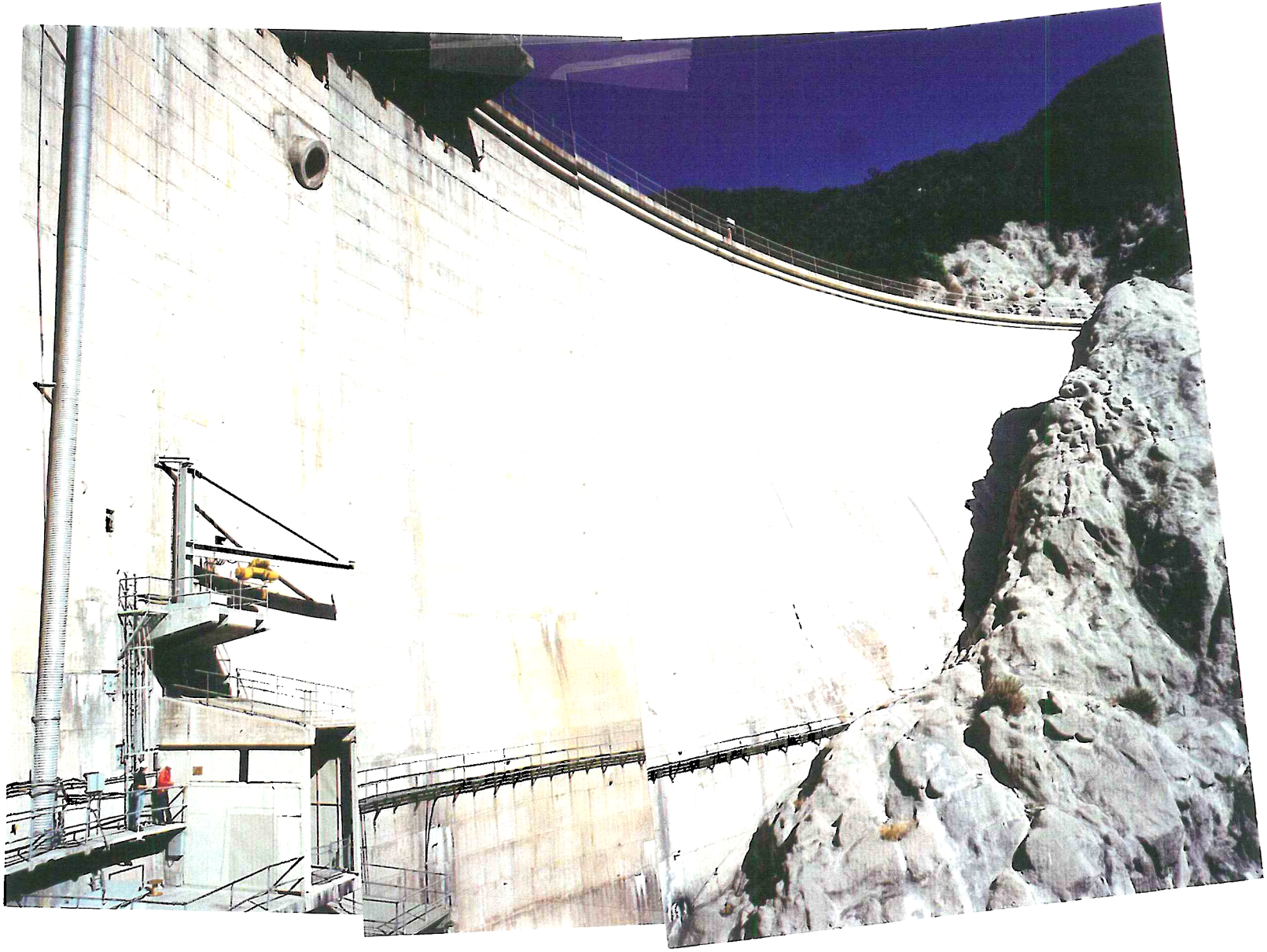
US Face, Detail of Cracking
Near Joint #6, El 1270
(On Left, Close-Up Below)





View of Downstream Face,
Right Abutment, Main and
Auxiliary Spillway

View of Downstream Face,
Outlet Works' Housing and
Right Abutment



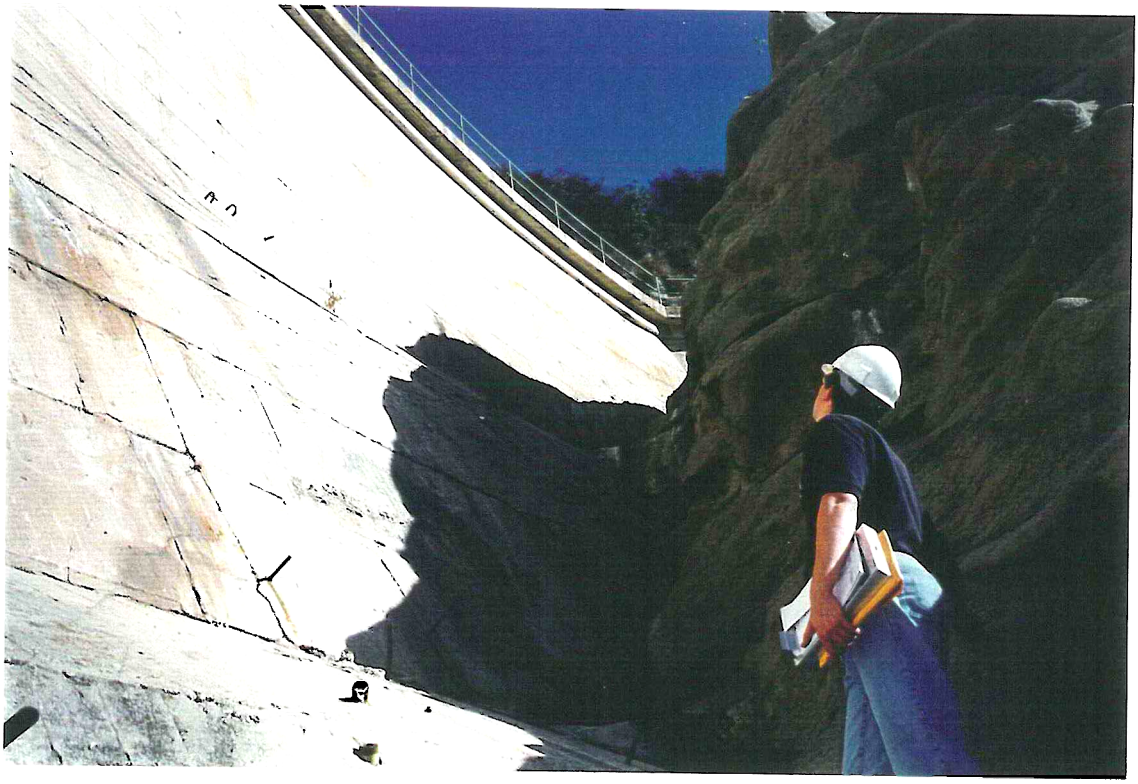
View of Downstream Face
and Left Abutment

Downstream View of
Sluice-way at El. 1179 5

Main Spillway and
Control House

Downstream Face, Detail of Joint #6 (Left)

Downstream Face, Detail of Left Abutment (Below)



Downstream Face, Detail of
Transverse Cracking near
Right Abutment

Downstream Face, Detail of Vertical Crack Near Right Abutment (Left)

Downstream Face, Detail of Cracking Near Left Abutment, El 1205 (Below)

Los Angeles County
Department of Public Works

Draft Final Report

Santa Anita Dam Reanalysis Study Phase 2 – Concept Validation and Preliminary Design

MWH

Date: November 2005

November 1, 2005
1001105

Keith Lilley
Water Resources Division
Department of Public Works,
County of Los Angeles
900 South Freemont Avenue
Alhambra, CA 91803-1331

Attention. Aracely Lasso

Subject: Draft Final Report
Santa Anita Dam Reanalysis Study
Phase 2 – Concept Validation and Preliminary Design

Dear Mr Lilley·

MWH Americas, Inc. (MWH) is pleased to submit the enclosed Draft Final Report for the subject project.

We appreciate this opportunity to serve the County If you have any questions regarding this report, please fell free to call me at 925-975-3400.

Sincerely,

Daniel L. Wade

Daniel L. Wade, P.E., G.E.
Project Manager

DLW:sa

E:\Shared\Working Projects\L A D P W\Santa Anita\Tasks4and5\Report\Draft Final Report-Cover Letter.doc

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PROFESSIONAL CERTIFICATION

DRAFT FINAL REPORT

SANTA ANITA DAM REANALYSIS STUDY
PHASE 2 – CONCEPT VALIDATION AND PRELIMINARY DESIGN
for
Los Angeles County Department of Public Works

November, 2005

This report has been prepared by MWH Americas, Inc (MWH) under the professional supervision of the Principal and senior staff whose seals and signatures appear herein. The findings, interpretations of data, recommendations, specifications or professional opinions are presented within the limits prescribed by available information at the time the report was prepared, in accordance with generally accepted professional engineering and geologic practice and within the requirements by the Client. There is no other warranty, either expressed or implied.

The findings of this report are based on the readily available data and information obtained from public and private sources. Additional studies (at greater cost) may or may not disclose information which may significantly modify the findings of this report. In the event that there are any changes in the nature, design or location of the project, or if additional subsurface data are obtained or any future additions are planned, the conclusions and recommendations contained in the report will need to be reevaluated by MWH in light of the proposed changes or additional information obtained.

This report was prepared solely for the benefit of MWH's Client. No other entity or person shall use or rely upon this report or any of MWH's work products unless expressly authorized by MWH. Any use of or reliance upon MWH's work product by any party, other than the Client, shall be solely at the risk of such party

"Draft"

"Draft"

Daniel L. Wade, P.E., G.E.
Principal

Vik Iso-Ahola, P.E.
Project Engineer

1.0 INTRODUCTION

1 1 Introduction

Santa Anita Dam is located on the Big Santa Anita Creek approximately four miles north of the City of Arcadia, Los Angeles County, California. The dam, constructed in 1924 through 1927, is owned and operated by the County of Los Angeles Department of Public Works (LACDPW).

The dam is a constant-angle variable-radius concrete arch constructed for the purpose of flood control and water conservation. The dam's crest height above the streambed at the downstream toe of the dam is 225 feet (dam crest El. 1,325), and the crest length of the arch (including the abutments) is 612 feet. The thickness of the dam varies from 10 feet at the crest (7 feet at the arch crest) to 61.5 feet at the base. The mass concrete was placed in blocks between nine full height vertical trapezoidal construction joints spaced approximately 50 feet apart.

The main spillway (crest El. 1,316) consists of a rectangular covered box section channel with rounded entrance lip and sidewalls located on the right abutment. The total length of the chute section is approximately 170 feet along the centerline. An auxiliary spillway consists of a semi bowl-shaped inlet on the upstream dam face discharging into a circular conduit through the dam near the crest. Several valves (two needle valves each with diameter of 30 inches and one gate valve with diameter of 20 inches) are located between 120 to 160 feet below the dam crest in the central portion of the arch to regulate water level in the reservoir. A fourth 18-inch diameter gate valve is located near the base of the dam, and a 72-inch diameter hydraulic sluice gate is located approximately 146 feet below the dam crest.

The Dam has been investigated and its hydraulic and seismic safety has been evaluated by International Engineering Company (IECO, 1977), LACDPW (1978) and the State of California Department of Water Resources, Division of Safety of Dams (DSOD, 1980).

Previous studies of the Santa Anita Dam have indicated the need for safety improvements for two types of possible (though highly unlikely) extreme events:

a. Probable maximum flood (PMF); and

b. Maximum credible earthquake (MCE).

The "Report – Supplemental Services for Santa Anita Dam Reanalysis Study" (MWH February 6, 2002) concluded Phase 1 of the current study. The Report presented analyses for five spillway augmentation concepts and two seismic rehabilitation concepts, which were considered as alternatives to mitigate the dam for the PMF and MCE, respectively. Specifically, each concept was analyzed to include hydraulic analyses, 3-D finite element dynamic stress analyses, constructability, and cost. However, an error in the finite element analyses was found,

requiring a re-evaluation of the finite element model results presented in the report. Those results are presented in Appendix A.

LACDPW subsequently requested that MWH perform analysis on two additional concepts, "Partial Rehabilitation" and "Intake Riser Modification" for delivery of two thirty percent design concepts to complete Phase 2 of the study. MWH performed and submitted, via email on December 27 & 30, 2004, additional analyses to evaluate a revised, two-level notch configuration and to establish a "safe" reservoir level for the Intake Riser Modification concept. The safe reservoir level for Intake Riser Modification concept was unofficially accepted by DSOD during a conference call between MWH, DSOD, and LACDPW on January 6, 2005 (see minutes in Appendix B). Also, MWH performed an analysis of the Partial Rehabilitation concept as part of this report. These two combined hydraulic/seismic rehabilitation concepts have been further developed and validated, as presented herein this Phase 2 Report.

1.2 Approach

The objective of Phase 2 is to provide for validation of the above-cited combined hydraulic/seismic rehabilitation concepts for modification of Santa Anita Dam to safely withstand extreme flooding and earthquake criteria. Accordingly, the following analyses have been performed and are presented herein for the combined hydraulic/seismic rehabilitation concepts:

- Structural design validation analyses; and
- Hydraulic design validation analyses.

These supplemental Phase 2 tasks, along with the analyses previously performed under Phase 1 bring the selected combined hydraulic/seismic rehabilitation concepts to the 30% design level, including the following preliminary drawings presented in this report:

- A plan view drawing of modifications; and
- Profile and typical cross-section of modifications.

A preliminary list of drawings to be developed during final design is also provided in this report.

1.3 Document Organization

Following this introductory section, Section 2.0 of this report presents a verbal summary of the modifications and the preliminary design drawings. Section 3.0 of this report presents the structural design validation analyses, and Section 4.0 presents the hydraulic design validation analyses for the combined hydraulic/seismic rehabilitation concepts. Constructability and opinions of probable construction cost are presented in

Section 5.0, considerations related to alkali-aggregate reaction are presented in Section 6.0, and a comparison of dam modification alternatives is presented in Section 7.0. References are listed in Section 8.

1.4 Acknowledgement

This Phase 1 Report for Santa Anita Dam and associated studies and analyses were prepared by a multi-disciplinary team of MWH professionals and consultants. Vik S. Iso-Ahola, P.E., Civil Engineer performed the linear finite element analyses prepared main portions of the report. John C. Haapala, P.E. performed hydraulic analyses and descriptions. Daniel L. Wade, P.E., Project Manager and Glenn Tarbox, P.E., Principal-in-Charge provided supervision, technical direction, and review for main portions of the report. We would like to express our appreciation to the staff of the Los Angeles County Department of Public Works for their cooperation in providing data, guidance, and review and comment of documents.

2.0 PROJECT DESCRIPTION

2.1 Description of Combined Hydraulic/Seismic Rehabilitation Concepts

2.1.1 *Concept 1 – Partial Rehabilitation*

The Partial Rehabilitation concept would require modifications to the existing dam, constructing a notch spillway centered on the downstream channel (oblique channel arm downstream of existing channel armoring) to safely pass the newly updated PMF of 26,100 cfs, constructing an ogee spillway and flip bucket extension on the notch spillway, constructing a 6-foot diameter octagonal orifice spillway with upstream invert at El. 1270 and exiting at El 1238 (through the buttress), and constructing a downstream buttress or thickening of the existing arch dam to reduce stress levels to be mostly within the linear elastic range of concrete below El. 1270. The angled orifice and flip-bucket would discharge most flows beyond the toe of the thickened dam centered into the channel over 110-feet downstream, thus preventing undermining erosion at the dam toe. The thickened dam would provide a safe normal operating reservoir level at El 1271

Drawings 2-1 and 2-2 show a 0.55 to 1 downstream slope and 15-foot wide top width at El. 1270 for the concrete buttress to be constructed as part of this concept. The sluiceway and valves penstocks would be extended through the buttress for use in continued operation and sluicing of the reservoir below El. 1270.

2.1.2 *Concept 2 – Intake Riser Modification*

In this concept, the proposed project will include construction of a notch spillway as in Concept 1 for passing the PMF, except that the notch would be centered on the crest of the dam, similar to the existing emergency spillway. This notch configuration takes advantage of the existing armoring downstream of the dam. In addition, the reservoir would be drawn down through the existing sluice tunnel near the base of the dam, thus, no orifice would be constructed. All gates/valves would be abandoned in place and their associated intakes and/or trash racks would be extended to El 1246 to promote/expedite drainage through the outlets above the elevation of sediment accumulation associated with the Design Debris Event (DDE), as shown in Drawings 2-3 and 2-4. The existing sluice gate would be removed and installed on the outside of a new riser constructed to El. 1230 in order to allow for continued sediment sluicing. The normal operating reservoir 'safe elevation' has been established (described in Section 3.0) at El 1231 for the modified dam.

2.1.3 *Spillway Notch*

Both rehabilitation concepts described above accommodate overtopping during the PMF and seismic rehabilitation for the MCE by restricting the reservoir level either to El. 1270 (partial rehabilitation concept) or El. 1230 (intake riser modification concept). The proposed spillway modifications are shown in the attached 30 percent design level

Drawings 2-1 and 2-2 for the partial rehabilitation concept and Drawings 2-3 and 2-4 for the intake riser modification concept.

As can be seen in the drawings, the crest of the new notch spillway (intake riser modification) will extend to El 1300 at a forty-foot (40') width and widens by seventy-two feet (72') at El 1310 for a total width of one hundred twelve feet (112'). The partial rehabilitation notch spillway will be smaller due to use of an ogee crest and flip bucket with dimensions at El 1300 at a thirty-five feet (35') width and widens by forty-five feet (45') at El 1310 for a total width of eighty feet (80'). The six-foot diameter octagonal orifice spillway for the partial rehabilitation concept is centered below the spillway notch with the invert of the orifice at El 1270 on the upstream and El. 1238 at its outlet on the downtream face of the buttress.

2.1.4 *Spillway Bridge*

A spillway bridge for pedestrian and equipment access will be constructed across the spillway notch for both options (Drawing 2-2 & 2-4). Because a large amount of debris during the PMF could impact against the northern half of the proposed bridge crossing, it is doubtful that a proposed bridge at El. 1325 (crest elevation) could withstand a substantial debris accumulation, even with strengthening. Thus, the new bridge will be raised to El 1330 (cord) by raising the proposed bridge foundation and ramps.

2.1.5 *Downstream Armoring*

For both concepts, the downstream channel below the new spillway configuration will be armored with reinforced concrete or shotcrete to dissipate the hydraulic energy from the overtopping water as the flow cascades through the spillway notch and the orifice spillway or sluiceway. The flow will be directed onto the downstream armoring before flowing into the channel downstream of the dam.

The preliminary extents of the steel-reinforced concrete/shotcrete armoring for the partial rehabilitation and intake riser modification are shown in Drawing 2-1 and 2-3, respectively, where the hatching indicates the location of additional armoring above and around the existing armoring. Both drawings show armoring extending approximately 100 feet downstream from the toe of the rehabilitated dam alternatives and approximately 130 feet wide with a minimum thickness of 3 feet. The armoring will be held in position with tie-back anchors to be drilled and grouted into the bedrock.

2.2 Sequence of Work

After LACDPW has selected a preferred alternative for final design, the work will generally proceed in the following manner:

- Evaluate disposal and/or re-use options for concrete removed from the dam,

- Perform geotechnical investigations, as necessary This would include a geotechnical investigation of the parking area on the right abutment slated to be used as the staging area for equipment and heavy concrete blocks removed from the dam (both concepts) and geotechnical investigations in the abutments and in the buttress foundation for the partial rehabilitation concept;
- Evaluate and make recommendations of the existing electrical, valves, and instrumentation systems for the dam controls,
- Prepare final design criteria for the preferred dam modification/rehabilitation and develop construction plans and specifications;
- Prepare 60% design plans and specifications and submit to LACDPW LACDPW will forward documents to DSOD; MWH will meet with LACDPW and DSOD to obtain their approval for the 60% design,
- Prepare 90% design plans and specifications and submit to LACDPW; LACDPW will forward documents to DSOD; MWH will meet with LACDPW and DSOD to obtain their approval for the 90% design,
- Prepare draft 100% design plans and specifications and submit to LACDPW; LACDPW will forward documents to DSOD; meet with LACDPW and DSOD to obtain their approval for the Draft 100% Design,
- Prepare construction quantities, engineer's cost estimate and schedule;
- Submit the final, 100% design plans and specifications to LACDPW. LACDPW will forward the documents to DSOD; and
- Assist LACDPW during the bidding process by attending a pre-bid meeting and answering questions from LACDPW and potential bidders.

2.3 Preliminary List of Drawings for Final Design

We anticipate that the drawings listed below will be developed for final design. We emphasize that this list is preliminary; the total number and scope of drawings may change based on final design requirements. Technical specifications will be developed in support of the drawings.

- Title Sheet, Site Location Map and Drawing Index
- Symbols, Abbreviations and General Notes
- Site Plan
- Survey Control
- Dam Modification – Demolition Plan and Sections
- Dam Modification – Demolition Elevation and Sections
- Dam Modification – Demolition Sections and Details
- Dam Modification – Plan and Profile
- Dam Modification – Sections and Details, Sheet 1
- Dam Modification – Sections and Details, Sheet 2
- Dam Modification – General Structural Notes
- Dam Modification – General Structural Details, Sheet 1
- Dam Modification – General Structural Details, Sheet 2
- Dam Modification – General Structural Details, Sheet 3
- Dam Modification – General Structural Details, Sheet 4

- Dam Modification – General Structural Details, Sheet 5
- Dam Modification – Gallery, Sheet 1 *(only for Partial Rehabilitation)*
- Dam Modification – Gallery, Sheet 2 *(only for Partial Rehabilitation)*
- Dam Modification – Drainage Plan, Profile and Section *(only for Partial Rehabilitation)*
- Dam Modification – Drainage Sections *(only for Partial Rehabilitation)*
- Dam Modification – Foundation Excavation Plan *(only for Partial Rehabilitation)*
- Dam Modification – Foundation Excavation Profile *(only for Partial Rehabilitation)*
- Dam Modification – Foundation Grouting and Treatment Plan *(only for Partial Rehabilitation)*
- Dam Modification – Intake Riser/Trashrack Modification Plan and Profile, Sheet 1 *(only for Intake Riser Modification)*
- Dam Modification – Intake Riser/Trashrack Modification Plan and Profile, Sheet 2 *(only for Intake Riser Modification)*
- Dam Modification – Intake Riser/Trashrack Modification Sections, Sheet 1 *(only for Intake Riser Modification)*
- Dam Modification – Intake Riser/Trashrack Modification Sections, Sheet 2 *(only for Intake Riser Modification)*
- Dam Modification – Notch Spillway Plan
- Dam Modification – Notch Spillway Sections and Details
- Dam Modification – Valvehouse Floor Plan *(only for Partial Rehabilitation)*
- Dam Modification – Valvehouse Sections and Details *(only for Partial Rehabilitation)*
- Dam Modification – Valvehouse Roof Plan *(only for Partial Rehabilitation)*
- Dam Modification – Valvehouse Transverse Section *(only for Partial Rehabilitation)*
- Downstream Armoring – Plan and Sections
- Downstream Armoring – Sections and Details
- Pedestrian/Equipment Bridge (outline drawing only; final design by bridge supplier)
- Dam Accessways Downstream – Plan and Sections
- Dam Accessways Downstream – Sections and Details, Sheet 1
- Dam Accessways Downstream – Sections and Details, Sheet 2
- Miscellaneous Metal, Sheet 1
- Miscellaneous Metal, Sheet 2
- Miscellaneous Metal, Sheet 3
- Miscellaneous Metal, Sheet 4
- Miscellaneous Metal, Sheet 5
- Mechanical Standard Symbols and Notes
- Mechanical Standard Details
- Penstock General Arrangement Plan *(only for Partial Rehabilitation)*
- Penstock Details, Sheet 1 *(only for Partial Rehabilitation)*
- Penstock Details, Sheet 2 *(only for Partial Rehabilitation)*
- Penstock Details, Sheet 3 *(only for Partial Rehabilitation)*
- Penstock Details, Sheet 4 *(only for Partial Rehabilitation)*
- Valve House Mechanical Plan *(only for Partial Rehabilitation)*

- Valve House Mechanical Elevation *(only for Partial Rehabilitation)*
- Valve House Mechanical Sections and Details *(only for Partial Rehabilitation)*
- Valve House Water and Drainage Plan *(only for Partial Rehabilitation)*
- 18" Outlet, Sections and Details *(only for Partial Rehabilitation)*
- 30" Outlet, Sections and Details *(only for Partial Rehabilitation)*
- 30" Outlet, Sections and Details *(only for Partial Rehabilitation)*
- 60" Outlet (Sluiceway), Sections and Details *(only for Partial Rehabilitation)*
- Hoist Tower (Jib Crane) General Arrangement Plan and Sections
- Sluiceway Gate Relocation and Modification Plan and Section *(only for Intake Riser Modification)*
- Sluiceway Gate Relocation and Modification Details *(only for Intake Riser Modification)*
- Electrical Standard Symbols and Abbreviations
- Power One Line Diagram
- Panel Schedules
- Electrical Site Plan
- Lighting Details and Schedule
- Electrical Demolition Details
- Dam Gallery Power, Lighting & Control Plan, Sheet 1*(only for Partial Rehabilitation)*
- Dam Gallery Power, Lighting & Control Plan, Sheet 2 *(only for Partial Rehabilitation)*
- Dam Power, Lighting & Control Plan, Sheet 1
- Dam Power, Lighting & Control Plan, Sheet 2
- Dam Power, Lighting & Control Plan, Sheet 3
- Dam Power, Lighting & Control Plan, Sheet 4
- Dam Sections and Details Sheet 1
- Dam Sections and Details Sheet 2
- Dam Sections and Details Sheet 3
- Valve House Power & Control Plan *(only for Partial Rehabilitation)*
- Valve House Sections and Details *(only for Partial Rehabilitation)*
- Valve House Lighting Plan *(only for Partial Rehabilitation)*
- Communication and Alarm Details
- Communication and Alarm Valve House
- Communication and Alarm Miscellaneous Plans
- Utility Routing Relocation Plan
- Utility Routing Relocation Details, Sheet 1
- Utility Routing Relocation Details, Sheet 2
- Instrumentation Plan
- Instrumentation Profile
- Instrumentation Details, Sheet 1
- Instrumentation Details, Sheet 2
- Instrumentation Details, Sheet 3
- Valve Control Panel Arrangement Elevation & Details *(only for Partial Rehabilitation)*
- Valve Control Panel Schematic, Sheet 1 *(only for Partial Rehabilitation)*

- Valve Control Panel Schematic, Sheet 2 *(only for Partial Rehabilitation)*
- Valve Control Panel Schematic, Sheet 3 *(only for Partial Rehabilitation)*
- Valve Control Panel Schematic, Sheet 4 *(only for Partial Rehabilitation)*
- Valve Control Panel Interconnection Details *(only for Partial Rehabilitation)*
- Valve Control Panel Raceway Routing Plan *(only for Partial Rehabilitation)*
- Valve Control Panel, Exist. Controller Modification Details *(only for Partial Rehabilitation)*

3.0 STRUCTURAL DESIGN VALIDATION

This section presents the findings of the dynamic finite element method (FEM) analyses performed for both dam rehabilitation concepts, including the proposed spillway modifications for the intake riser modification and buttress and spillway modifications for the partial rehabilitation. The computer program ANSYS was used to model linear-elastic (no vertical construction joints in the model) response of the rehabilitation concepts. The purpose of these analyses was to provide an indication of locations where the proposed modified dam concepts may be potentially overstressed during the EQ1, EQ2, and EQ3 ground motion record. These ground motion records were previously presented in the "Existing Condition Assessment Report for Santa Anita Dam" (Harza Engineering Company / April 3, 2000).

The FEM model, material properties, modal analyses, hydrodynamic effects, results and conclusions of the static and dynamic analyses are summarized below.

3.1 Introduction

An FEM model was previously developed using computer program ANSYS as presented in the "Report – Supplemental Services for Santa Anita Dam Analysis Study" (MWH, 2002). This model was modified to evaluate the dynamic response of the existing dam with the above-cited proposed spillway modifications and strengthening. Namely, the FEM model was modified to include a 70-foot wide notch placed at the center of arch with the bottom of the notch at El. 1305, and an octagonal orifice inscribed inside an 8-foot diameter circle, placed horizontally through the dam with the bottom of the orifice at El. 1270.

The above FEM model was modified to validate the structural design for the extreme seismic condition of two combined hydraulic/seismic rehabilitation concepts. The combined concepts, previously described in Section 2.0, consist of an intake riser modification and partial rehabilitation. The intake riser modification consists of cutting a 112-foot wide stepped notch centered on the dam crest (lower step width of notch is 40-feet), extending the intake risers, and restricting the maximum reservoir elevation to 1230-feet. The partial rehabilitation has a similar notch configuration as the intake riser modification, but slightly smaller due to the ogee spillway crest with the lower level notch width at 35 feet and total width of 80 feet. Also, the partial rehabilitation consists of thickening the existing dam up to elevation 1270, has a 6-foot diameter orifice, and has a restricted maximum reservoir elevation of 1270-feet.

Results of analyses of the two rehabilitated dam concepts performed with EQ1, EQ 2, and EQ3 are presented herein. In addition, results of static stress analyses have been evaluated, including:

- Gravity load (i.e., self-weight of the dam);

- Reservoir water load; and
- Temperature load for both summer and winter conditions.

The internal stresses due to static loading are superimposed on dynamic stresses, element by element, such that the stresses due to static plus dynamic loading conditions may be reviewed.

3.2 Finite Element Model

3.2.1 *Model Development*

As part of the current validation study, this model was further developed to better simulate the updated geometry of each rehabilitation concept. The automated meshing capabilities of ANSYS were utilized to model the site geometry and the dam structure in the finite element model. The partial rehabilitation model shown in Figures 3-1 to 3-2 consists of 6,785 elements and 17,468 nodes. The intake riser modification shown in Figure 3-33 model consists of 4,105 elements and 13,703 nodes. In both models, mostly hexagonal ("brick") elements are used, while prism ("wedge") and tetrahedral elements are utilized in portions of the model where the geometry is not regular All elements have mid-side nodes in addition to the nodes at the corners of the elements.

3.2.2 *Hydrodynamic Effects and Model Calibration*

The reservoir hydrodynamic effects on the rehabilitated dam concepts were modeled up to El. 1270 and El. 1230 (partial rehabilitation and intake riser modification, respectively) using the added mass approach in accordance with the procedure established by Zanger (1952). The unit weights of water and equivalent unit weight of saturated silt were assumed to be 62.5 pcf and 82.5 pcf, respectively

The modal periods calculated for the modified dam for both of the lowered reservoir conditions described above (partial rehabilitation and riser modification) are presented in Table 3-1 For comparison, the modal periods for the existing dam at full reservoir level (El. 1316) and for the empty reservoir condition are also listed in Table 3-1 The modal periods for the full and empty reservoir conditions were previously presented in the "Existing Condition Assessment Report" (Harza, April 3, 2000).

As can be seen in Table 3-1, the modal periods calculated for the modified dam with the lower reservoir levels are closer to those of the empty reservoir model of the existing dam. This is due to the fact that there is a significant increase in dam thickness between El. 1325 (full reservoir elevation) and the modified reservoir elevations (El. 1270 and El. 1230). In addition, a portion of the dam upper area is removed due to notching for the PMF (both concepts) and is thickened at the base (partial rehabilitation concept), effectively increasing stiffness and thereby reducing modal frequencies. Thus, the hydrodynamic effects for the lower reservoir conditions have significantly reduced effect on the modal

periods of the dam as compared to the full reservoir condition, causing the dam to behave very much as if the reservoir were empty

Table 3-1 Modal Analysis Comparison

Mode Number	Empty Reservoir (period, s)	Riser Modification, Reservoir El. 1230 (period, s)	Partial Rehabilitation, Reservoir El. 1270 (period, s)	Full Reservoir, Reservoir El. 1300 (period, s)
1	0.15	0.14	0.12	0.18
2	0.14	0.13	0.10	0.16
3	0.11	0.11	0.09	0.12
4	0.10	0.10	0.09	0.11
5	0.08	0.08	0.07	0.09
6	0.08	0.08	0.07	0.09
7	0.07	0.07	0.06	0.08
8	0.06	0.06		0.07
9	0.06	0.06		0.07
10	0.06	0.06		0.07
20	0.04	0.04		0.04

3.2.3 *Material Properties*

The material properties used in the dam models are shown in Table 3-2. These material properties are the same as those used in the "Existing Condition Assessment Report" (Harza, April 3, 2000).

Table 3-2 Material Properties
Finite Element Analysis
Santa Anita Dam

	Static Properties		Dynamic Properties	
Property	Concrete	Rock	Concrete	Rock
Density	148 pcf	0 (Massless)	148	0 (Massless)
Poisson's Ratio	0.2	0.2	0.2	0.2
Coefficient of Thermal Expansion	5.6×10^{-6} / °F	-		-

Elastic Modulus	2.9 x 10^6 psi	2.6 x 10^6 psi*	3.4 x 10^6 psi (above El. 1,275) 4.2 x 10^6 psi (below El. 1,275)	2.6 x 10^6 psi*

* Mass modulus of elasticity of foundation and abutment rock is 2.6 x 10^6 psi except as follows:

- Right abutment above El. 1,300: 5.0 x 10^5 psi
- Right abutment below El. 1,300: 2.0 x 10^6 psi
- Right abutment between El. 1,170 and 1,190: 1.0 x 10^6 psi
- Left abutment above El. 1,300: 1.2 x 10^6 psi

3.3 Finite Element Model Results

Dynamic linear elastic analyses were performed on the rehabilitated dam concepts with the design EQ1, EQ2, and EQ3 ground-motion records previously developed as described in Harza (2000). A constant damping ratio of 5 percent was used for all analyses.

The results of dynamic analyses are presented in the following subsections. In addition, the most critical dynamic stresses were superimposed on the following static load cases to simulate pre-earthquake conditions for both the summer and winter·

- Summer Condition = Gravity (dead) load + hydrostatic water load at maximum pool (El. 1270 for Partial Rehabilitation and El. 1230 for Intake Riser Modification) + summer temperature load,
- Winter Condition = Gravity (dead) load + hydrostatic water load at maximum pool (El. 1270 for Partial Rehabilitation and El. 1230 for Intake Riser Modification) + winter temperature load.

3.3.1 *Results of Dynamic Finite Element Analyses for Partial Rehabilitation*

3.3.1.1 Results Using EQ 1 Ground Motion Record, Partial Rehabilitation

The EQ1 ground motion is a modified version of the 1978 Tabas, Iran earthquake scaled to spectrally match the target spectra for the Santa Anita Dam.

Figures 3-1 and 3-2 provide node locations on the partial rehabilitation model and show the finite element model geometry, respectively After the dynamic analysis, a macro was run in ANSYS to determine which time-points during EQ1 had the largest tensile

stresses in the dam during the summer and winter conditions. The highest tensile stresses were found at time (t) = 11.18 seconds and 11.42 seconds during the winter condition. Figures 3-3 and 3-4 were plotted to investigate the stresses at these time points. These figures present the maximum tensile (major principal) stresses within the dam during the earthquake and are summarized in Table 3-3 with the winter static conditions superimposed. It is to be noted and emphasized that the values of maximum stresses shown in Table 3-3 are not to be accumulated. These are maximum values based on stress contours shown in the figures, which occur in different locations (elements) in the dam at different times.

Table 3-3. EQ1 DYNAMIC STRESSES*
Santa Anita Dam
Partial Rehabilitation

	Winter Conditions			
	Time = 11.18 sec.		Time = 11.42 sec.	
	Max Tension (psi)	Max Compression (psi)	Max Tension (psi)	Max Compression (psi)
Upstream Face				
S1	1375	0	353	0
S3	0	702	0	600
Downstream Face				
S1	195	0	450	0
S3	0	715	0	1,310

*Reports maximum values within the dam based on stress contours excluding stress concentrations on or near the dam abutments due to non-regular element shape, transition from hexagonal to tetrahedral elements and/or geometric anomaly (i.e., sharp geometric transition in the model).
S1 = Major Principal Stress
S3 = Minor Principal Stress

The time-stress histories at El. 1260, 1270, 1280 through the dam and at the El. 1300 spillway notch level on both the upstream and downstream faces of the dam are included as Figures 3-5 through 3-12, which show major principal stresses with winter static conditions superimposed. As can be seen in Figure 3-5, the peak stress at node 9987 exceeds the dynamic tensile strength of concrete for one excursion at 11 18 seconds on the upstream face during the EQ1 time-stress history However, the tensile stresses through the dam (Figure 3-6) are well below the dynamic tensile strength of concrete for the entire time-history Figure 3-7 plots the time-history for downstream face node 10307 opposite of the

upstream face node 9987, showing that the maximum tensile stress in the dam exceeds the dynamic tensile strength of the concrete for two excursions.

Node 450 (Figure 3-7) shows one tensile stress excursion while the interior nodes show none (Figure 3-8) and node 10410 (Figure 3-9) shows about 9 excursions. The stress excursions at node 10410 can be attributed to the sharp change in geometry and resulting stress concentrations at the dam-buttress interface at El. 1270. The stress distribution at this location is typified in the lower plot of Figure 3-4, where the yellow and red stress contours show the concentration of peak stresses along the dam-buttress interface. The bottom of Figure 3-9 shows the stress time-history for node 15718 on downstream face of the buttress, 15 feet from node 10410. As shown, the tensile stresses for the entire time-history are below the dynamic tensile strength of concrete.

Figures 3-10 to 3-12 show a time-history profile of stresses through the dam at El. 1260. In addition to the interior nodes (nodes 10619, 10621, & 10616), nodes on the upstream face (node 446) and downstream face (node 15690) show no stress excursions above the dynamic tensile strength of concrete.

3.3.1.2 Results Using EQ 2 Ground Motion Record, Partial Rehabilitation

The EQ2 ground motion was previously described in "Existing Condition Assessment Report" (Harza, 2000). This ground motion is a modified version of the original M7.4 1978 Tabas, Iran earthquake, scaled to spectrally match the target spectra for the Santa Anita Dam.

After the dynamic analysis, a macro was run in ANSYS to determine which time-points during EQ2 had the largest tensile stresses in the dam during the summer and winter conditions. The highest stresses were found at t = 10.04 seconds and 11 14 seconds during the winter condition. Figures 3-13 and 3-14 were plotted to investigate the stresses at these time points. The nodes described below were selected based upon locations of stress concentrations.

The time-stress histories at El. 1260, 1270, 1280 through the dam and at the El. 1300 spillway notch level on both the upstream and downstream faces of the dam are included as Figures 3-15 through 3-22, which show major principal stresses with winter static conditions superimposed. As can be seen in Figure 3-15, the stress at node 9987 exceeds the dynamic tensile strength of concrete for one excursion at 11 14 seconds on the upstream face during the EQ2 time-stress history. However, the tensile stresses through the dam (Figure 3-16) are well below the dynamic tensile strength of concrete for the entire time-history Figure 3-17 plots the time-history for downstream face node 10307 opposite of the upstream face node 9987, showing no excursions of the dynamic tensile strength in the dam.

Nodes 450, 10412, and 10430 (Figure 3-17 and 3-18) show no tensile stress excursions while node 10410 (Figure 3-19) shows 3 excursions. The stress excursions at node 10410 can be attributed to the sharp change in geometry and resulting stress

concentrations at the dam-buttress interface at El. 1270. The stress distribution at this location is typified in the lower plot of Figure 3-13, where the yellow and red stress contours show the concentration of peak stresses along the dam-buttress interface. The bottom of Figure 3-19 shows the stress time-history for node 15718 on downstream face of the buttress, 15' from node 10410 As shown, the tensile stresses for the entire time-history are below the dynamic tensile strength of concrete.

Figures 3-20 to 3-22 show a time-history profile of stresses through the dam at El. 1260. In addition to the interior nodes (nodes 10619, 10621, & 10616), nodes on the upstream face (node 446) and downstream face (node 15690) show no stress excursions above the dynamic tensile strength of concrete.

3.3.1.3 Results Using EQ 3 Ground Motion Record, Partial Rehabilitation

The EQ3 ground motion was previously described in "Existing Condition Assessment Report" (Harza, 2000). This ground motion is a modified version of the 1994 Northridge, California earthquake as recorded at Sylmar Converter Station-East, scaled to spectrally match the target spectra for the Santa Anita Dam.

After the dynamic analysis, a macro was run in ANSYS to determine which time-points during EQ3 had the largest tensile stresses in the dam during the summer and winter conditions. The highest stresses were found at t = 3.80 and 3.84 during the winter condition. Figures 3-23 and 3-24 were plotted to investigate the stresses at these time points. The nodes described below were selected based upon locations of stress concentrations.

The time-stress histories at El. 1260, 1270, 1280 through the dam and at the El. 1300 spillway notch level on both the upstream and downstream faces of the dam are included in Figures 3-25 through 3-32, which show major principal stresses with winter static conditions superimposed. Figure 3-25 shows that the peak stress for node 10862 at lower spillway notch (El. 1300) exceeds the dynamic tensile strength of concrete for one excursion on the upstream face at 3.58 seconds. However, the tensile stresses at El. 1280 on upstream face and through the dam (Figure 3-25 to 3-26) are below the dynamic tensile strength of concrete for the entire time-history Figure 3-27 plots the time-history for downstream face node 10307 opposite of the upstream face node 9987, showing one excursion approaching the dynamic tensile strength in the dam.

Nodes 450, 10412, and 10430 (Figure 3-17 and 3-18) show no tensile stress excursions while node 10410 (Figure 3-29) shows 4 excursions. The stress excursions at node 10410 can be attributed to the sharp change in geometry and resulting stress concentrations at the dam-buttress interface at El. 1270. The stress distribution at this location is typified in the lower plot of Figure 3-23, where the yellow and red stress contours show the concentration of peak stresses along the dam-buttress interface. The bottom of Figure 3-29 shows the stress time-history for node 15718 on downstream face of

the buttress, 15' from node 10410. As shown, the tensile stresses for the entire time-history are below the dynamic tensile strength of concrete.

Figures 3-30 to 3-32 show a time-history profile of stresses through the dam at El. 1260. In addition to the interior nodes (nodes 10619, 10621, & 10616), nodes on the upstream face (node 446) and downstream face (node 15690) show no stress excursions above the dynamic tensile strength of concrete.

3.3.2 *Results of Dynamic FEM Analyses for Intake Riser Modification*

3.3.2.1 Results Using EQ 1 Ground Motion Record, Intake Riser Modification

The EQ1 ground motion is a modified version of the 1978 Tabas, Iran earthquake scaled to spectrally match the target spectra for the Santa Anita Dam.

Figure 3-33 provides node locations on the intake riser modification model and shows the finite element model geometry. After the dynamic analysis, a macro was run in ANSYS to determine which time-points during EQ1 had the largest tensile stresses in the dam during the summer and winter conditions. The highest stresses were found at t = 10.10 seconds and 11.42 seconds during the summer condition. Figures 3-34 and 3-35 were plotted to investigate the stresses at these time points. These figures present the maximum tensile (major principal) stresses within the dam during the earthquake, which are summarized in Table 3-4 with the summer static conditions superimposed. It is to be noted and emphasized that the values of maximum stresses shown in Table 3-4 are not to be accumulated. These are maximum values based on stress contours shown in the figures, which occur in different locations (elements) in the dam at different times.

Table 3-4: EQ1 DYNAMIC STRESSES*
Santa Anita Dam
Intake Riser Modification

	Summer Conditions			
	Time = 10.1 sec.		Time = 11.42 sec.	
	Max Tension (psi)	Max Compression (psi)	Max Tension (psi)	Max Compression (psi)
Upstream Face				
S1	1,372	0	1,250	0
S3	0	1,080	0	1,180

Downstream Face				
S1	1,368	0	1,170	0
S3	0	710	0	707

*Reports maximum values within the dam based on stress contours excluding stress concentrations on or near the dam abutments due to non-regular element shape, transition from hexagonal to tetrahedral elements and/or geometric anomaly (i.e., sharp geometric transition in the model).
S1 = Major Principal Stress
S3 = Minor Principal Stress

Time-stress histories at El. 1220 and 1200 through the dam and on both the upstream and downstream faces of the dam are included in Figures 3-36 to 3-39, which show principal stresses with winter static conditions superimposed. As can be seen in Figure 3-36, the stress at node 987 exceeds the dynamic tensile strength of concrete twice at 10.10 and 11.42 seconds on the downstream face during the EQ1 time-stress history However, the tensile stresses through the dam (node 992 and 990) are well below the dynamic tensile strength of concrete for the entire time-history The bottom of Figure 3-37 plots the time-history for downstream face node 985 (opposite of the upstream face node 987), showing no excursions of the dynamic tensile strength of the concrete. Node 721 (Figure 3-38) shows eighteen tensile stress excursions while the interior nodes 727 and 725 and upstream face node 723 show none (Figure 3-38 & 3-39). The stress excursions at node 721 generally surficial and are shown to decrease significantly on the dam interior at node 727 (Figure 3-38).

Figures 3-40 and 3-41 were plotted to further investigate the high tensile stresses in the dam occurring at 11.42 seconds, showing arch and cantilever stresses in the dam. Comparison of Figure 3-40 (arch stress contours) vs. Figure 3-41 (cantilever stress contours) shows that a large portion of tensile stress in the dam during EQ1 at 11.42 seconds is in the arch direction. It is expected that a portion of the tensile stresses in the arch direction would be alleviated by contraction joint opening and closing.

Figures 3-42 to 3-53 show the arch and cantilever stress time-histories El. 1220 and El. 1200 through the dam (node locations shown on Figure 3-33). Only cantilever stress time-histories are shown in Figures 3-42 to 3-45 due to the limitations of plotting arch stress time-histories in ANSYS, where stress time-histories can only be shown in ANSYS in the global Cartesian coordinate system. Arch stresses occur in the dam in the arch direction, which is not compatible when plotting time-histories in the global coordinate system. However, Figures 3-46 to 3-53 were plotted to show the arch stress time-histories in addition to cantilever stress time-histories. Arch stresses are shown for these nodes because they are located at the center of the arch, which is tangent to the global Cartesian coordinate system at this location. The time-history plots in Figures 3-42 to 3-53 demonstrate that the tensile stresses in the cantilever direction are generally below the dynamic tensile strength

of concrete, except for one tensile stress excursion at nodes 723 (Figure 3-44) and 721 (Figure 3-45) on the upstream and downstream face, respectively In addition, the arch stress time-histories in Figures 3-46 to Figure 3-53 show that all tensile stresses at the center of the arch below El. 1230 are within the dynamic tensile strength of concrete.

3 3.2.2 Results Using EQ 2 Ground Motion Record, Intake Riser Modification

The EQ2 ground motion was previously described in "Existing Condition Assessment Report" (Harza, 2000). This ground motion is a modified version of the original M7.4 1978 Tabas, Iran earthquake, scaled to spectrally match the target spectra for the Santa Anita Dam.

After the dynamic analysis, a macro was run in ANSYS to determine which time-points during EQ2 had the largest tensile stresses in the dam during the summer and winter conditions. The highest stresses were found at t = 10.20 seconds and 11.38 seconds during the summer condition. Figures 3-54 and 3-55 were plotted to investigate the stresses at these time points. These figures present the maximum tensile (major principal) stresses within the dam during the earthquake. The nodes described below were selected based upon locations of stress concentrations.

The time-stress histories at El. 1220 and El. 1200 through the dam and on both the upstream and downstream faces of the dam are included in Figures 3-56 through 3-59 These plots show principal stresses with summer static conditions superimposed. As can be seen in Figure 3-56, the stress at node 987 exceeds the dynamic tensile strength of concrete for two excursions at 10.20 and 11.38 seconds on the upstream face during the EQ2 time-stress history However, the tensile stresses through the dam (nodes 992 and 990) and on the downstream face (node 985) are well below the dynamic tensile strength of concrete for the entire time-history Node 721 (Figure 3-58) shows thirteen tensile stress excursions while the interior nodes 727 and 725 and upstream face node 723 show none (Figure 3-58 & 3-59). The stress excursions at node 721 are generally surficial and are shown to decrease significantly within 12-feet on the dam interior at node 727

3.3.2.3 Results Using EQ 3 Ground Motion Record, Intake Riser Modification

The EQ3 ground motion was previously described in "Existing Condition Assessment Report" (Harza, 2000). This ground motion is a modified version of the 1994 Northridge, California earthquake as recorded at Sylmar Converter Station-East, scaled to spectrally match the target spectra for the Santa Anita Dam.

After the dynamic analysis, a macro was run in ANSYS to determine which time-points during EQ3 had the largest tensile stresses in the dam during the summer and winter conditions. The highest stress was found at t = 3.58 seconds during the summer condition. Figure 3-60 was plotted to investigate the stress at this time point. This figure presents the

maximum tensile (major principal) stresses within the dam during the earthquake. The nodes described below were selected based upon locations of stress concentrations.

The time-stress histories at El. 1220 and El. 1200 through the dam and on both the upstream and downstream faces of the dam are included in Figures 3-61 through 3-64. These plots show principal stresses with summer static conditions superimposed. As can be seen in Figure 3-61, the stress at node 987 exceeds the dynamic tensile strength of concrete for one excursion at 3.58 seconds on the upstream face during the EQ2 time-stress history. However, the tensile stresses through the dam (nodes 992 and 990) and on the downstream face (node 985) are well below the dynamic tensile strength of concrete for the entire time-history. Node 721 (Figure 3-63) shows five tensile stress excursions while the interior nodes 727 and 725 and upstream face node 723 show none (Figure 3-63 & 3-64). The stress excursions at node 721 generally surficial and are shown to decrease significantly on the dam interior at node 727 (Figure 3-63).

3.4 Summary and Conclusions of Structural Validation Studies

The figures and tables presented herein show that the results of dynamic stress analyses for EQ1, EQ2, and EQ3 are similar in magnitude. Specifically, the maximum stresses occur in approximately the same locations on the dam during EQ1, EQ2, and EQ3 and have approximately the same maximum values. However, because EQ1 was matched to the site spectral curve, EQ1 shows the highest tensile stress excursions in terms of magnitude and frequency. Thus, EQ1 analyses for both rehabilitation alternatives are considered the controlling loading cases. The results for both rehabilitation concepts are summarized below.

3.4.1 *Partial Rehabilitation*

The maximum tensile stress on the upstream face of the partially rehabilitated dam exceeds the previously established dynamic tensile strength of concrete near the restricted reservoir El. 1270. However, the tensile stress excursions on the upstream face only occur once in each of the three earthquakes evaluated (e.g., Figure 3-5). In addition, results from dynamic analyses using EQ1, EQ2 and EQ3 show that tensile stresses on the downstream face of the proposed concrete buttress do not exceed the tensile strength of concrete.

Additional stress excursions occur on the existing dam downstream face (Figure 3-9) at the crest of the buttress due to a sharp change in geometry (corner at the dam-buttress interface). These stresses, however, are localized at the crest of the interface and are shown to decrease significantly within 7-feet of the existing dam (Figure 3-8).

Although several tensile stress excursions occur in the partially rehabilitated dam during EQ1, EQ2, and EQ3, it should be noted that these tensile stresses are localized, momentary stresses that do not extend through the dam. While the MCE may or may not produce some minor cracking on the upstream and downstream faces of the rehabilitated dam, this cracking would mainly occur near or above the restricted reservoir level (El.

1270) during the extreme MCE event. Thus, the risk for immediate release of reservoir is considered neglible.

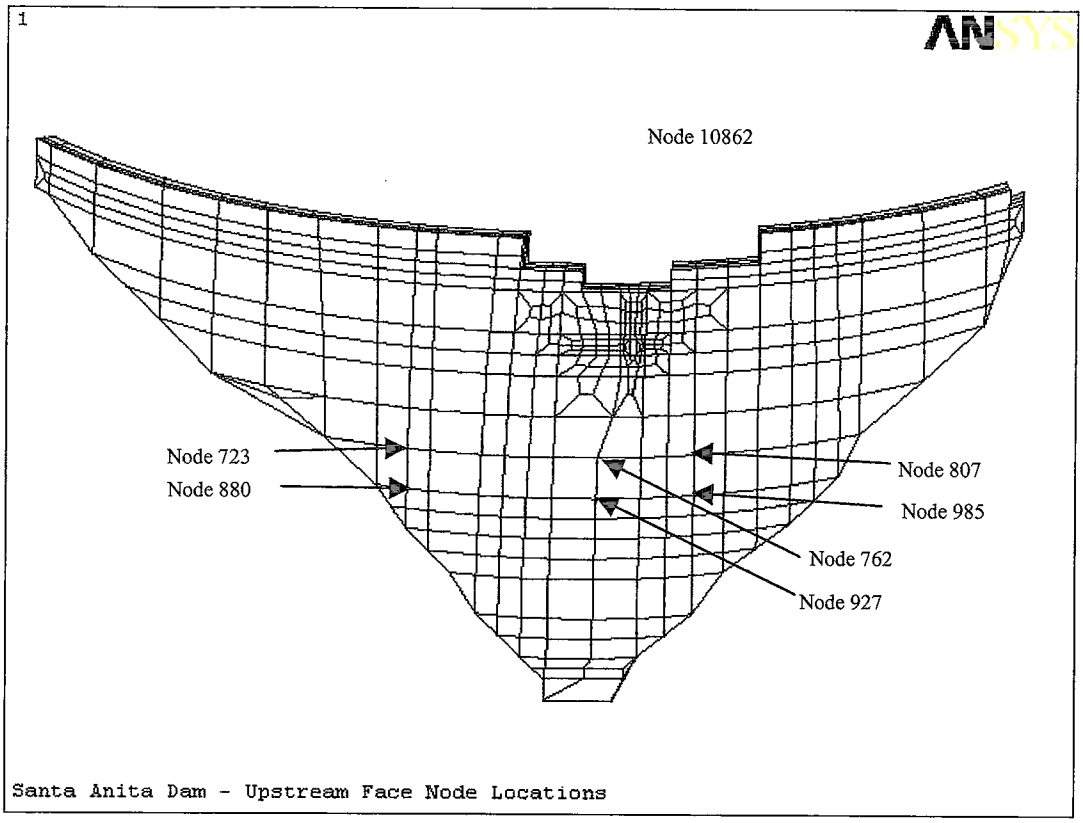
Based on these validation studies, the proposed concrete buttress to El. 1270 should provide the needed strengthening of the existing dam to provide safety during the MCE event (safe El. 1271).

3.4.2 *Intake Riser Modification*

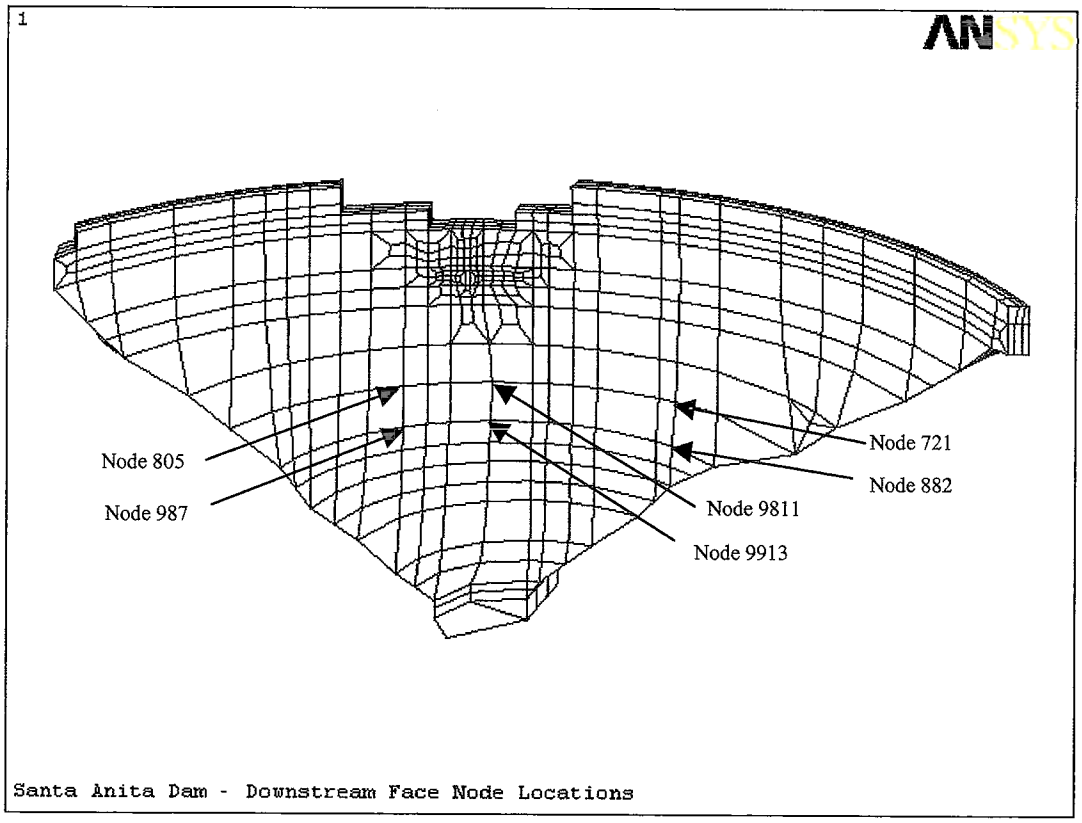
The maximum tensile (major principal) stress on the upstream face of the dam exceeds the previously established dynamic tensile strength of concrete immediately above El 1230 at the center of the arch for EQ1, EQ2 and EQ3 dynamic analyses. However, these stresses are demonstrated to be in the arch direction and thus should be alleviated through opening and closing of the contraction joints. Furthermore, while damage and cracking to the dam may occur above El. 1230 on the upstream face, these tensile stress excursions do not extend below the restricted reservoir elevation.

Results from EQ1, EQ2 and EQ3 also show that major principal (tensile) stresses on the downstream face of the dam exceed the tensile strength of concrete for up to eighteen cycles during EQ1, thirteen cycles during EQ2, and four times during EQ3 (see Figures 3-38, 3-58, and 3-63). It should be noted, however, that the tensile stress distribution in the arch and cantilever direction of the dam during these cycles are substantially lower (Figures 3-40 & 3-41) and are primarily above El. 1230. Moreover, the major principal tensile stresses are shown to be below the maximum tensile stress within 12 feet of the dam (Figure 3-38). Therefore, while the MCE may or may not produce some minor cracking on the downstream face at and above the El. 1230 reservoir restriction, the risk for immediate reservoir release during or after the extreme MCE event is considered low

Based on these validation studies, the proposed permanent reservoir restriction to El. 1230 (reservoir capacity ~60 acre-feet) should reduce dynamic loading of the existing dam and provide a safe permanent reservoir elevation during the MCE event (safe El. 1231).

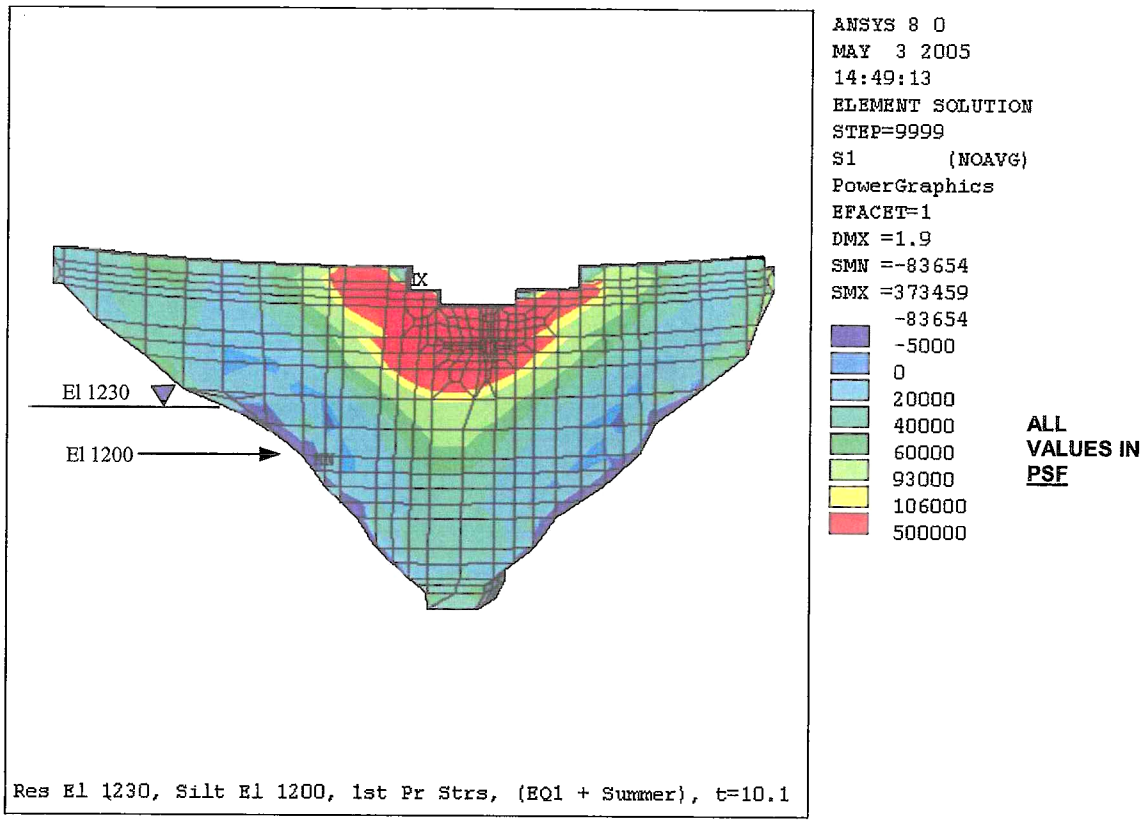


Upstream Face – Intake Riser Modification Model

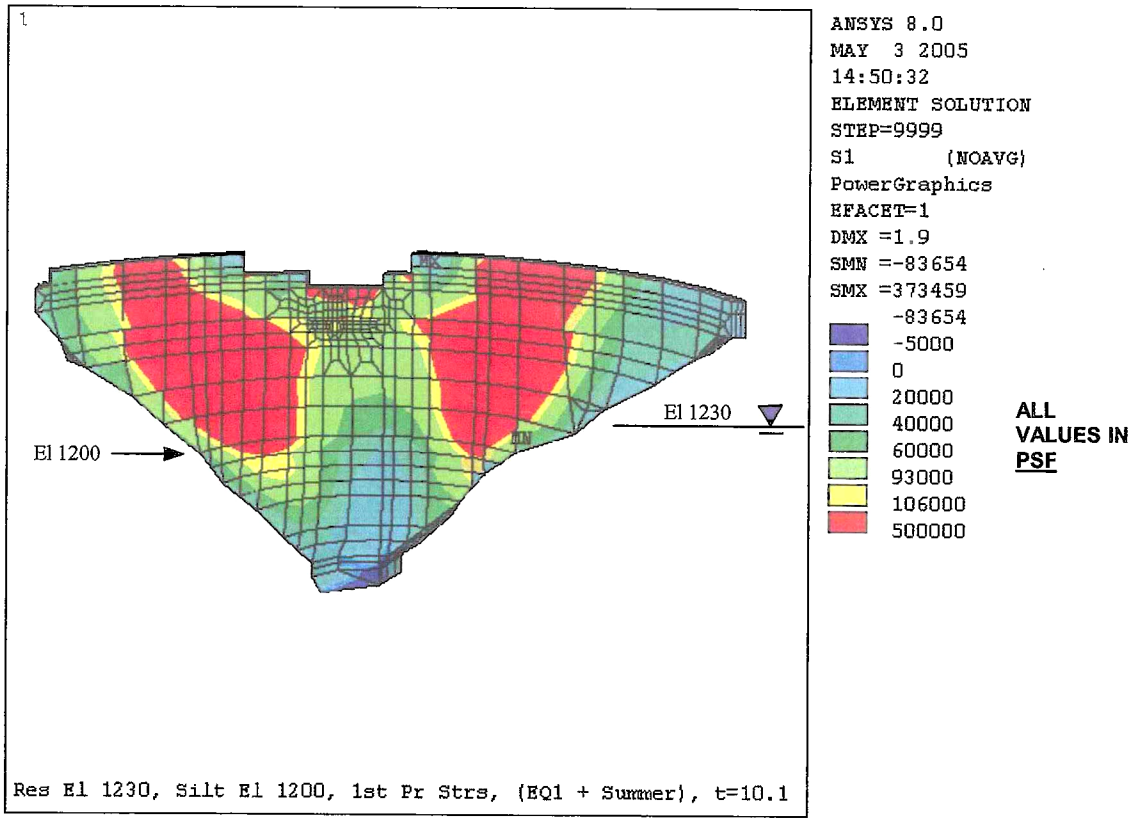


Downstream Face – Intake Riser Modification Model

Figure 3-33

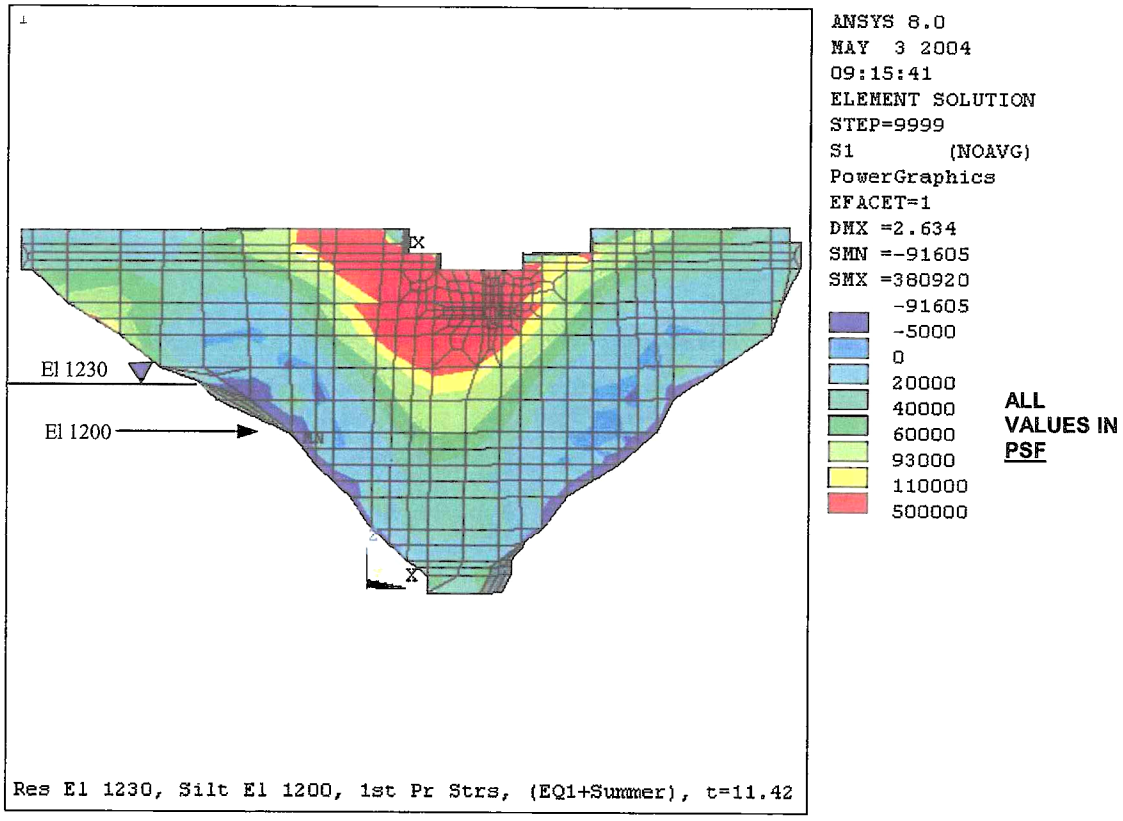


Upstream Face

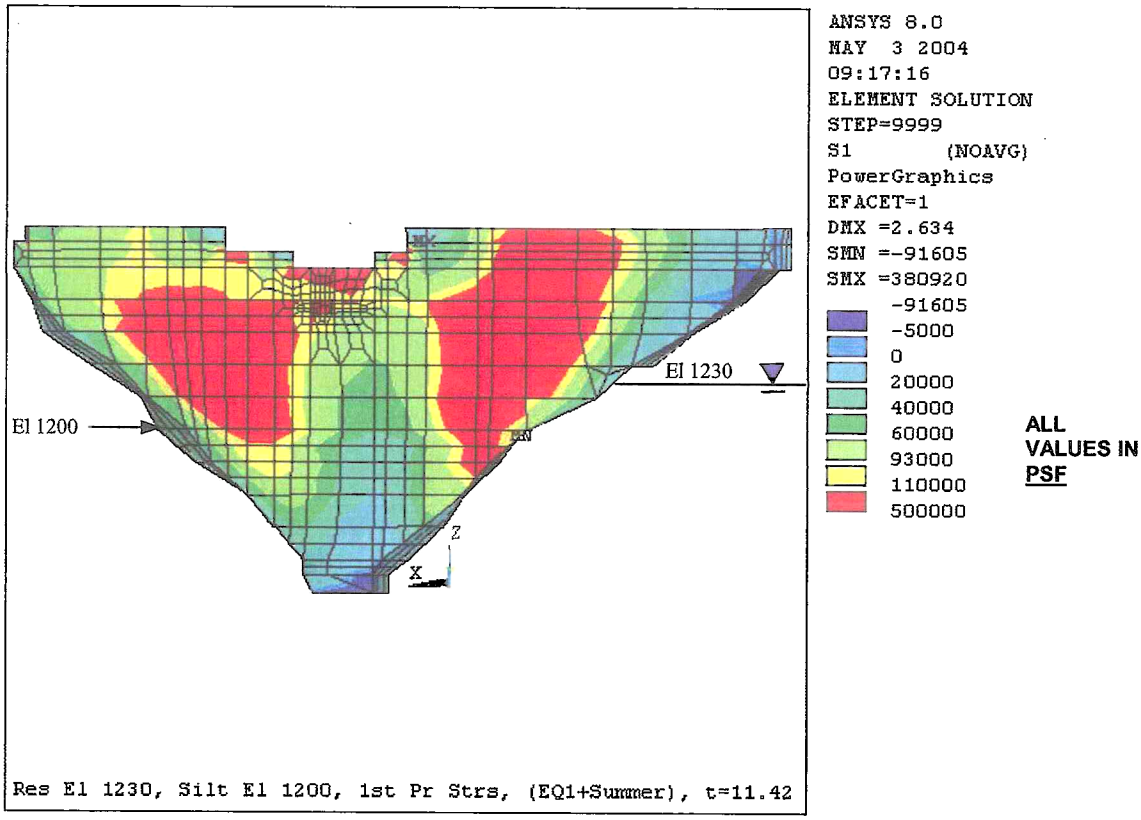


Downstream Face

Figure 3-34

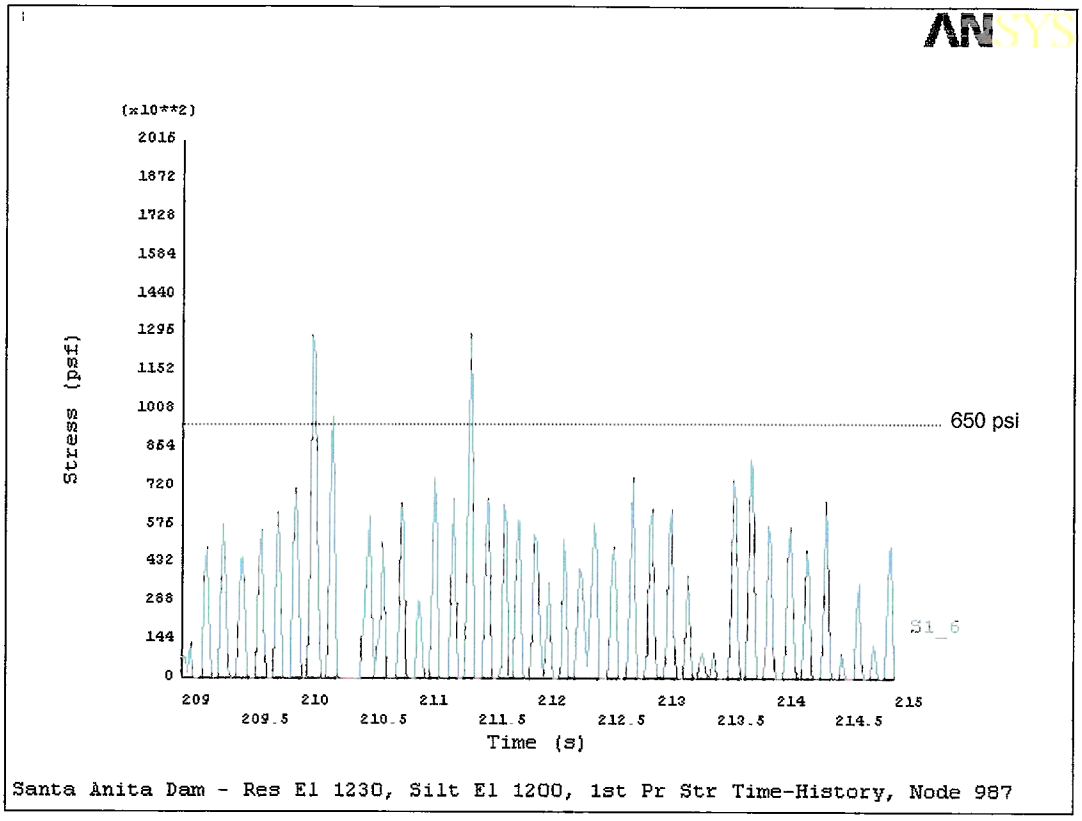


Upstream Face

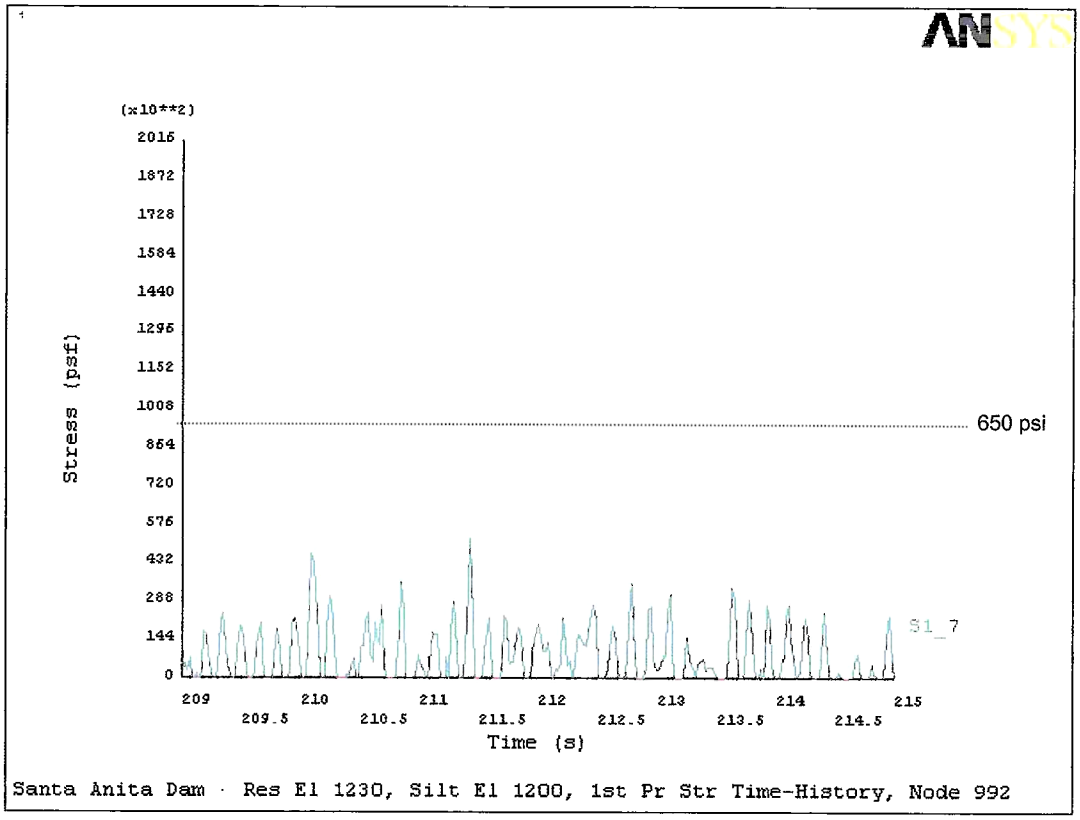


Downstream Face

Figure 3-35

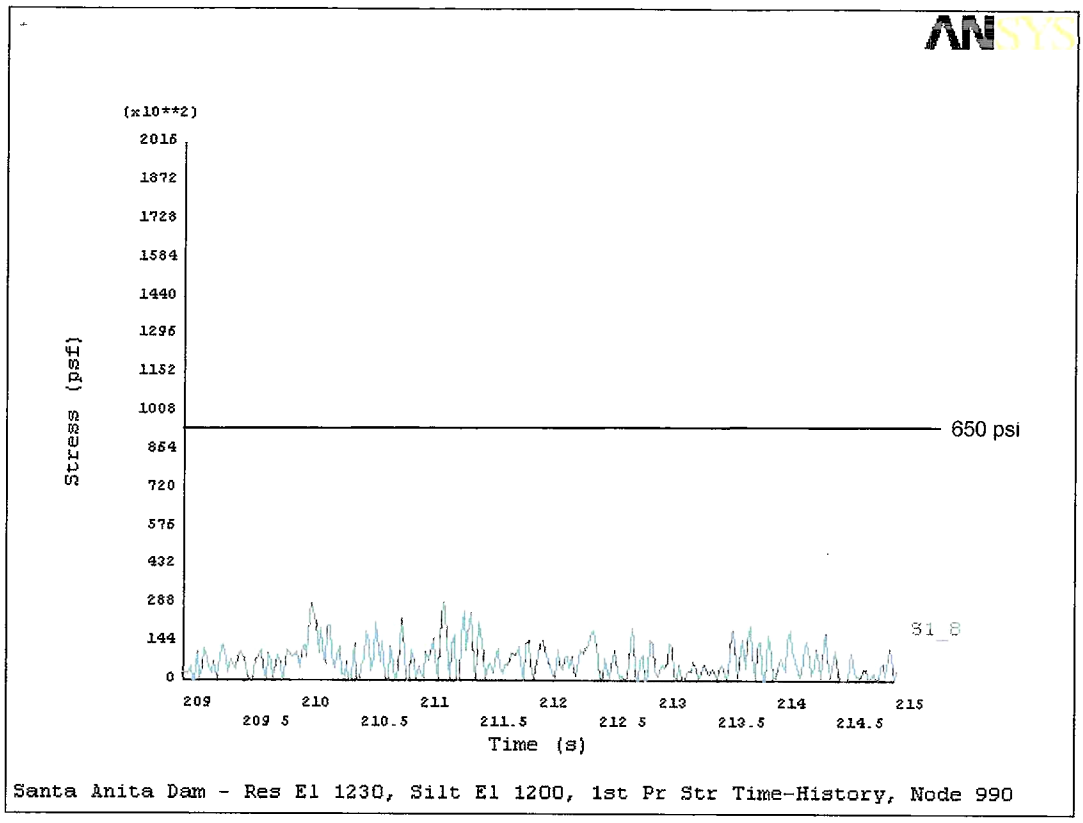


Downstream Face – Node 987

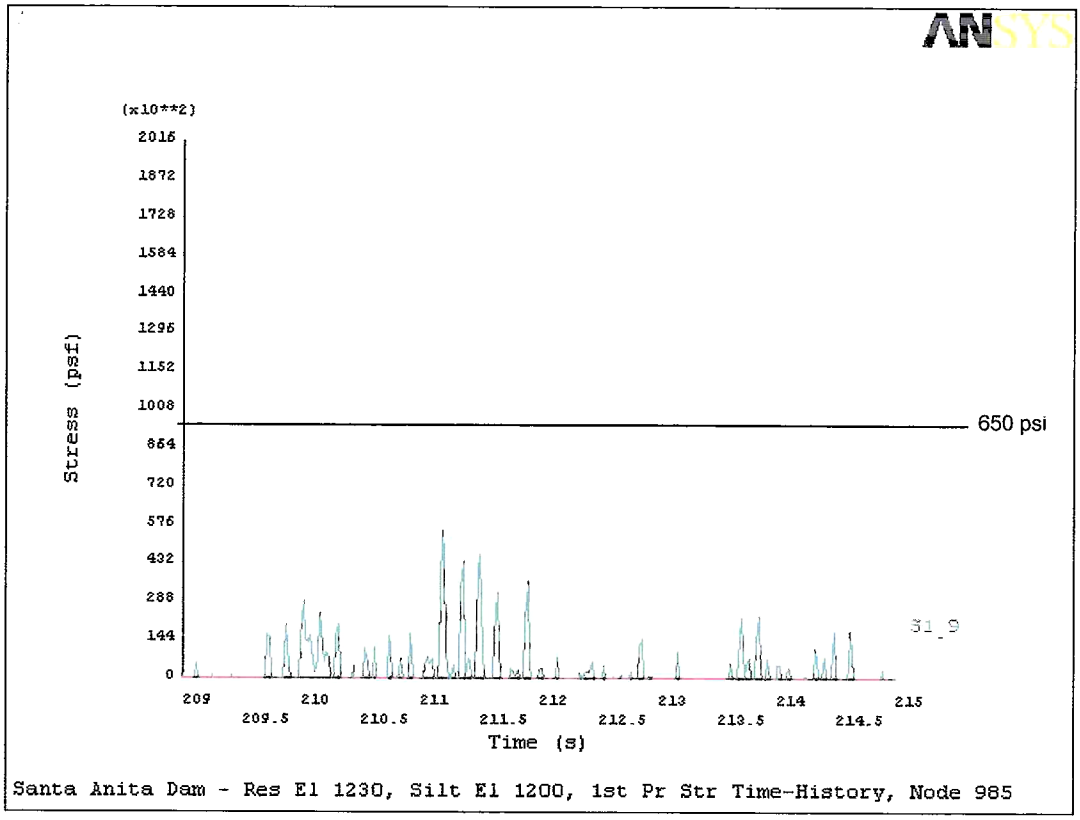


Node 992, 13' from Downstream Face Node 987

Figure 3-36

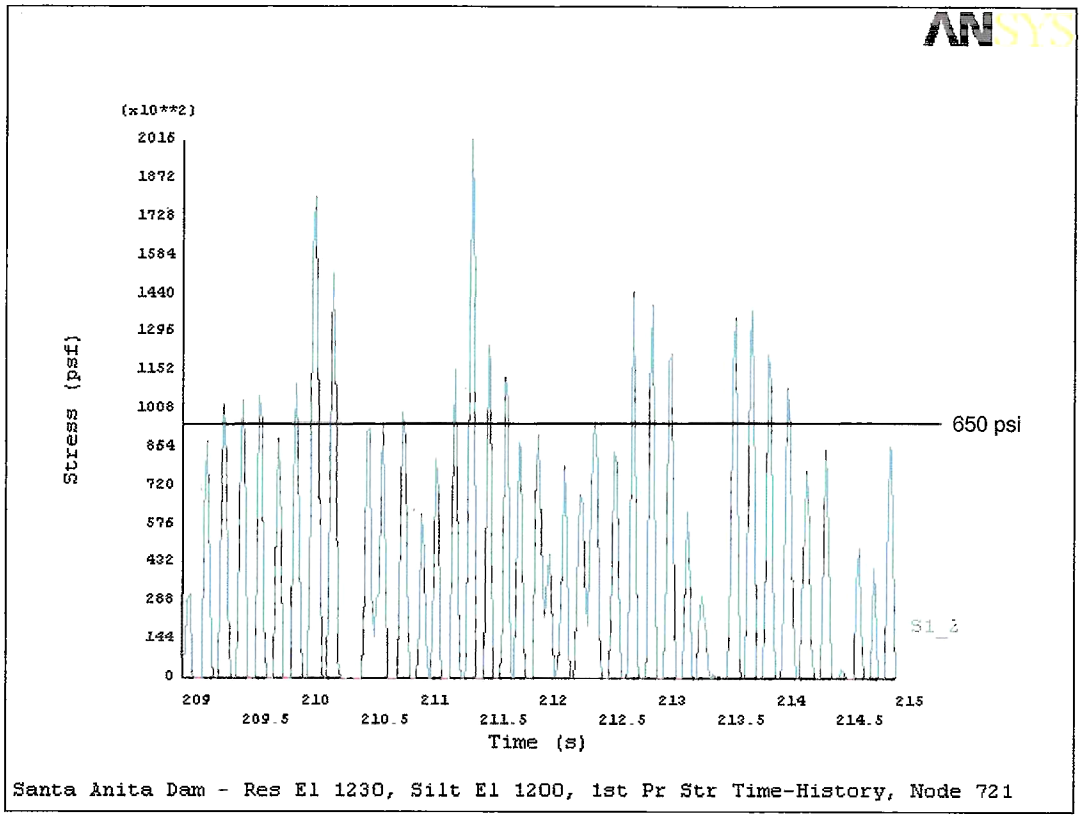


Node 990, 22' from Downstream Face Node 987

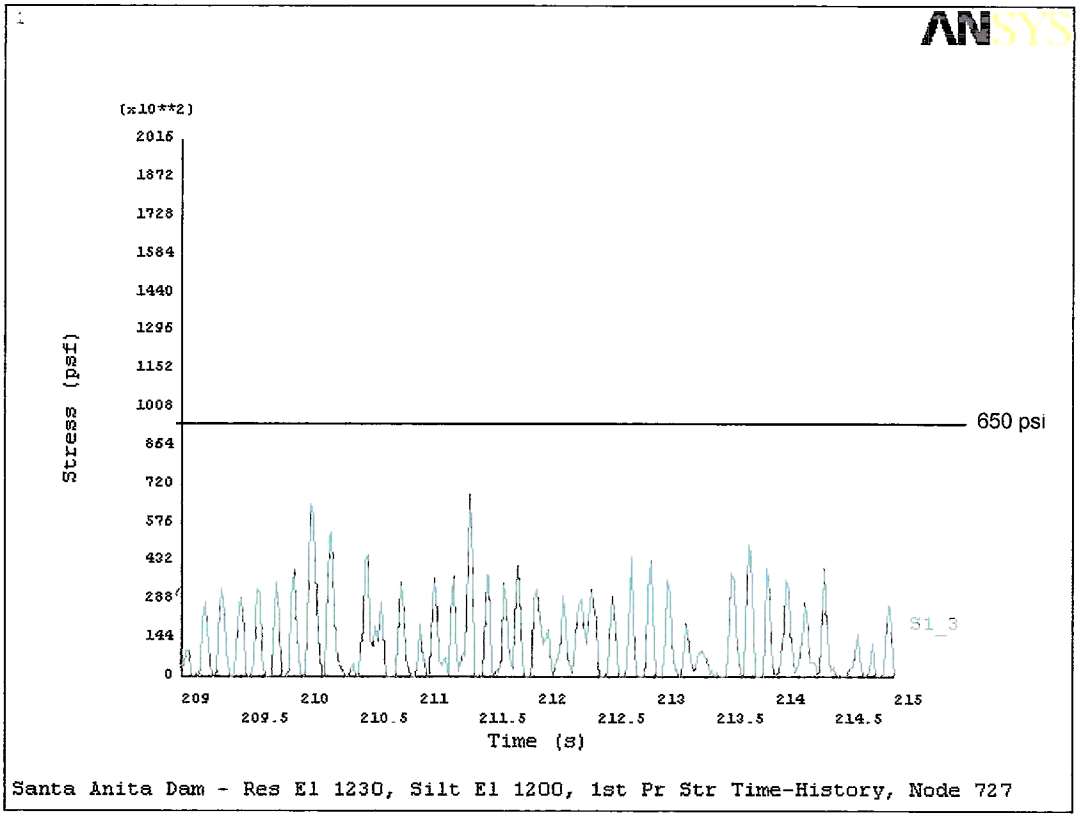


Node 985 on the Downstream Face, 33' from Node 987

Figure 3-37

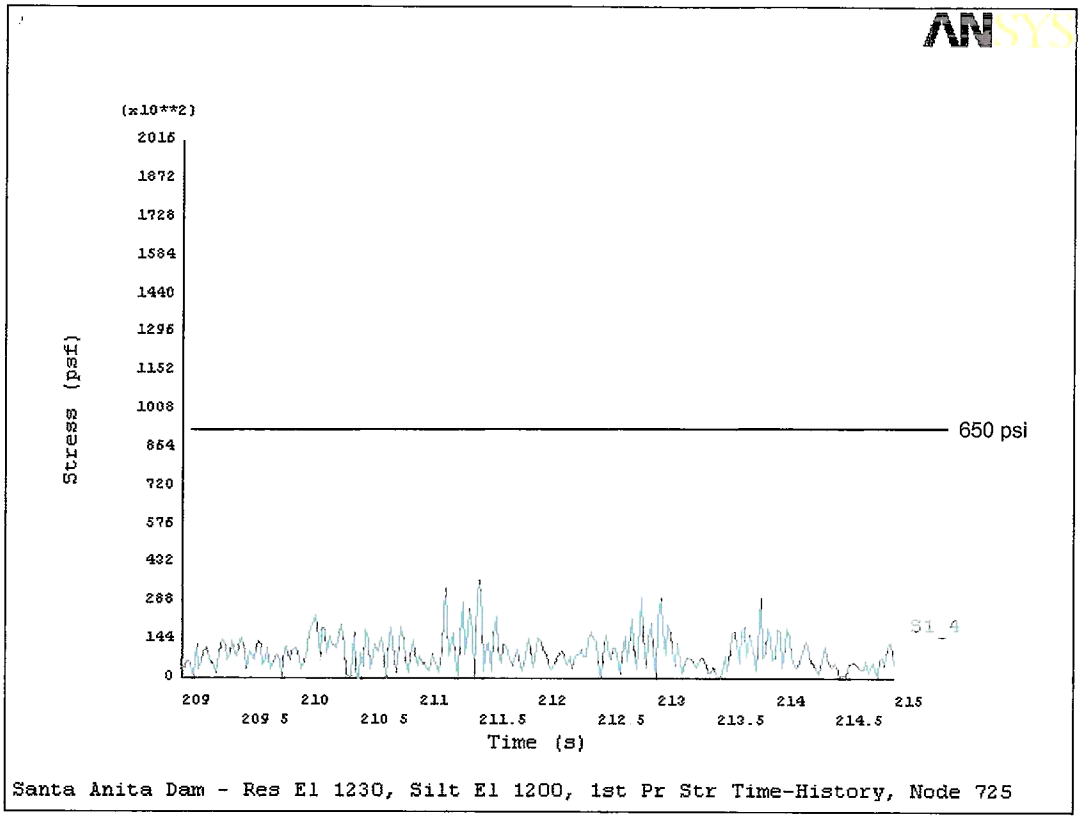


Downstream Face

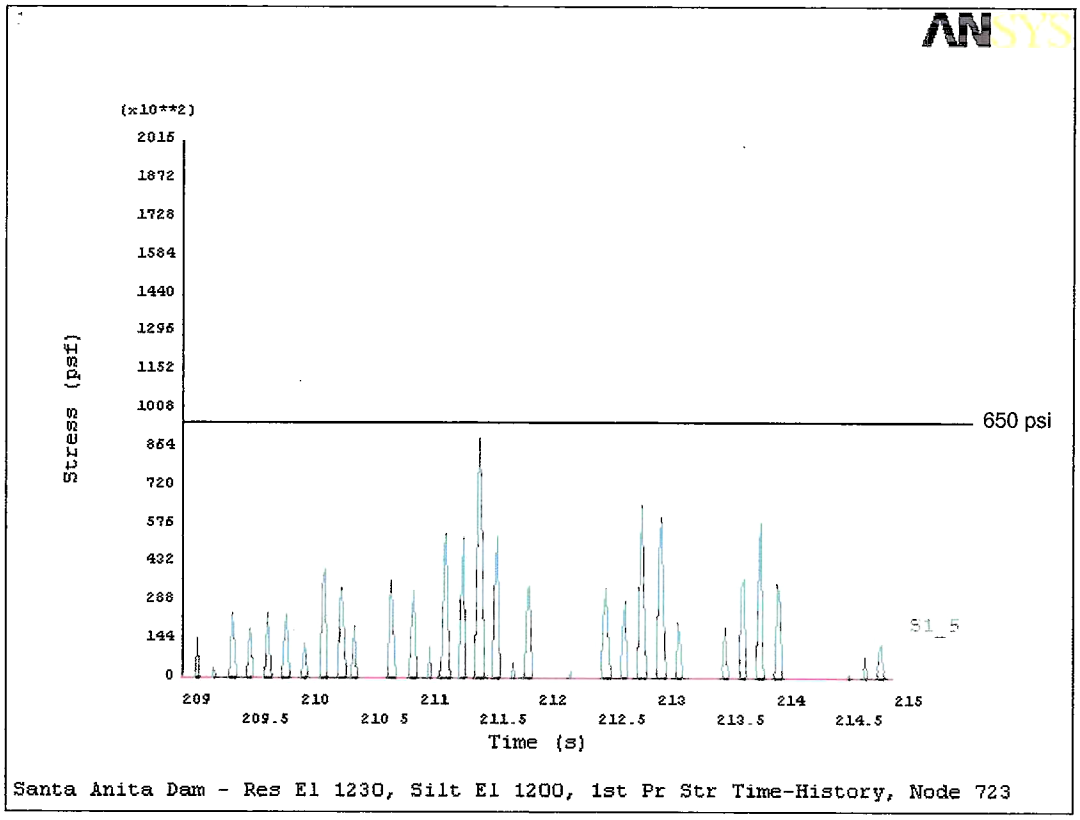


Node 727, 12' from Downstream Face Node 721

Figure 3-38

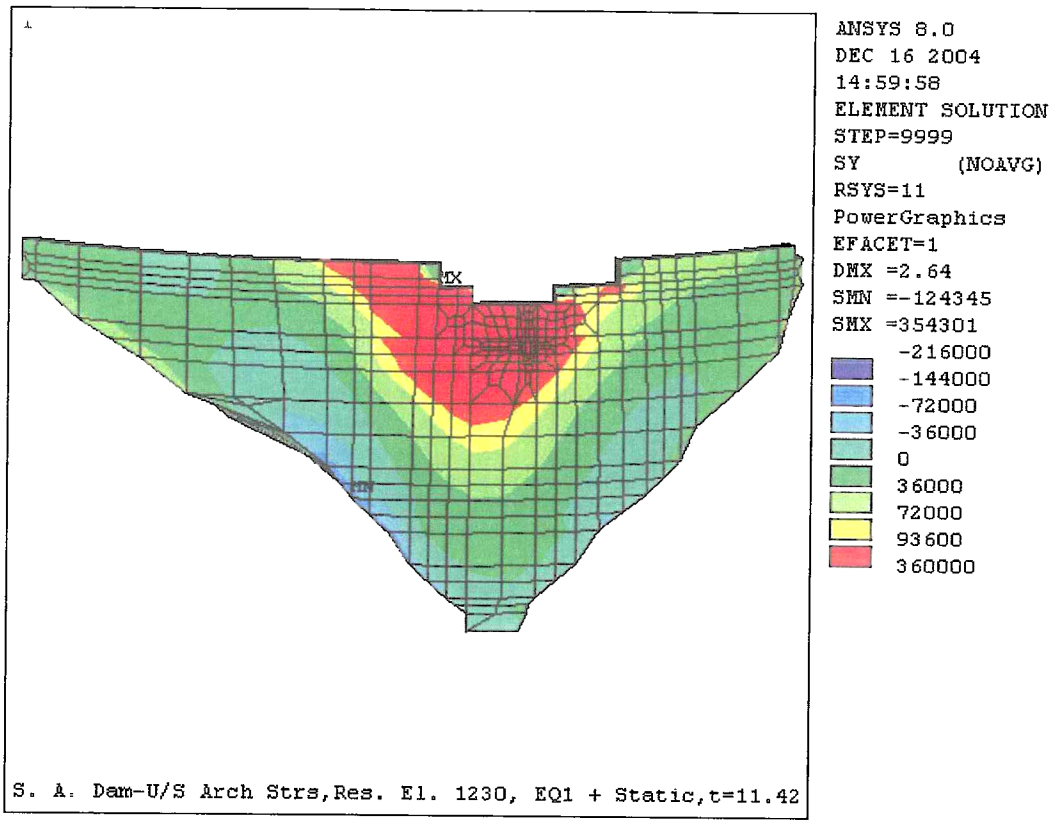


Node 725, 24' from Downstream Face Node 721



Node 723 on the Downstream Face, 36' from Node 721

Figure 3-39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-36000
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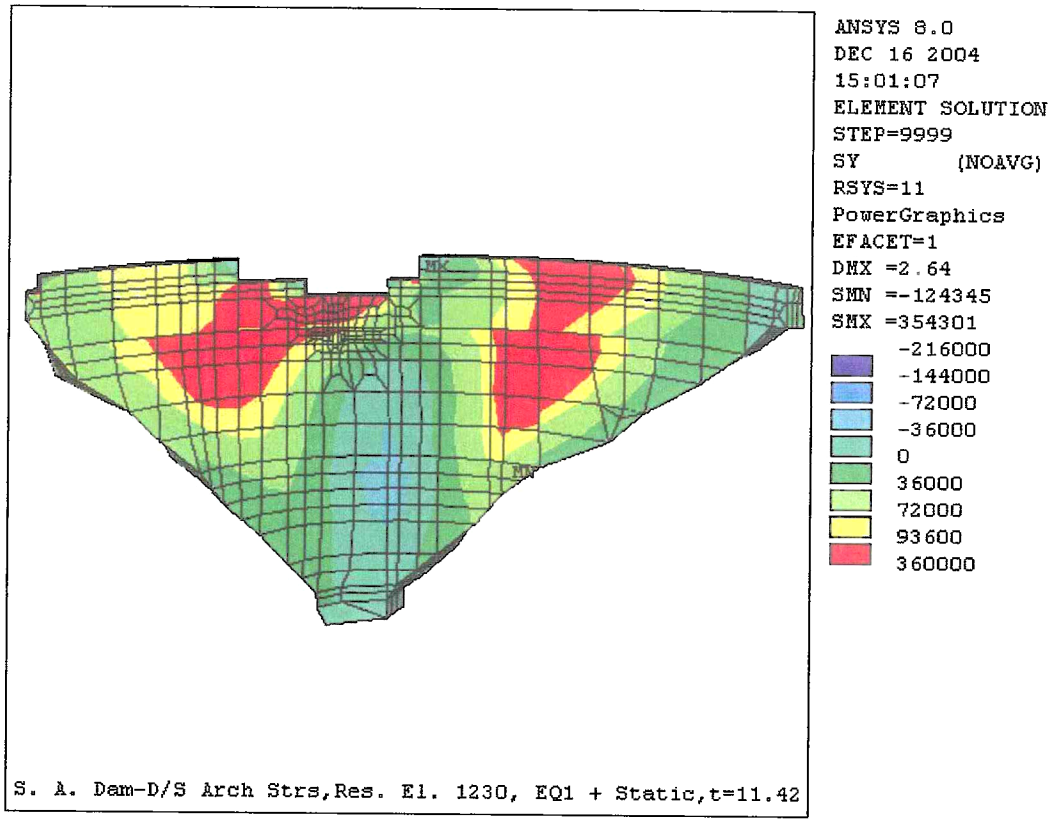
S. A. Dam-U/S Arch Strs,Res. El. 1230, EQ1 + Static,t=11.42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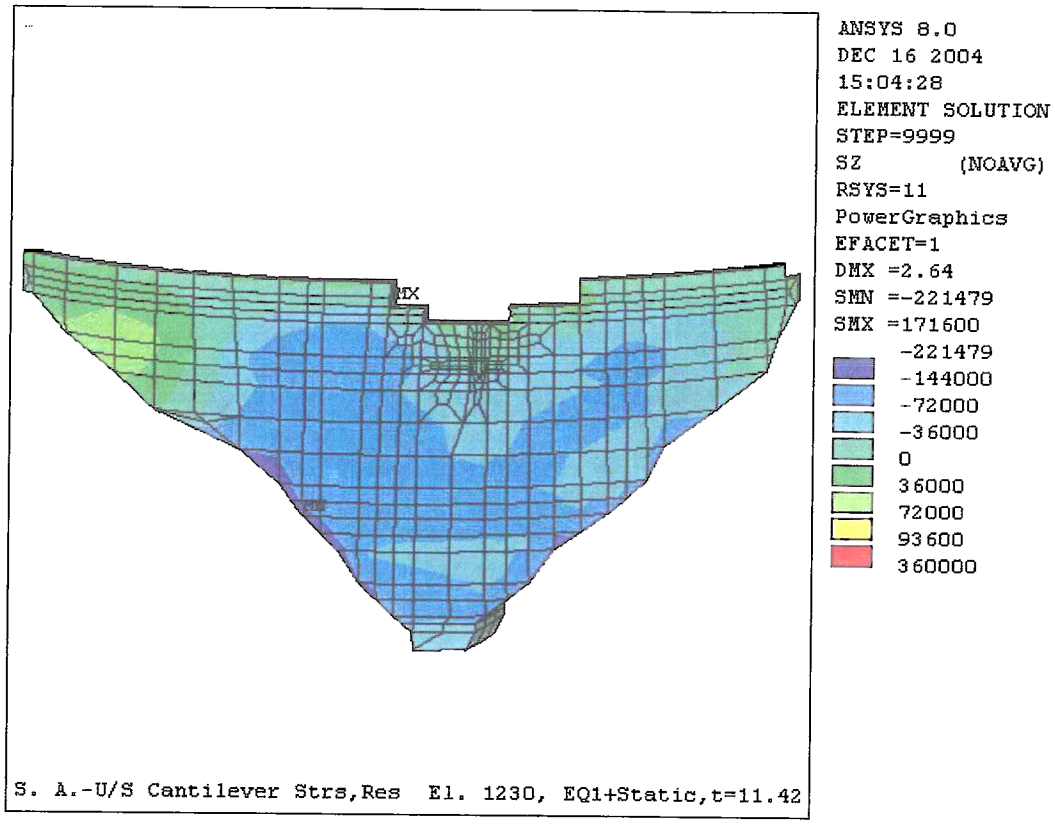
S. A. Dam-D/S Arch Strs,Res. El. 1230, EQ1 + Static,t=11.42

Figure 3-40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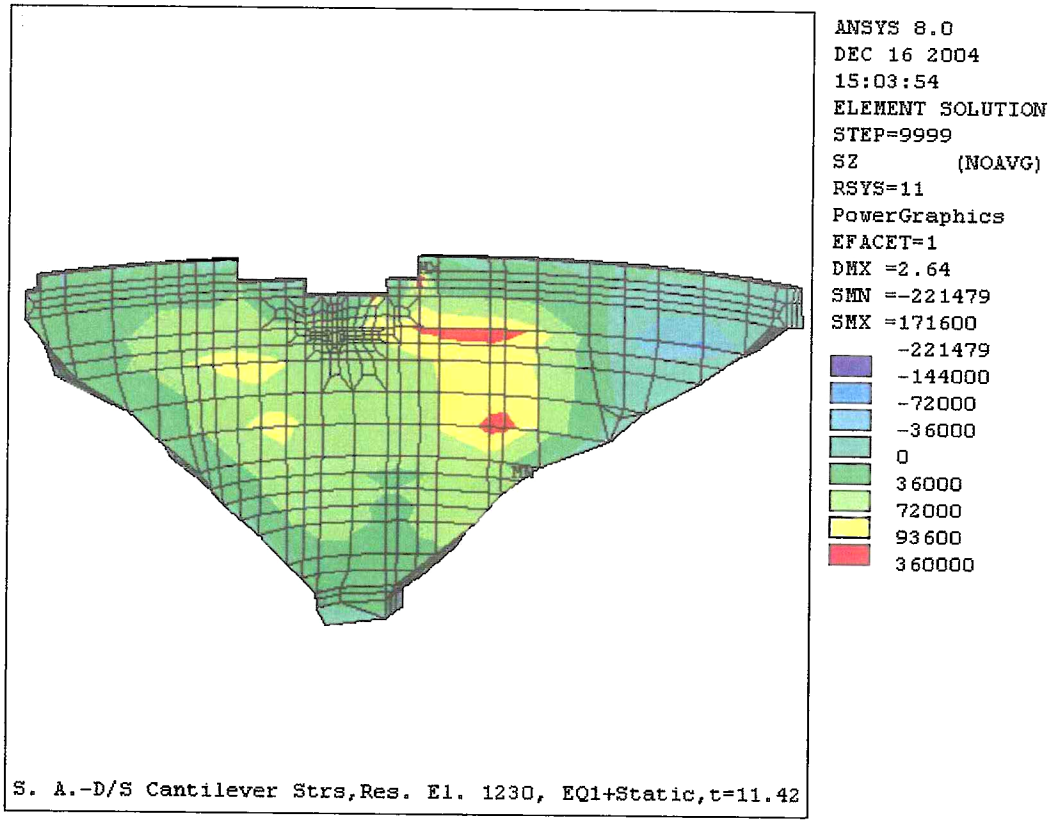
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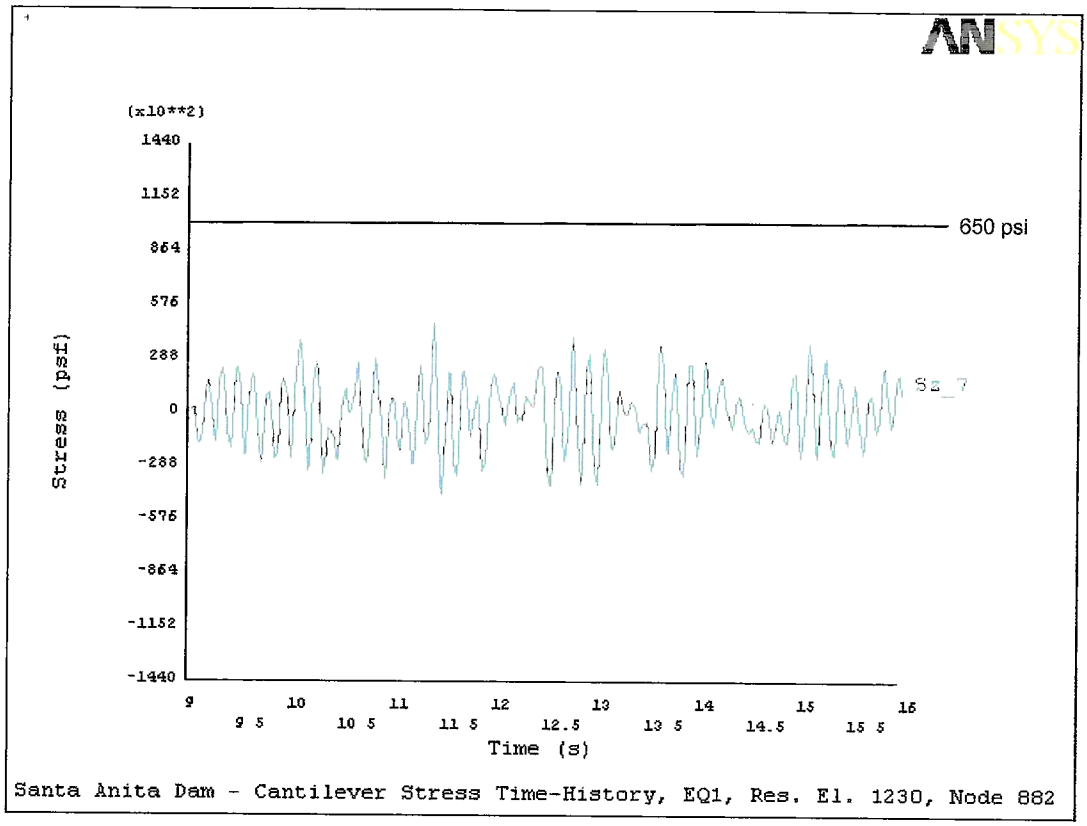
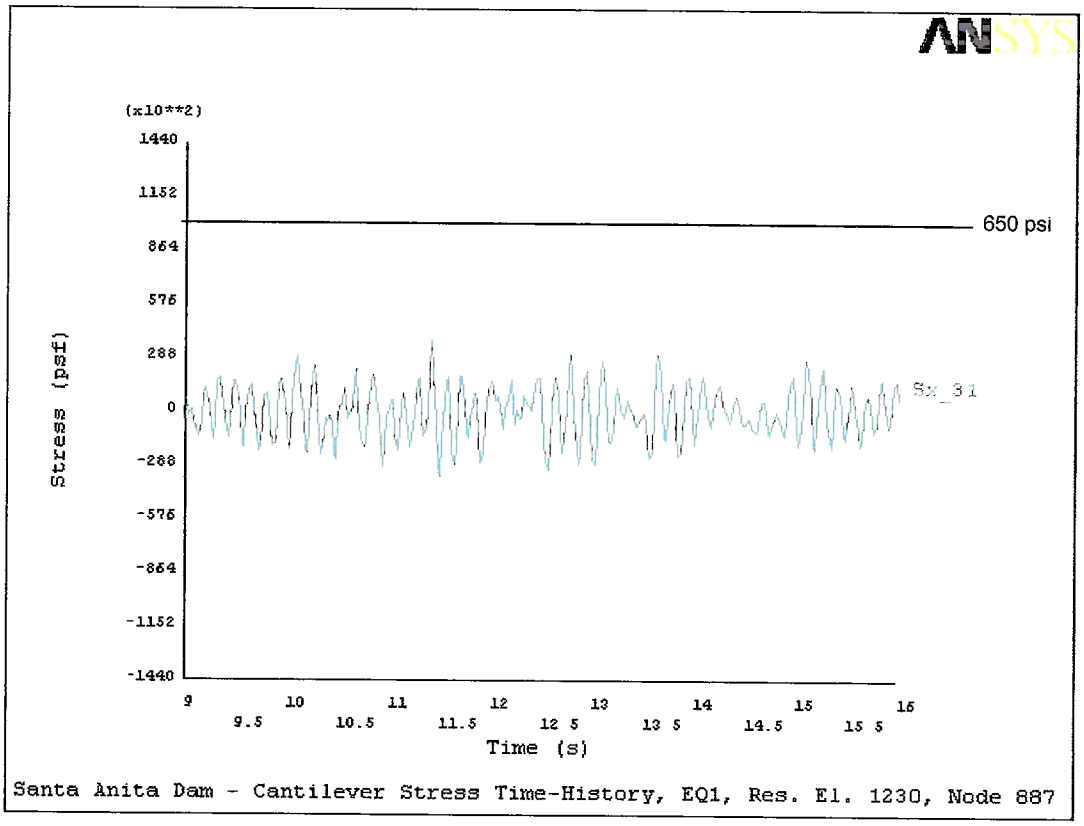
Figure 3-41

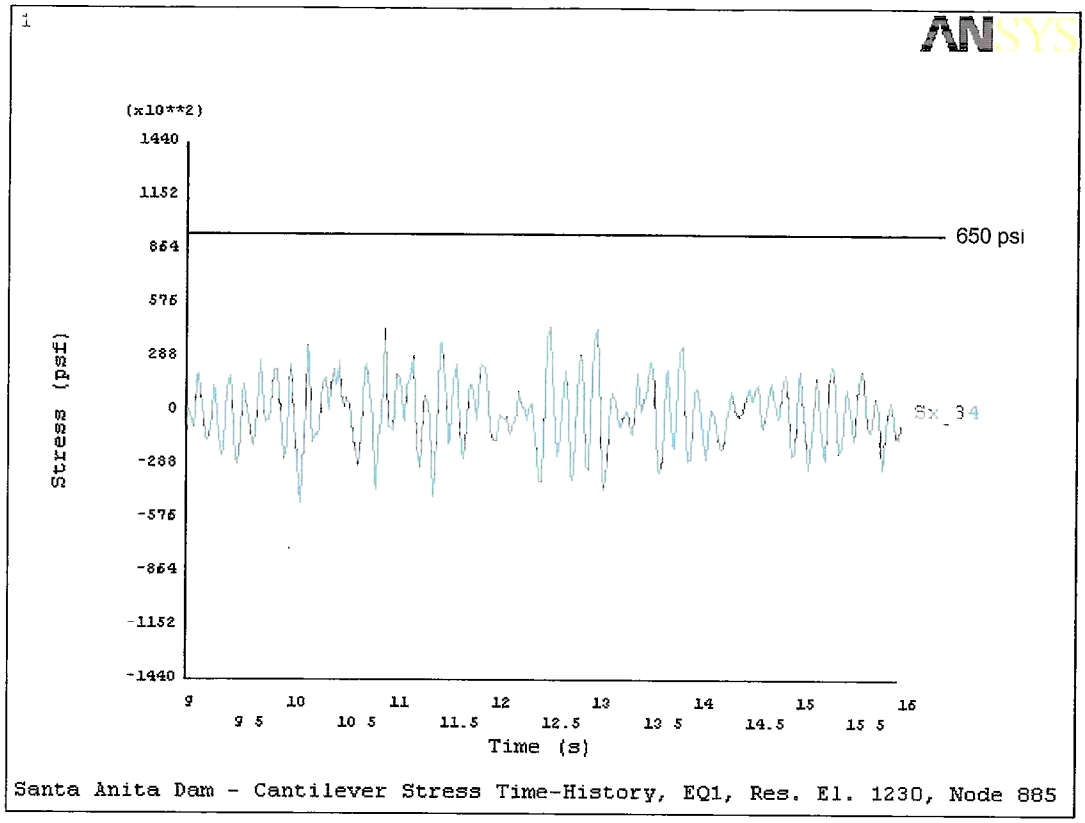


Node 882 Downstream Face

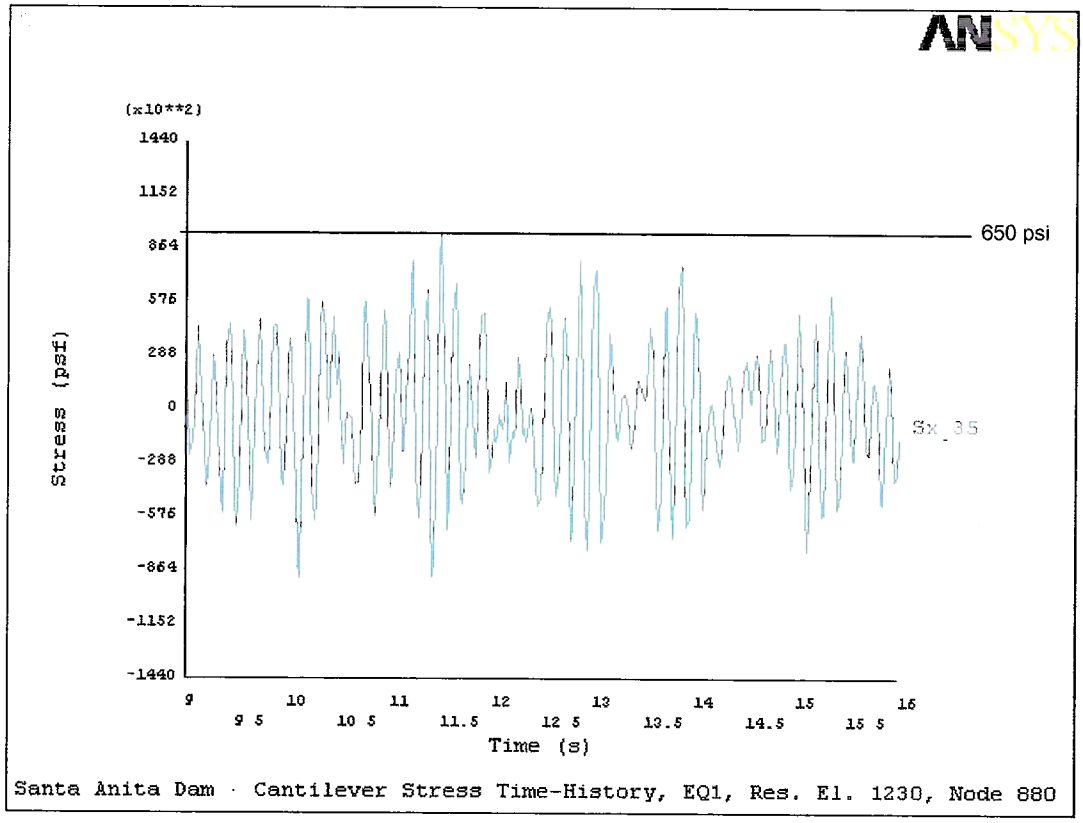


Node 887, 15' from Downstream Face Node 882

Figure 3-42

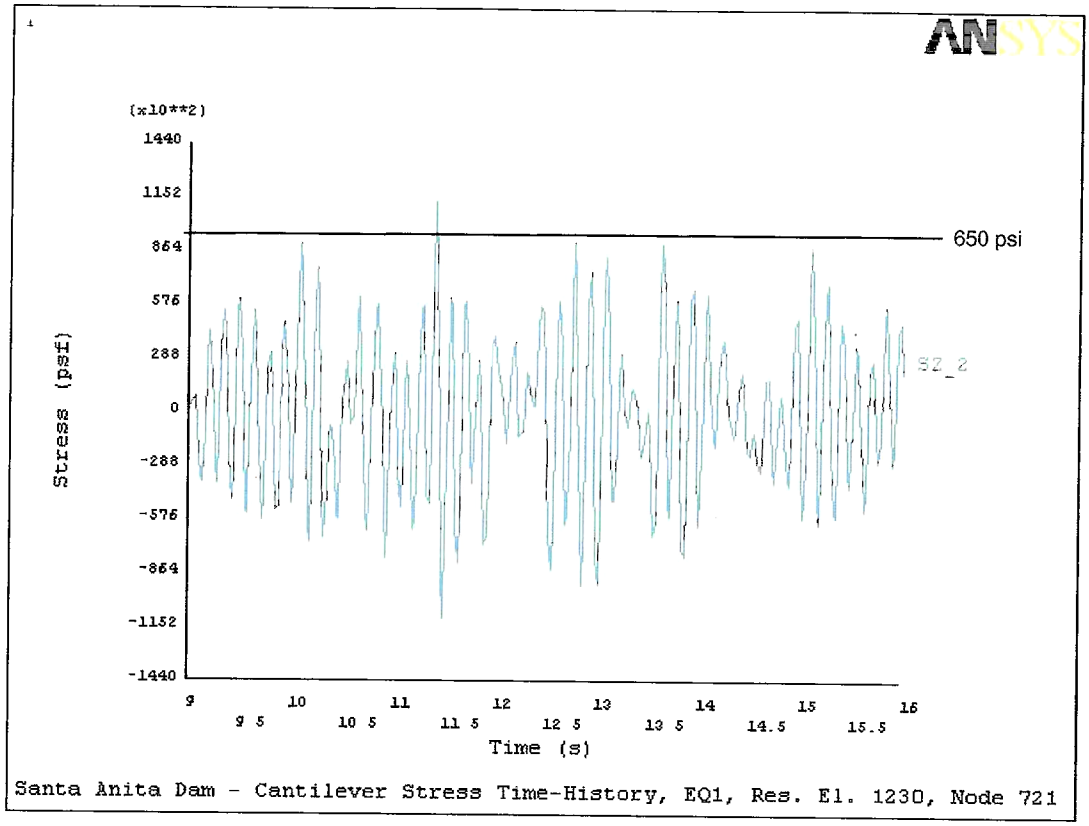


Node 885, 30' from Downstream Face Node 882

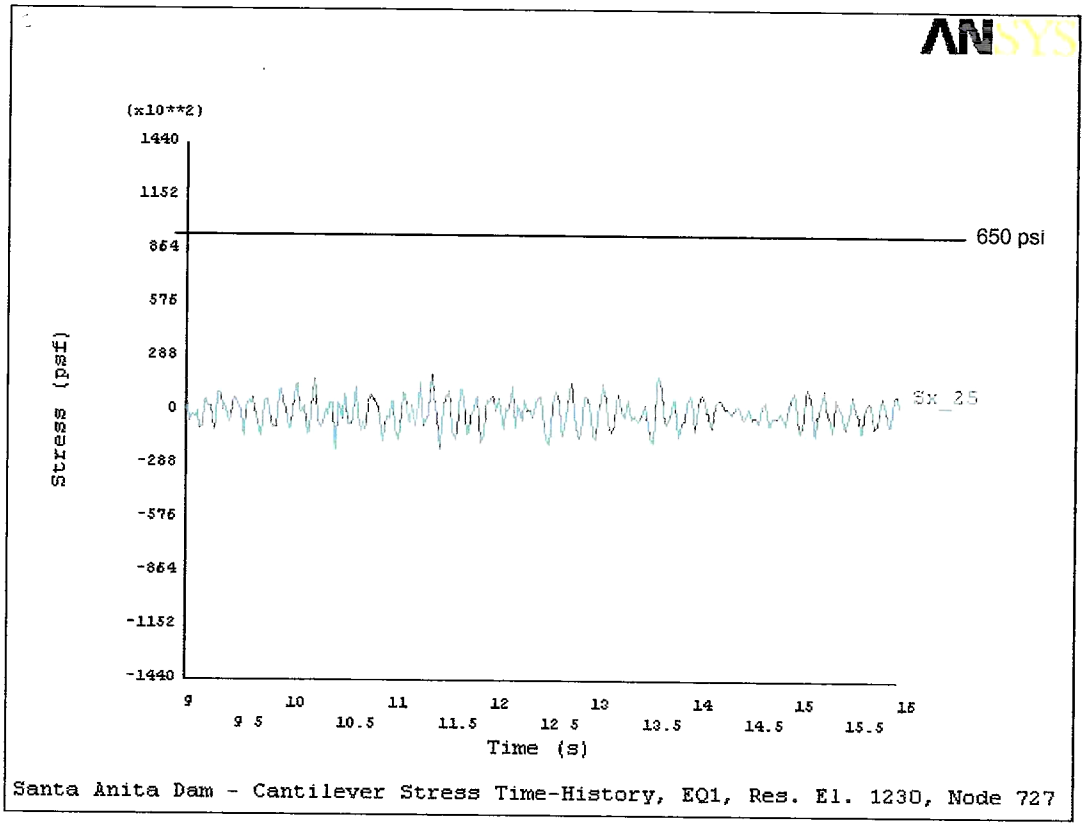


Node 880 on the Upstream Face, 45' from Node 882

Figure 3-43

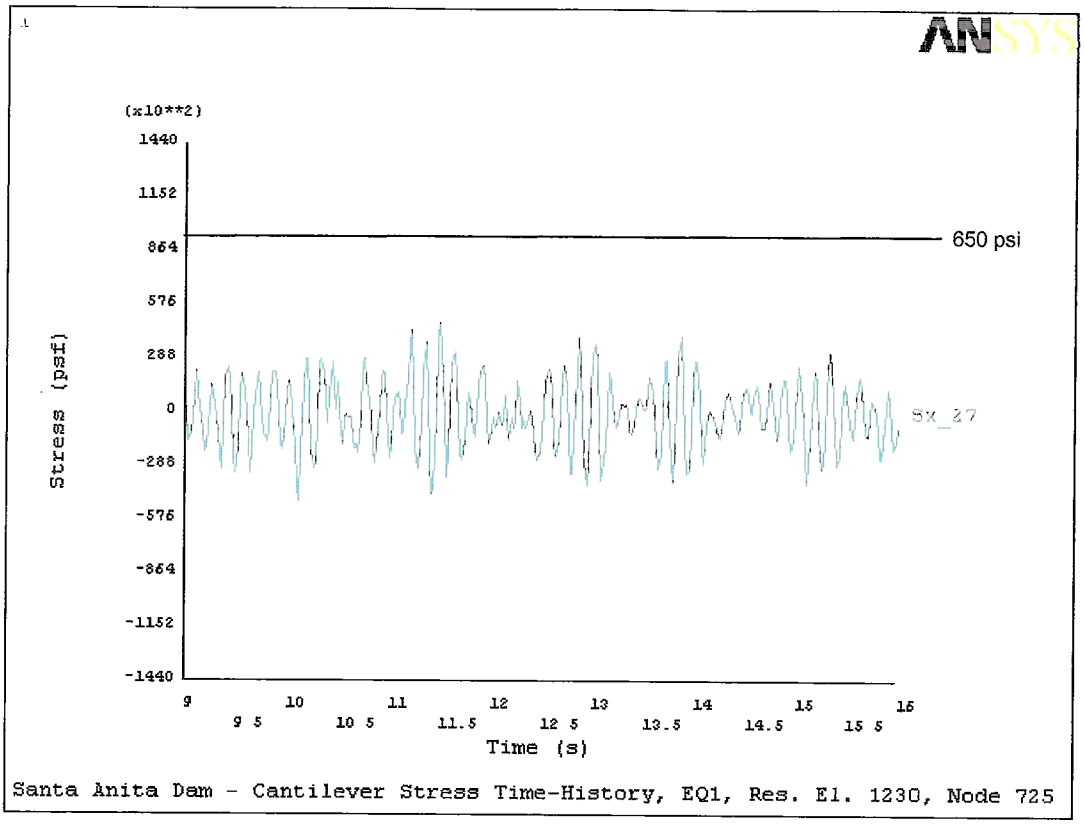


Downstream Face – Node 721

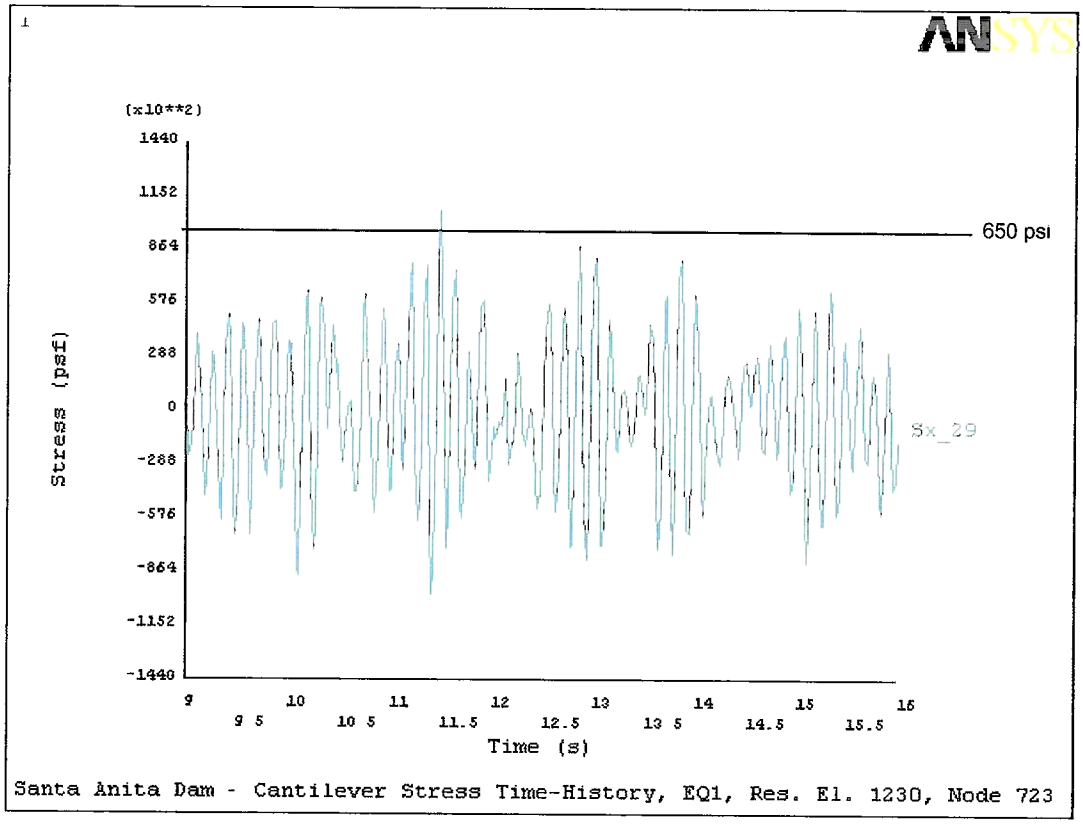


Node 727, 12' from Downstream Face Node 721

Figure 3-44

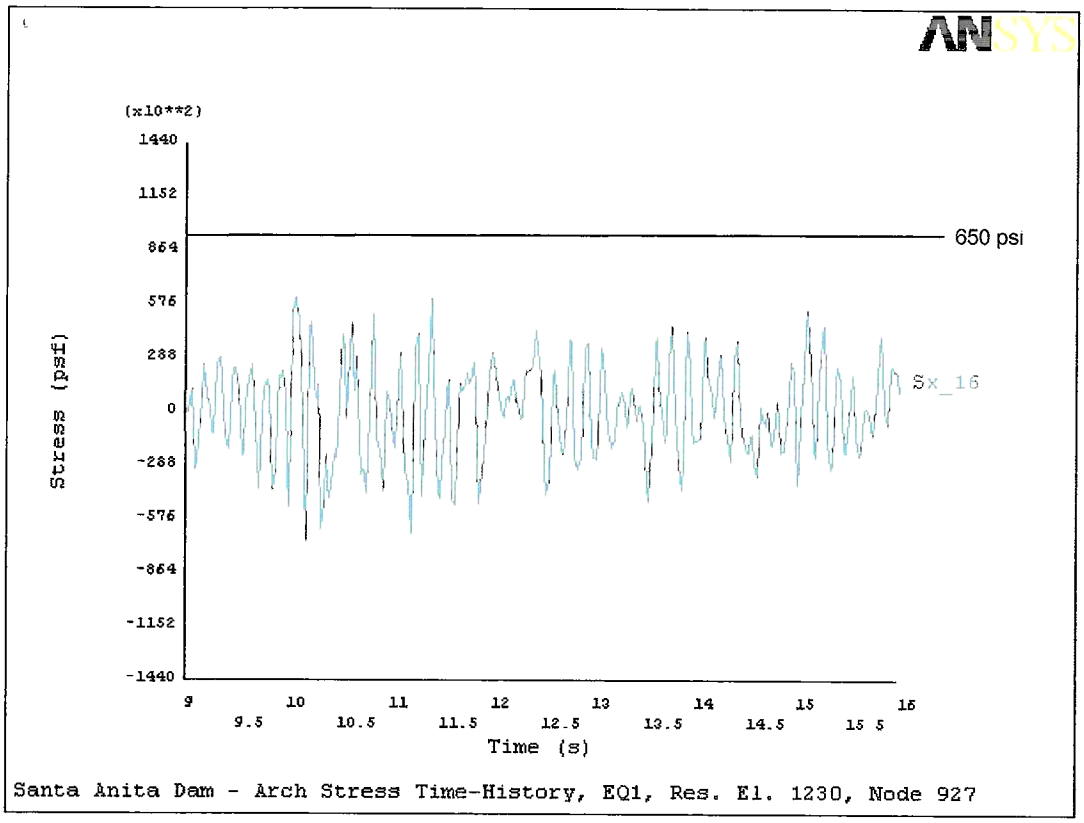


Node 725, 24' from Downstream Face Node 721

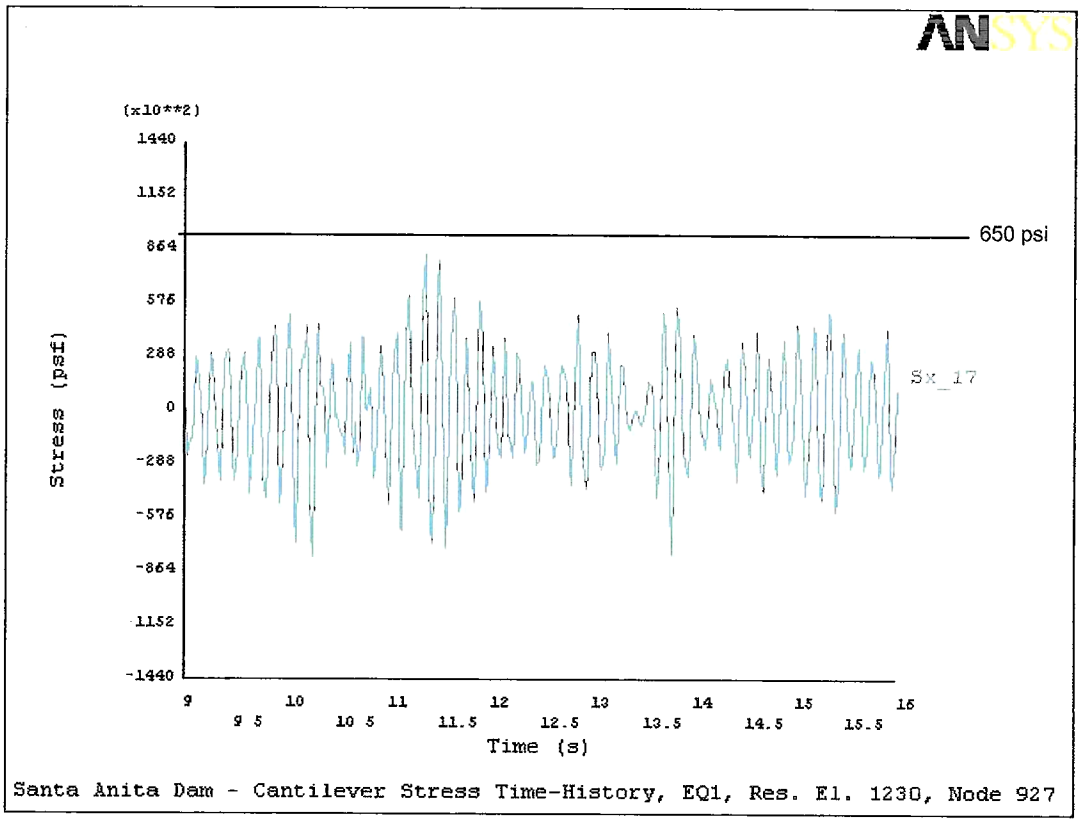


Node 723 on the Upstream Face, 36' from Node 721

Figure 3-45

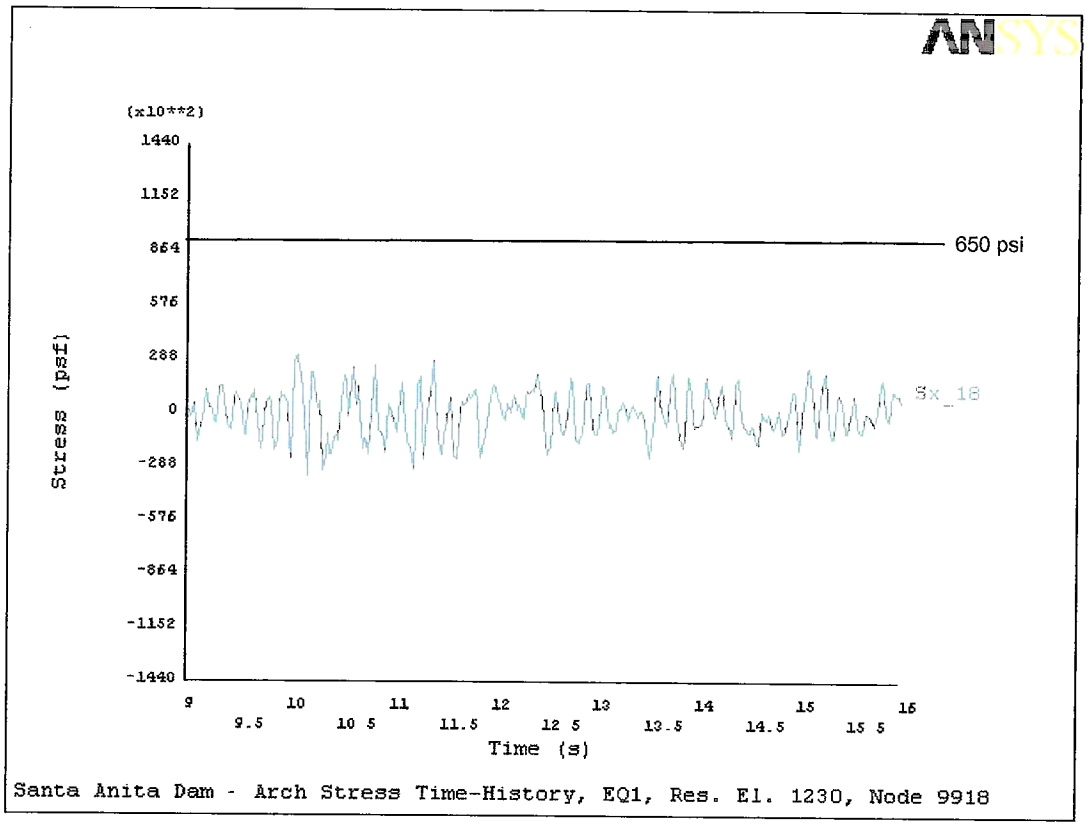


Upstream Face – Node 927

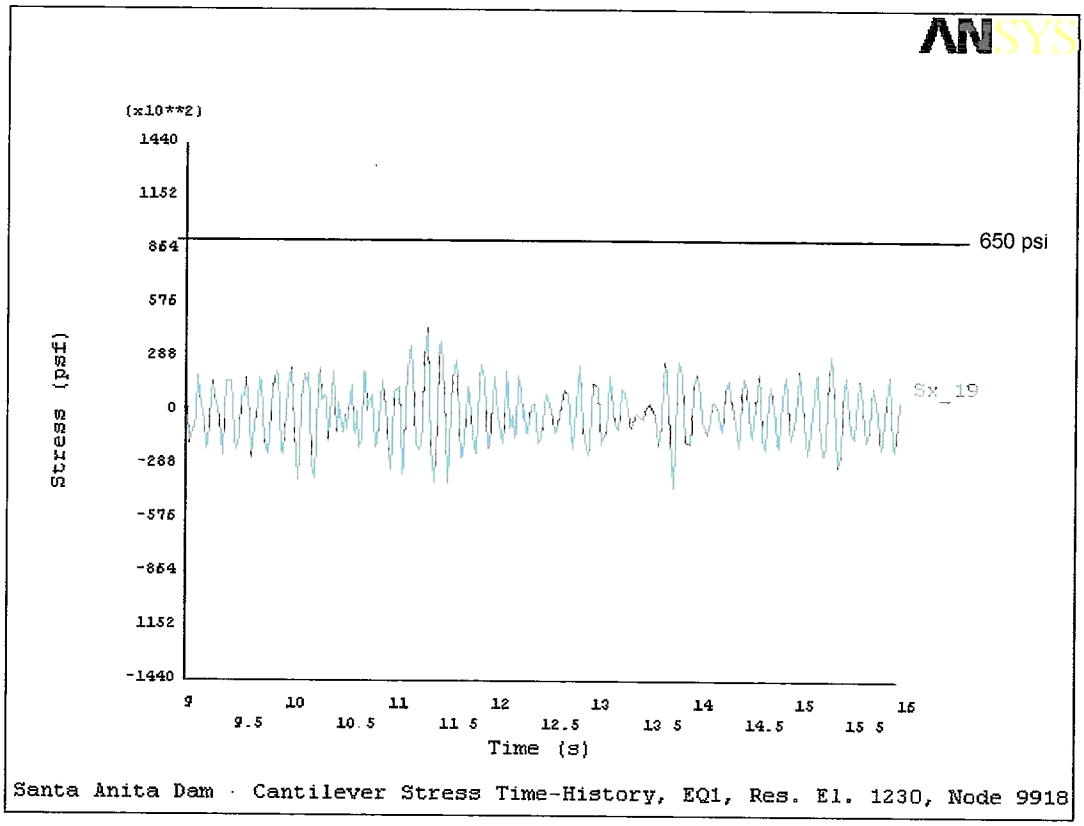


Upstream Face – Node 927

Figure 3-46

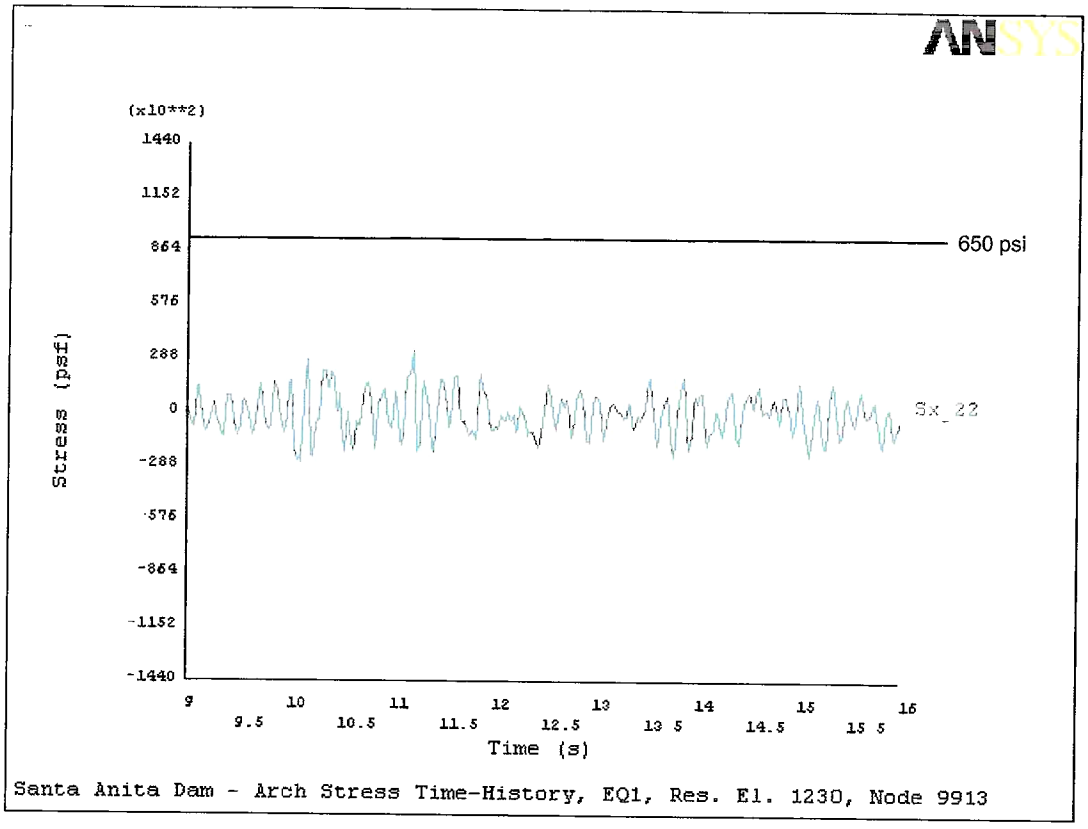


Node 9918, 10' from Upstream Face Node 927

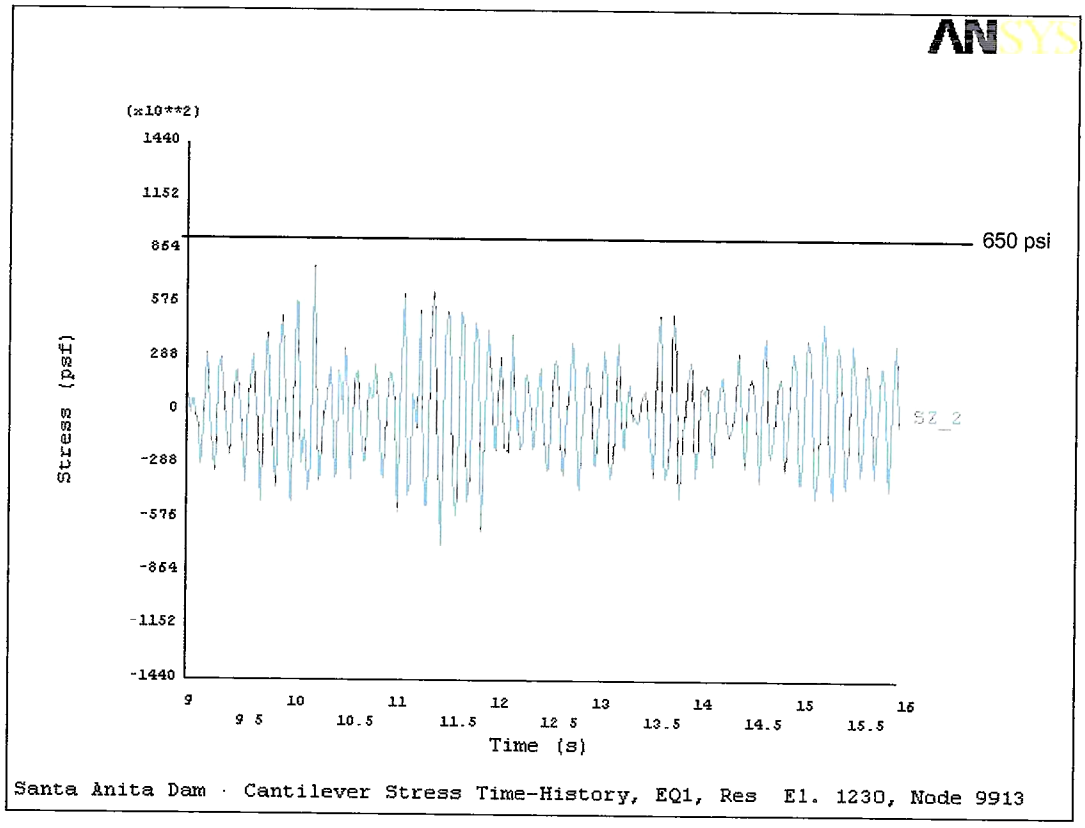


Node 9918, 10' from Upstream Face Node 927

Figure 3-47

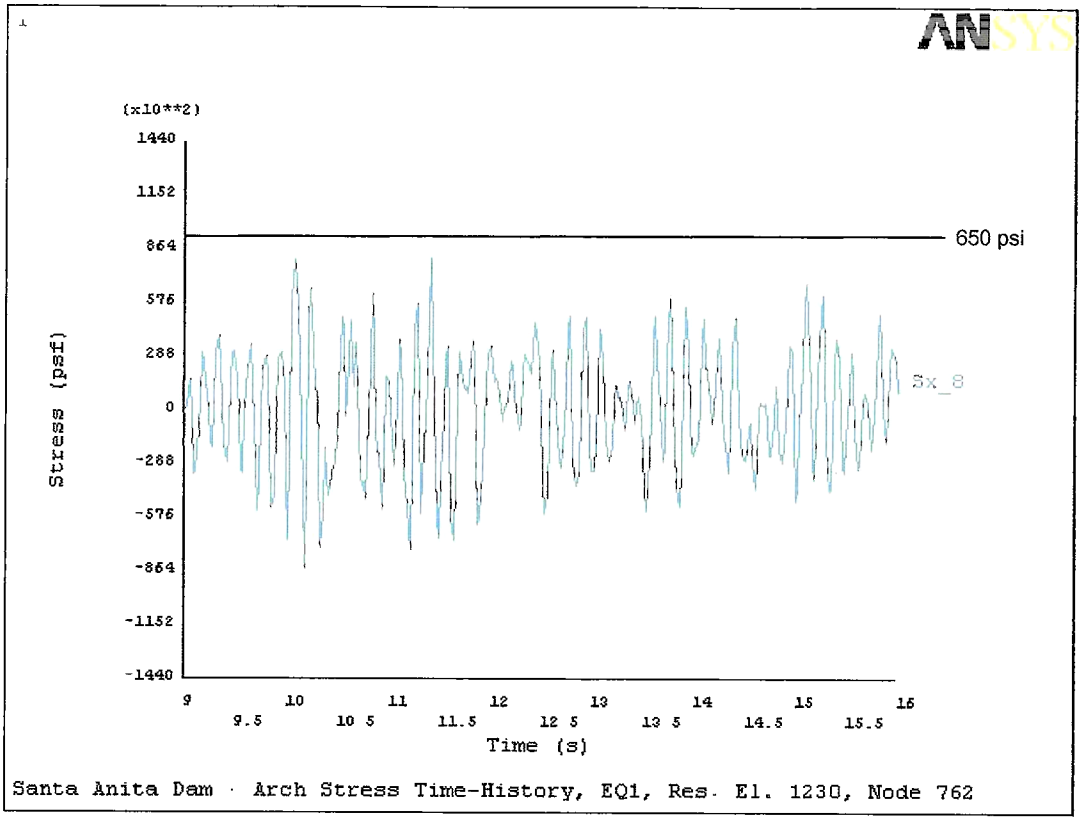


Node 9913 on the Downstream Face, 30' from Node 927

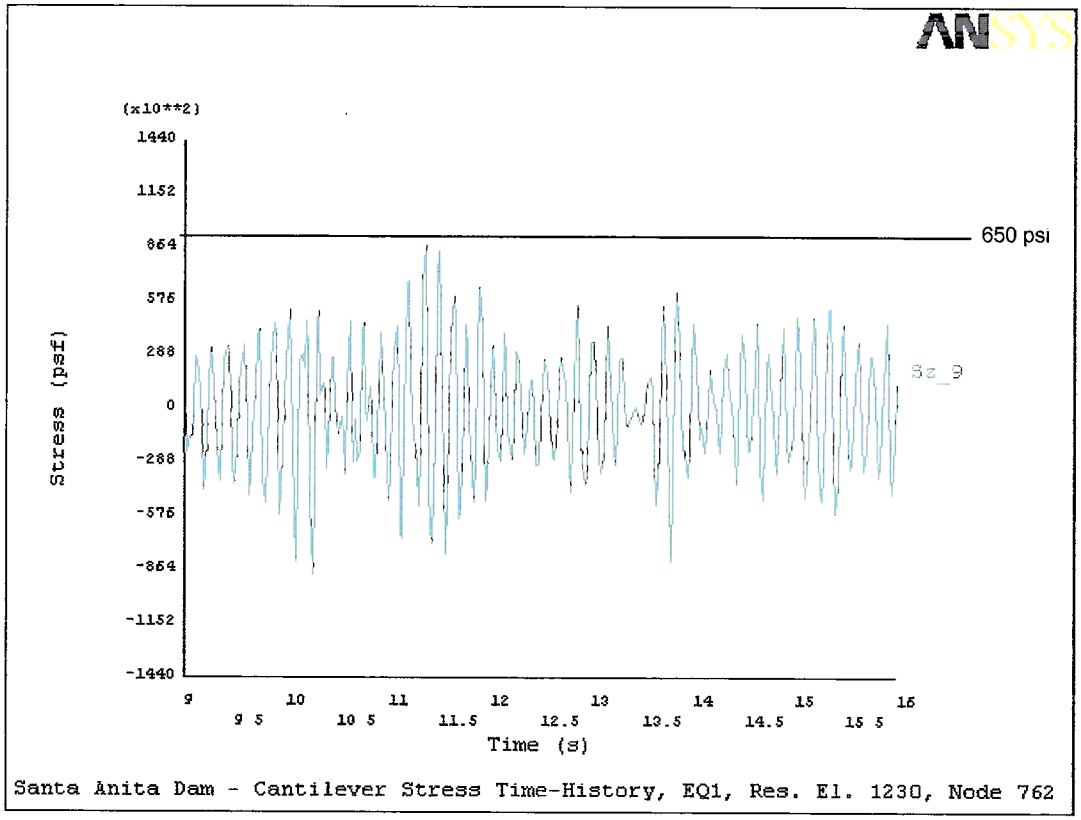


Node 9913 on the Downstream Face, 30' from Node 927

Figure 3-49

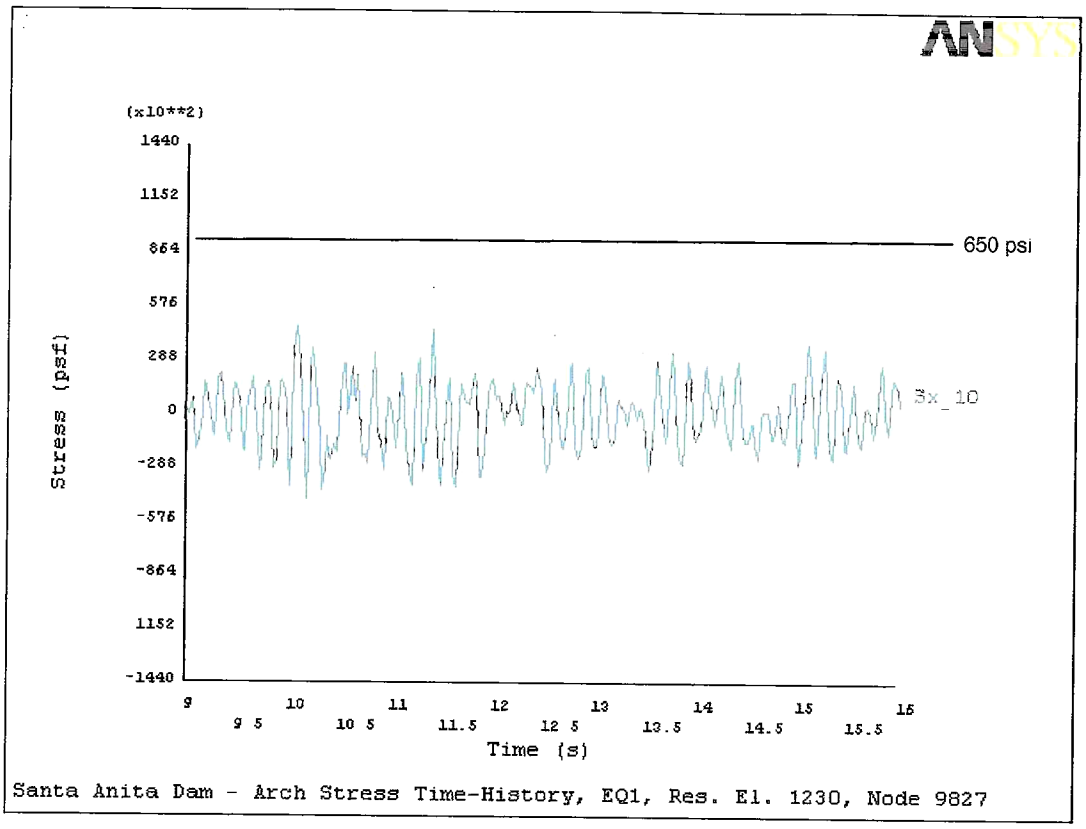


Upstream Face – Node 762

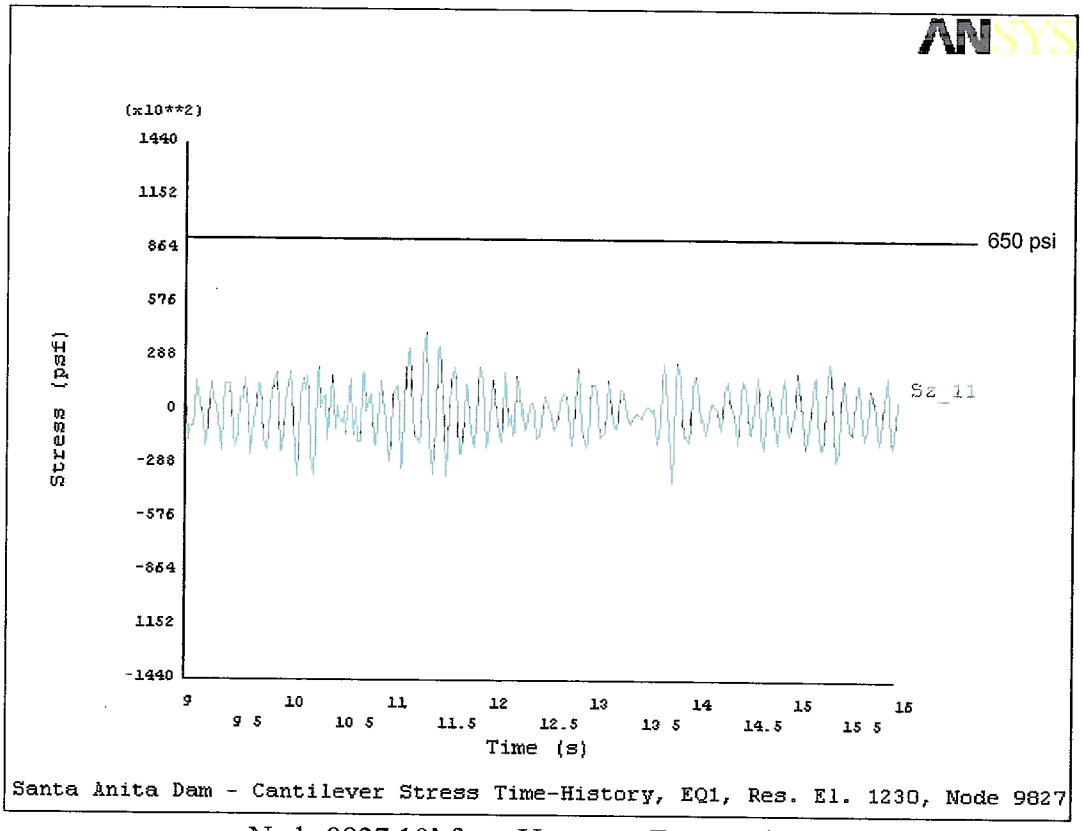


Upstream Face – Node 762

Figure 3-50

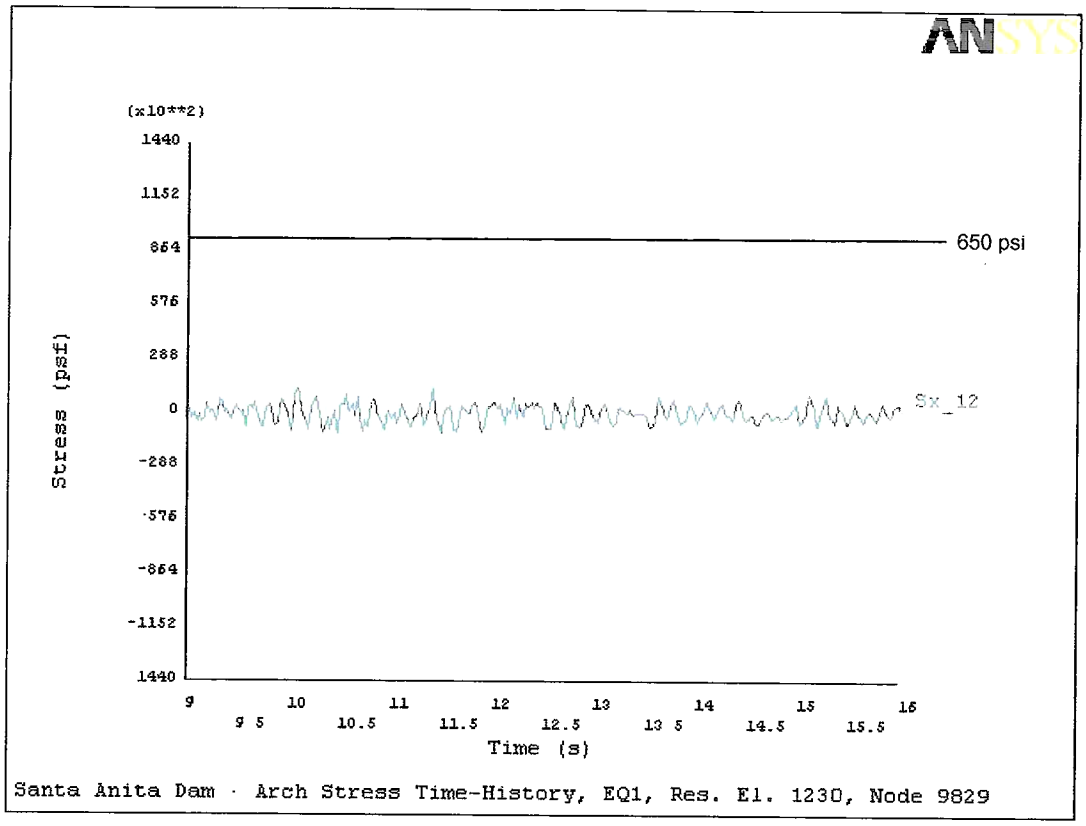


Node 9827 10' from Upstream Face Node 762

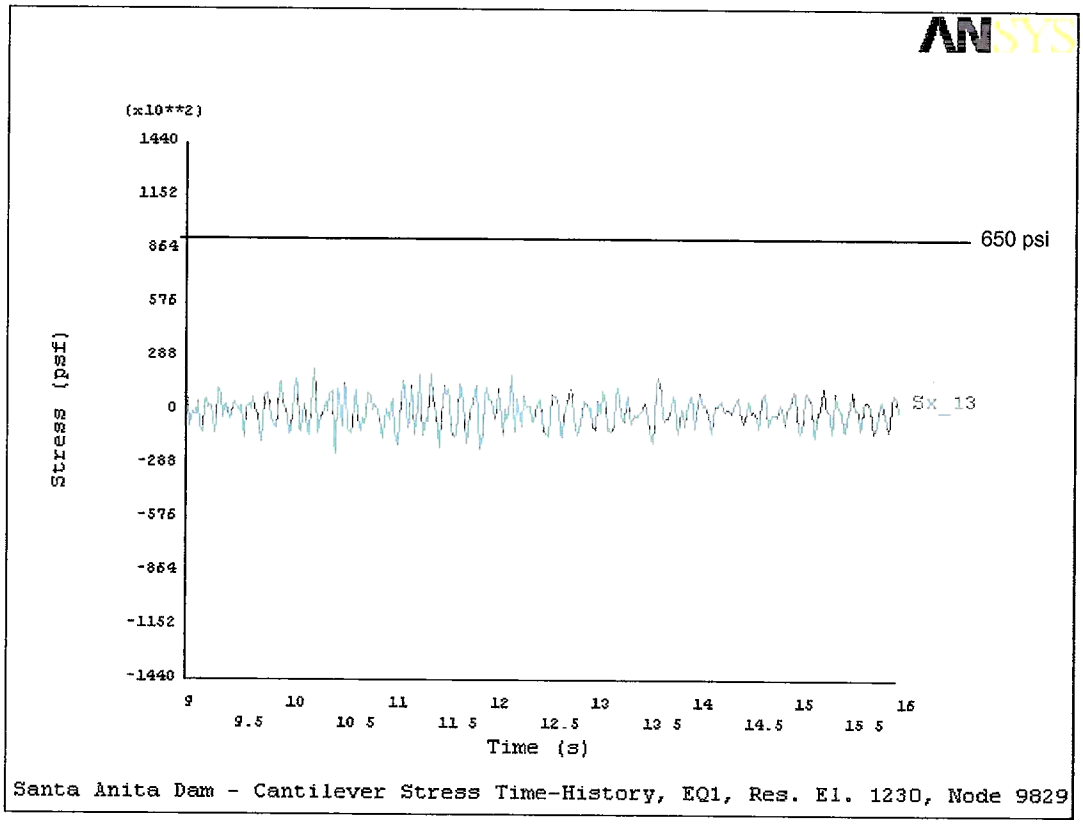


Node 9827 10' from Upstream Face Node 762

Figure 3-51

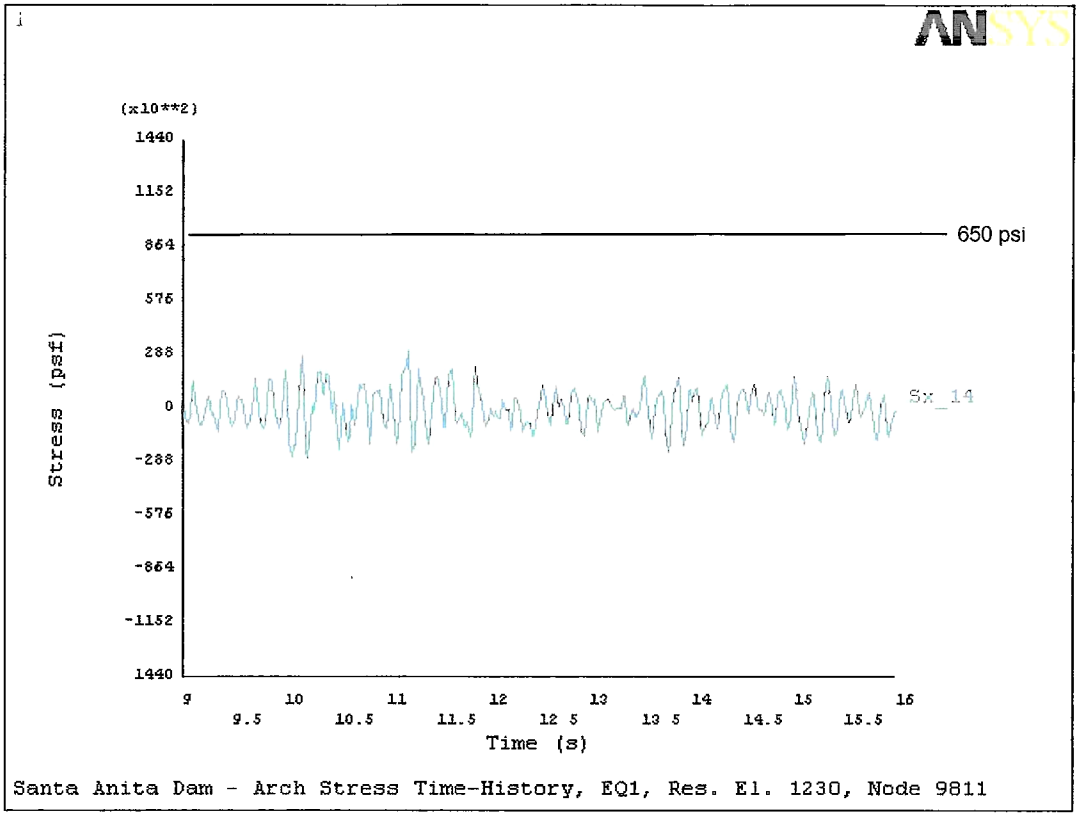


Node 9829 20' from Upstream Face Node 762

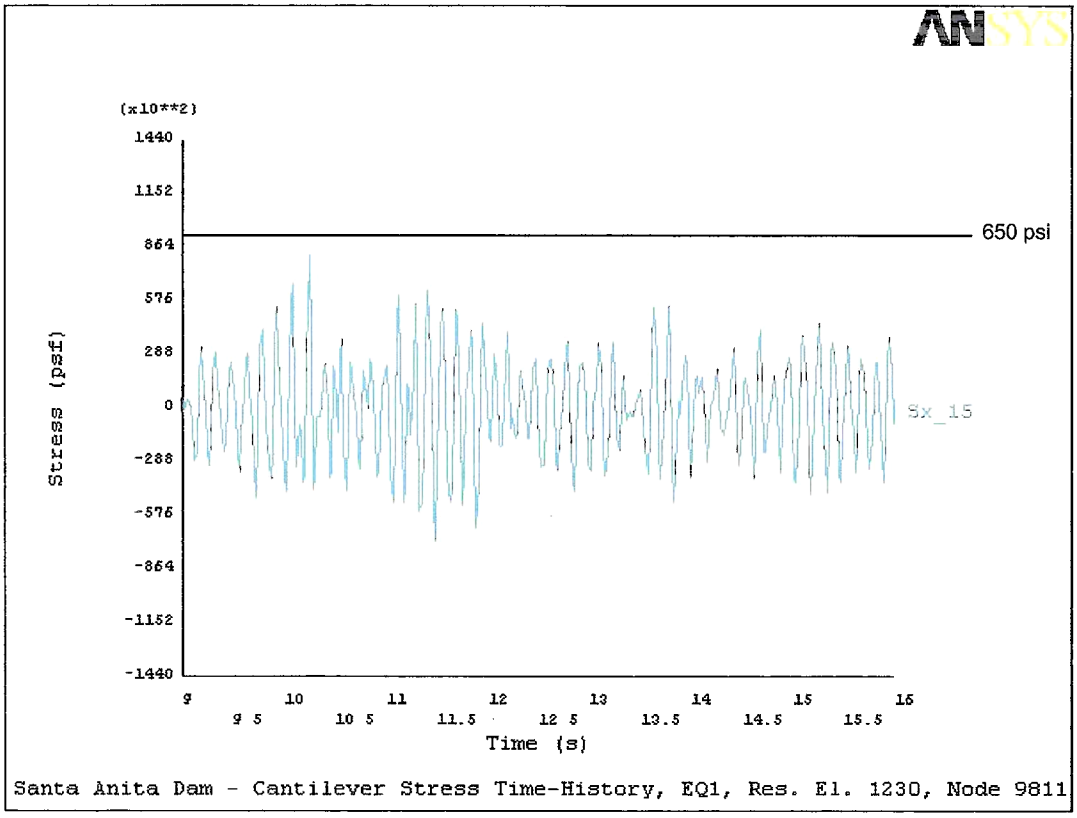


Node 9829 20' from Upstream Face Node 762

Figure 3-52

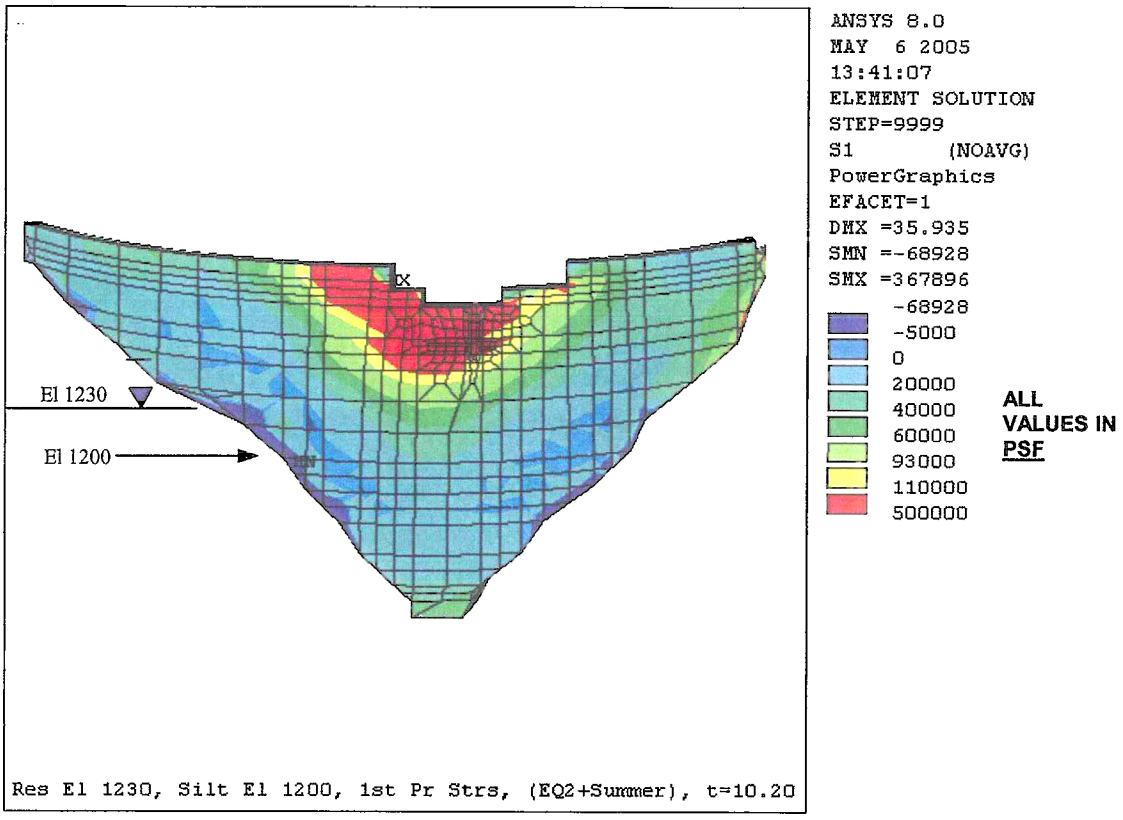


Node 9811 on the Downstream Face, 30' from Node 762

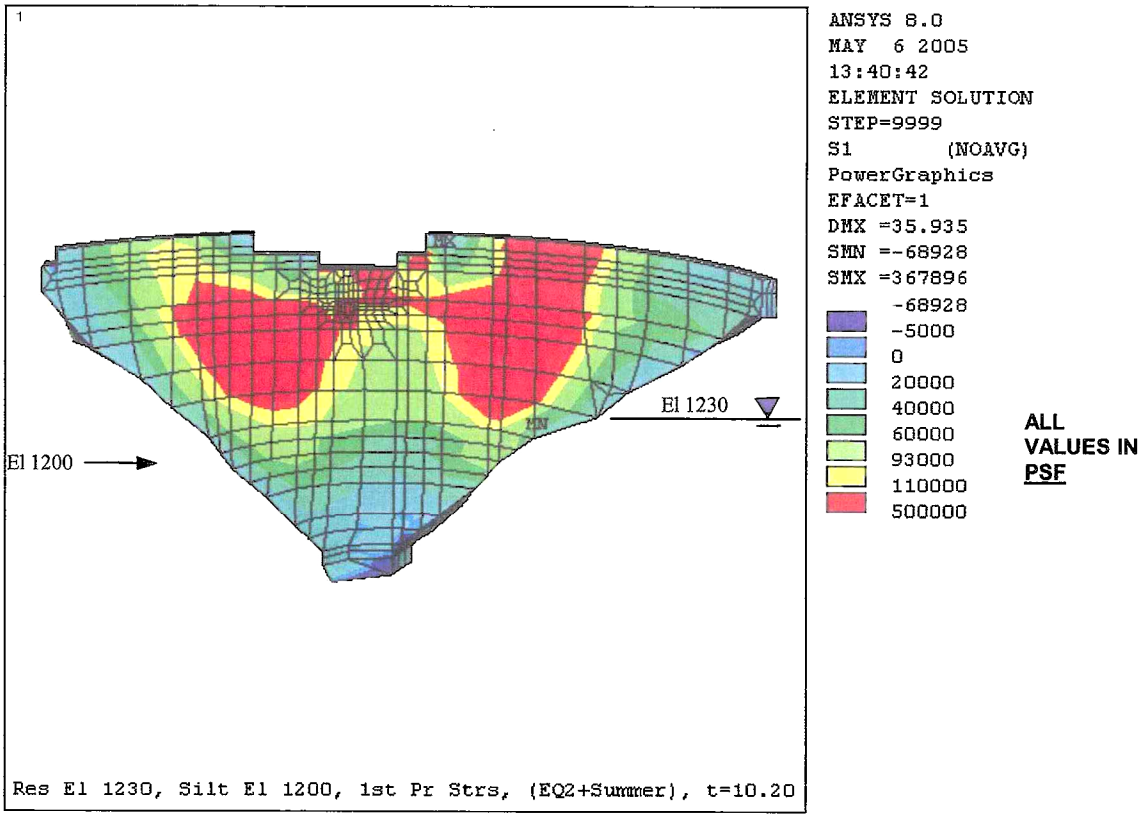


Node 9811 on the Downstream Face, 30' from Node 762

Figure 3-53

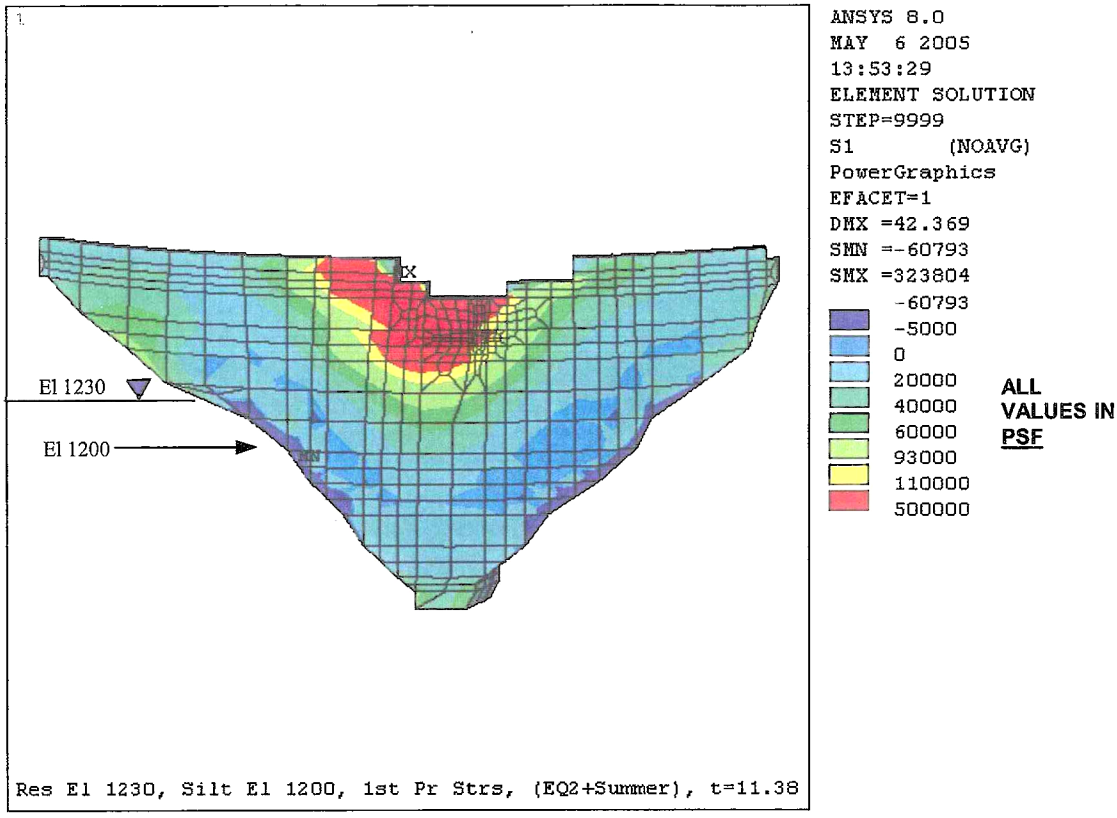


Upstream Face

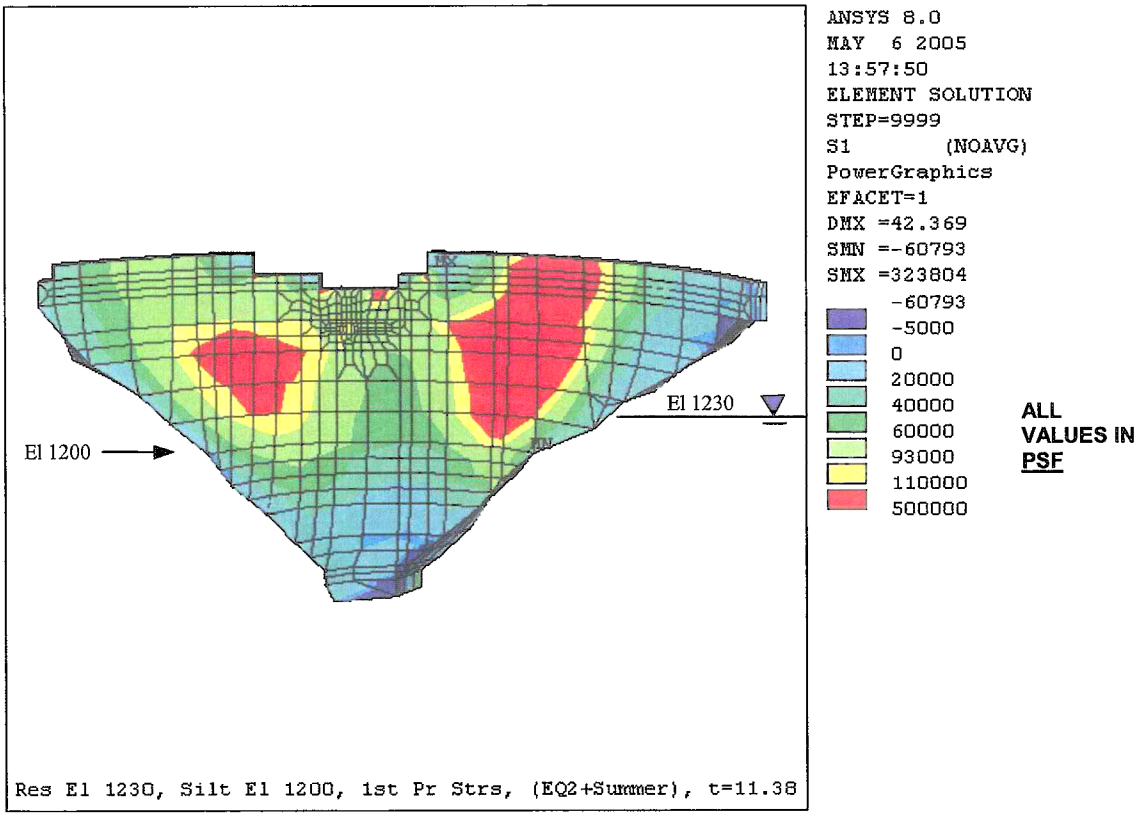


Downstream Face

Figure 3-54

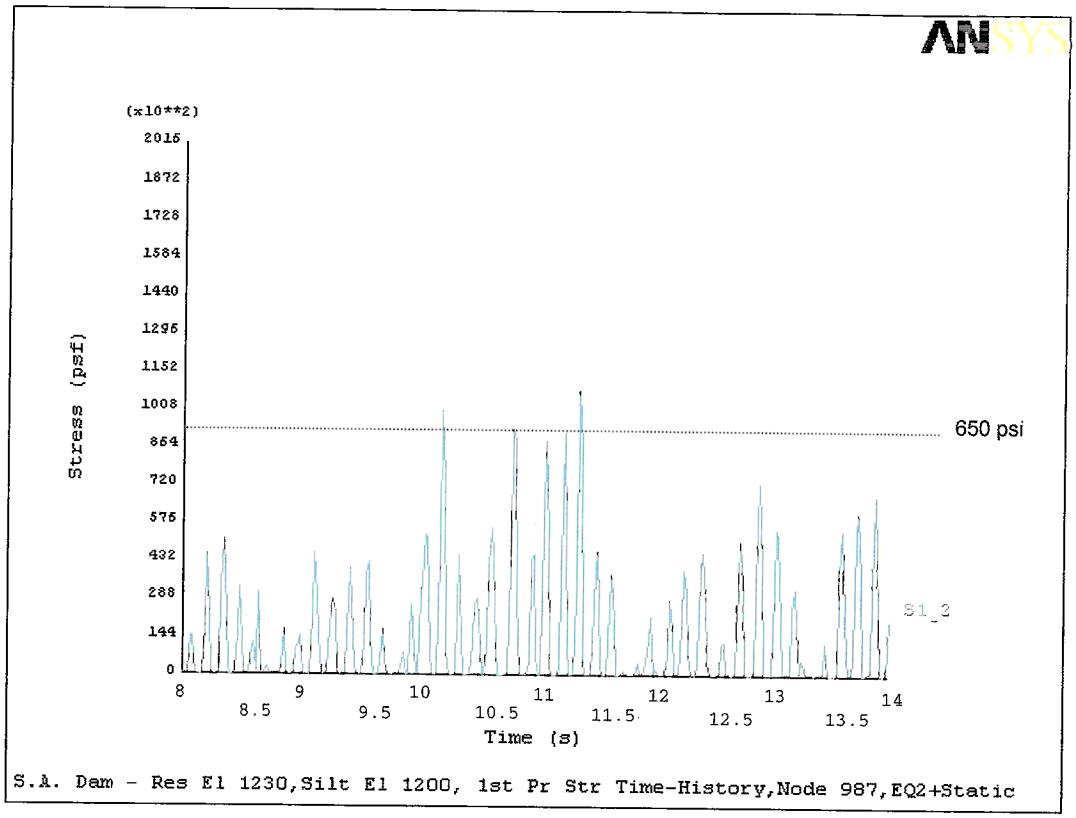


Upstream Face

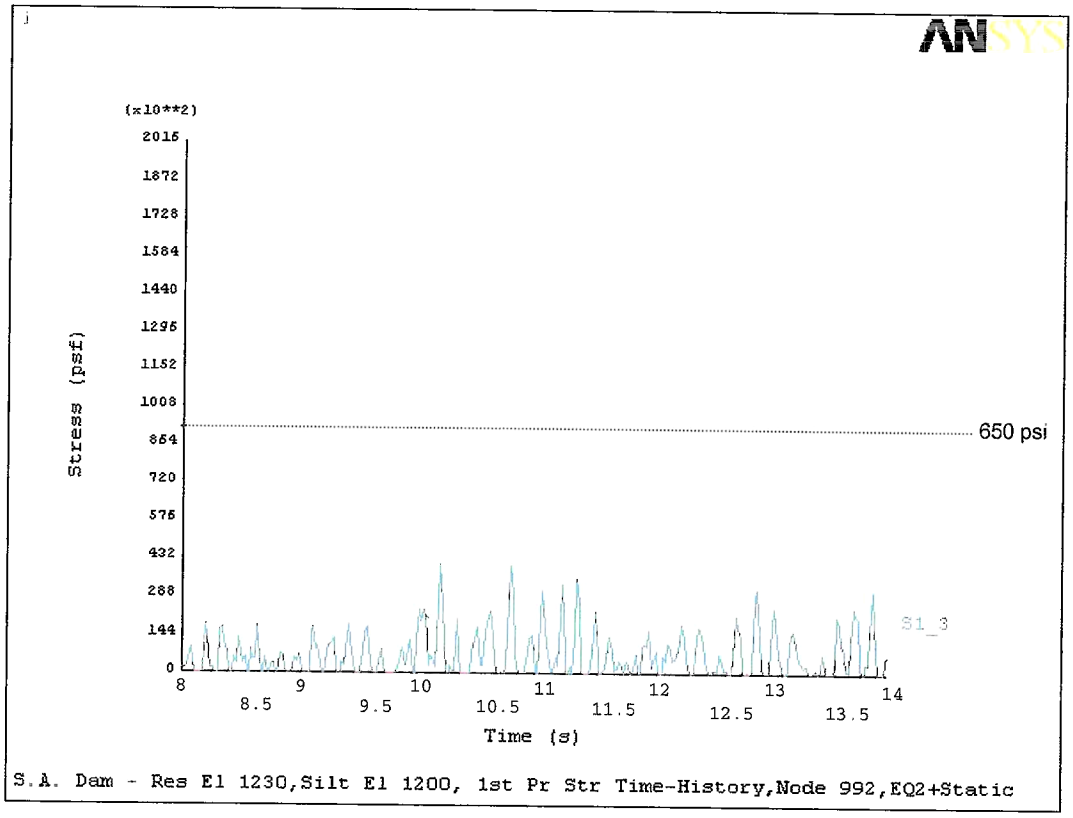


Downstream Face

Figure 3-55

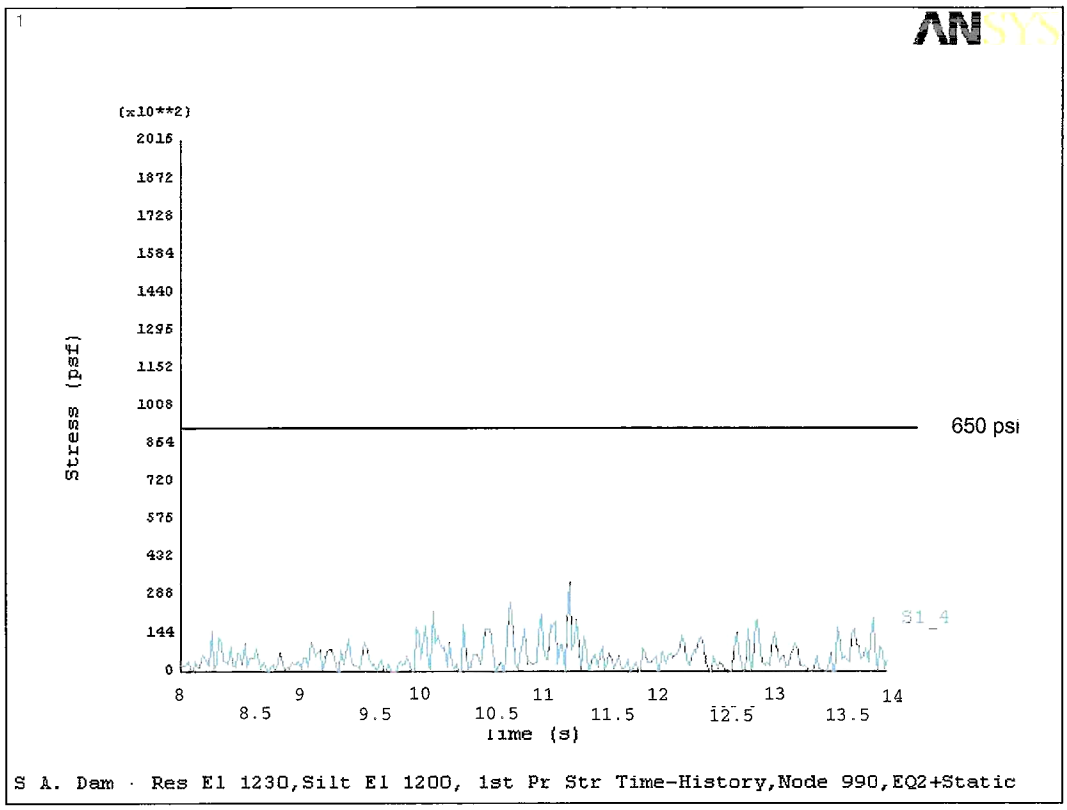


Downstream Face – Node 987

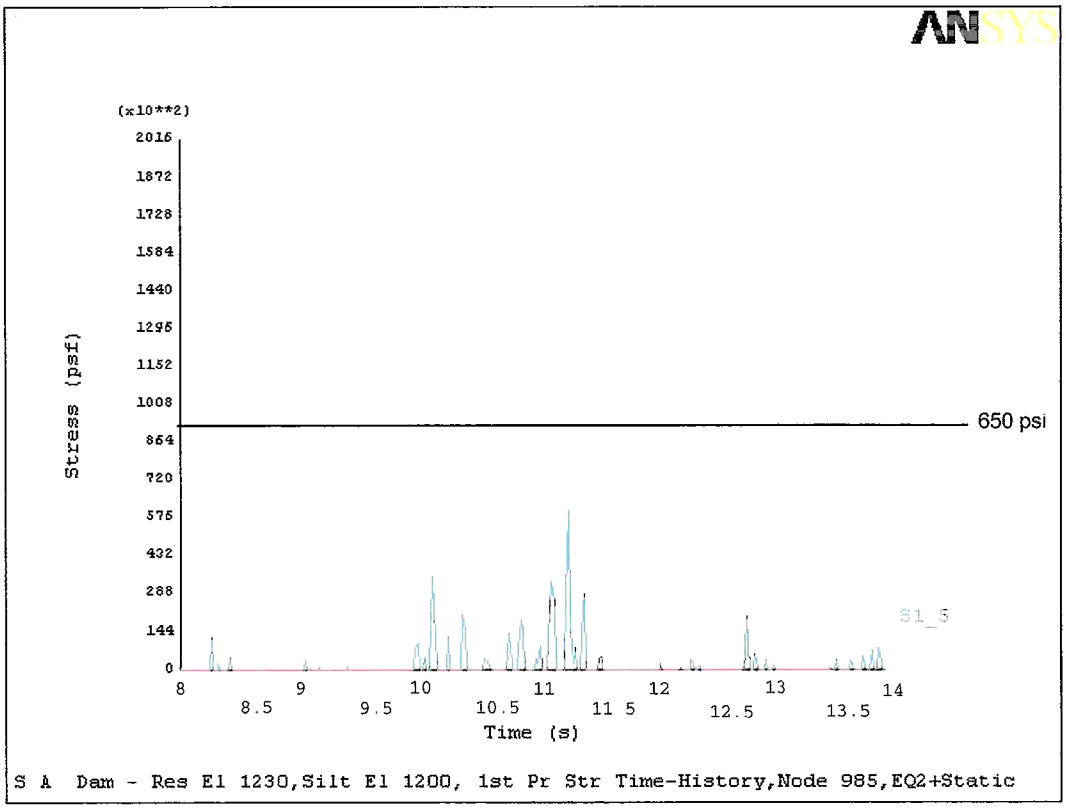


Node 992, 13' from Downstream Face Node 987

Figure 3-56

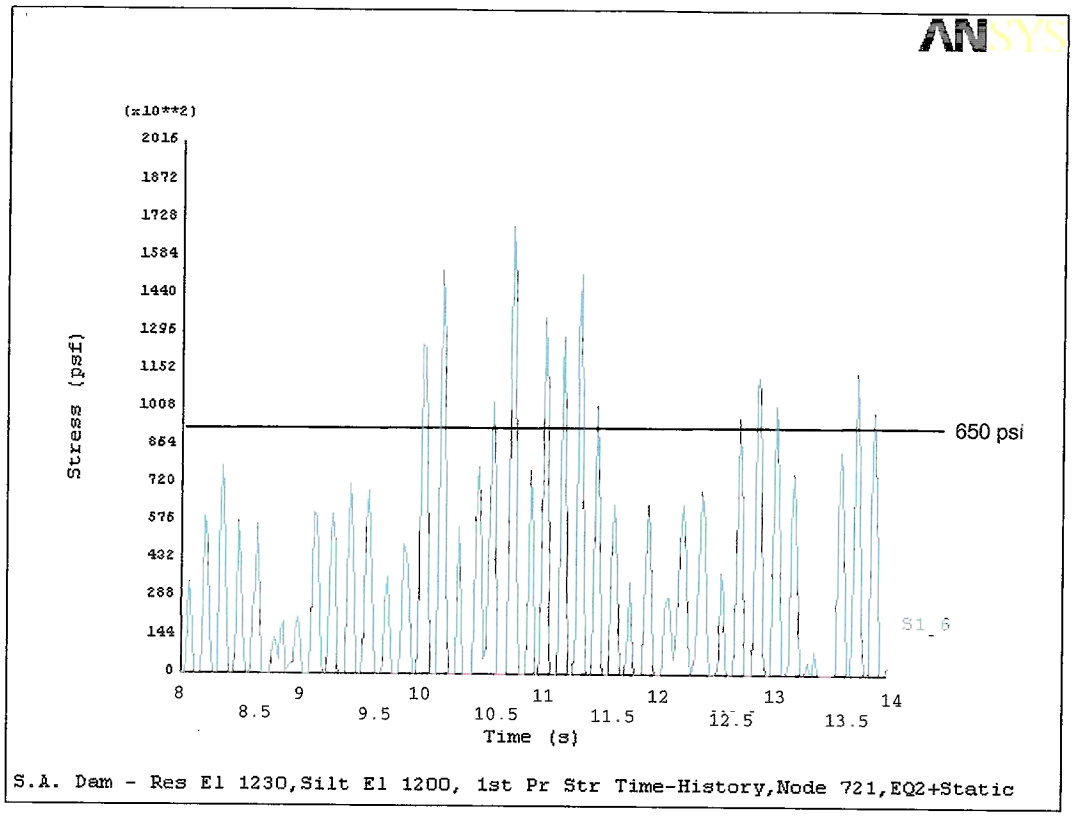


Node 990, 22' from Downstream Face Node 987

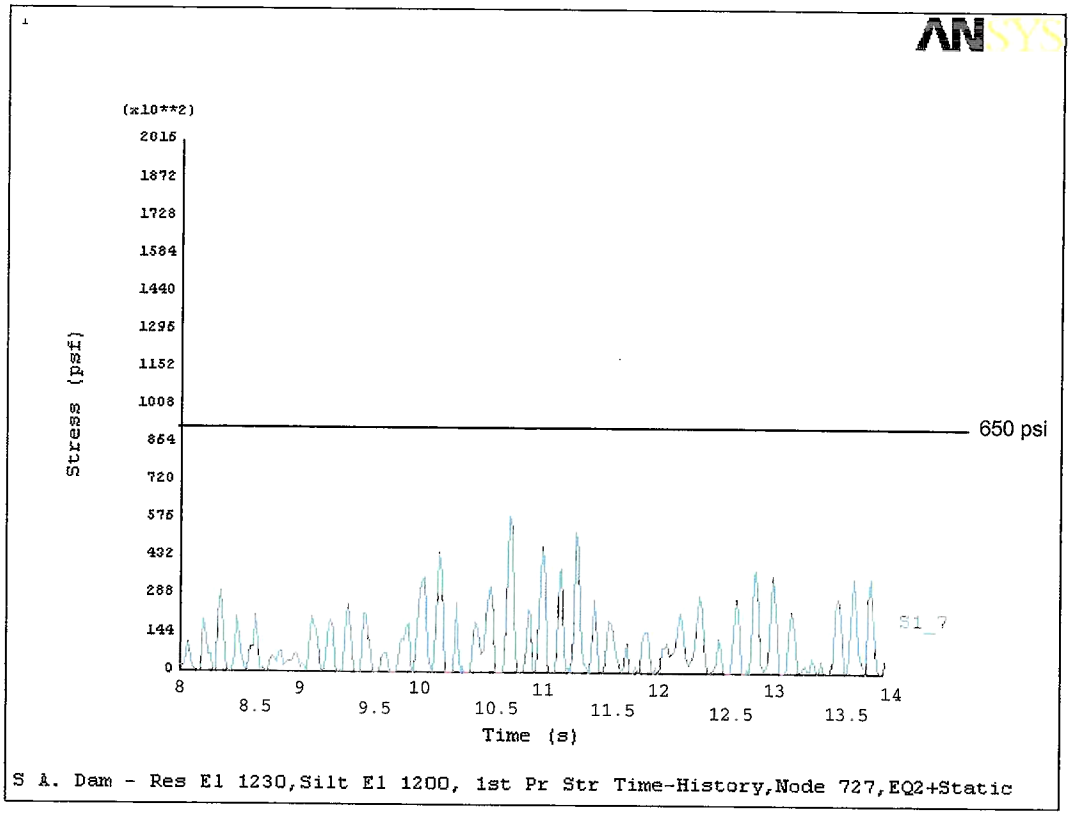


Node 985 on the Upstream Face, 33' from Node 987

Figure 3-57

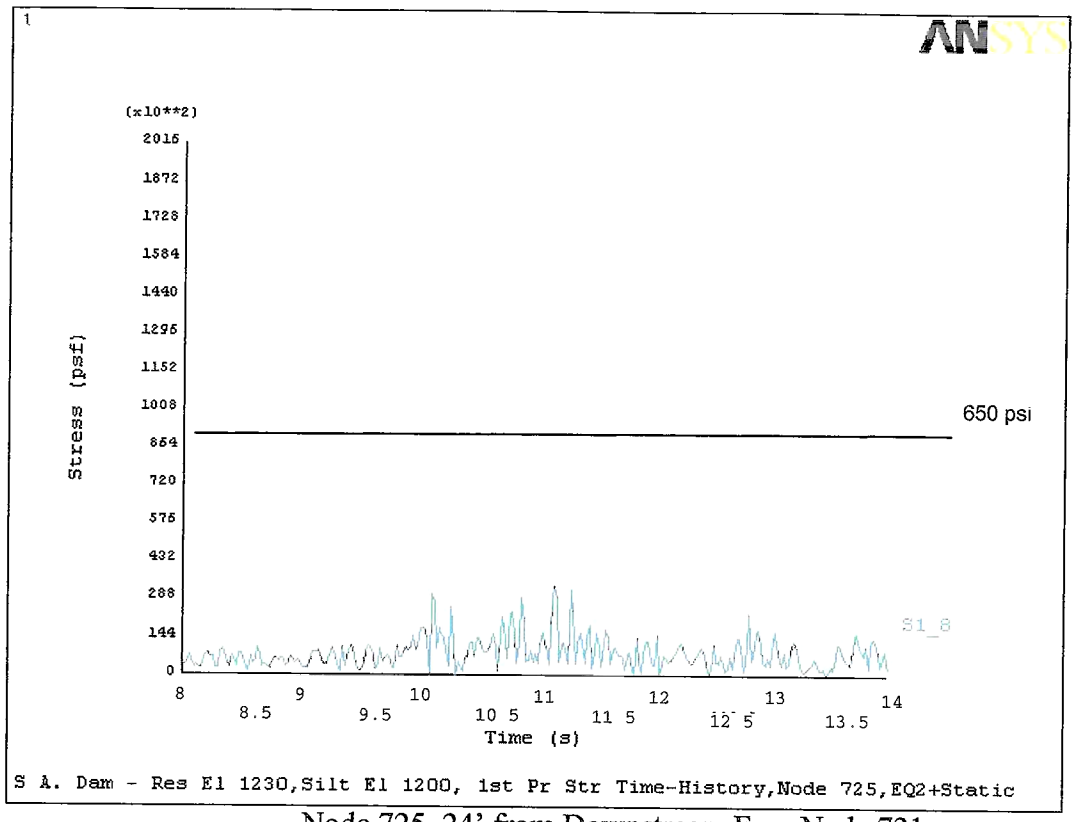


Downstream Face

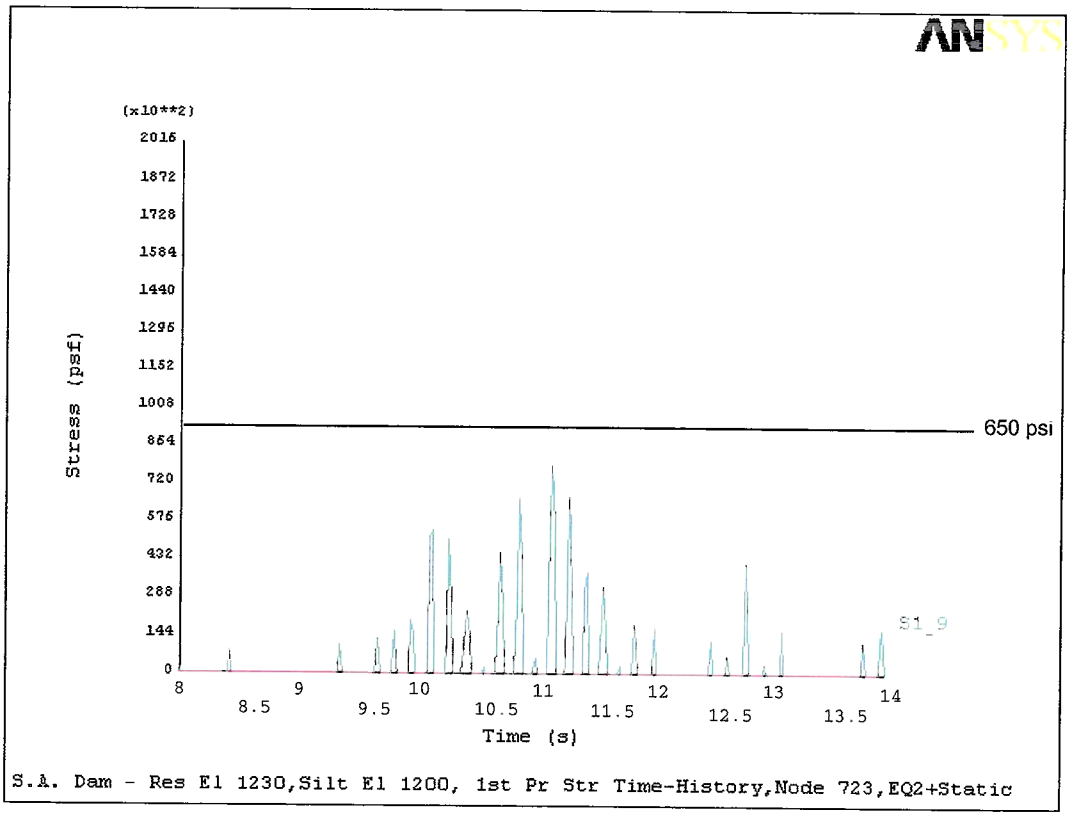


Node 727, 12' from Downstream Face Node 721

Figure 3-58

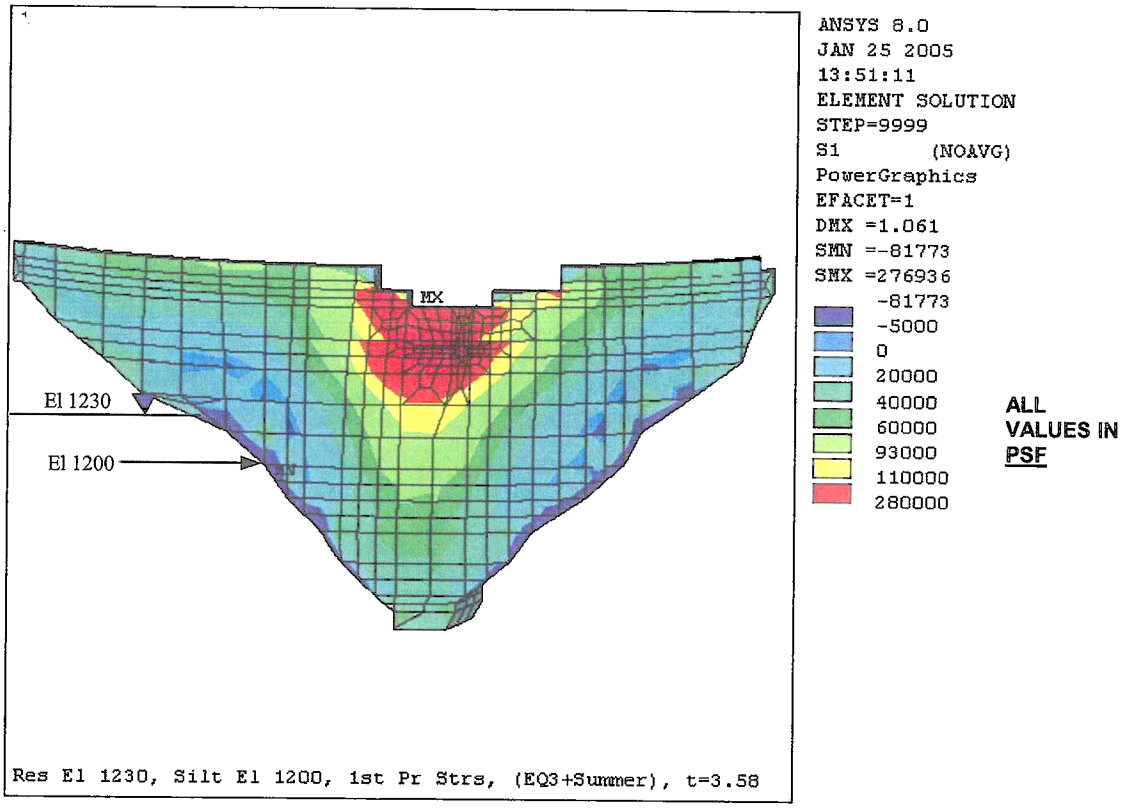


Node 725, 24' from Downstream Face Node 721

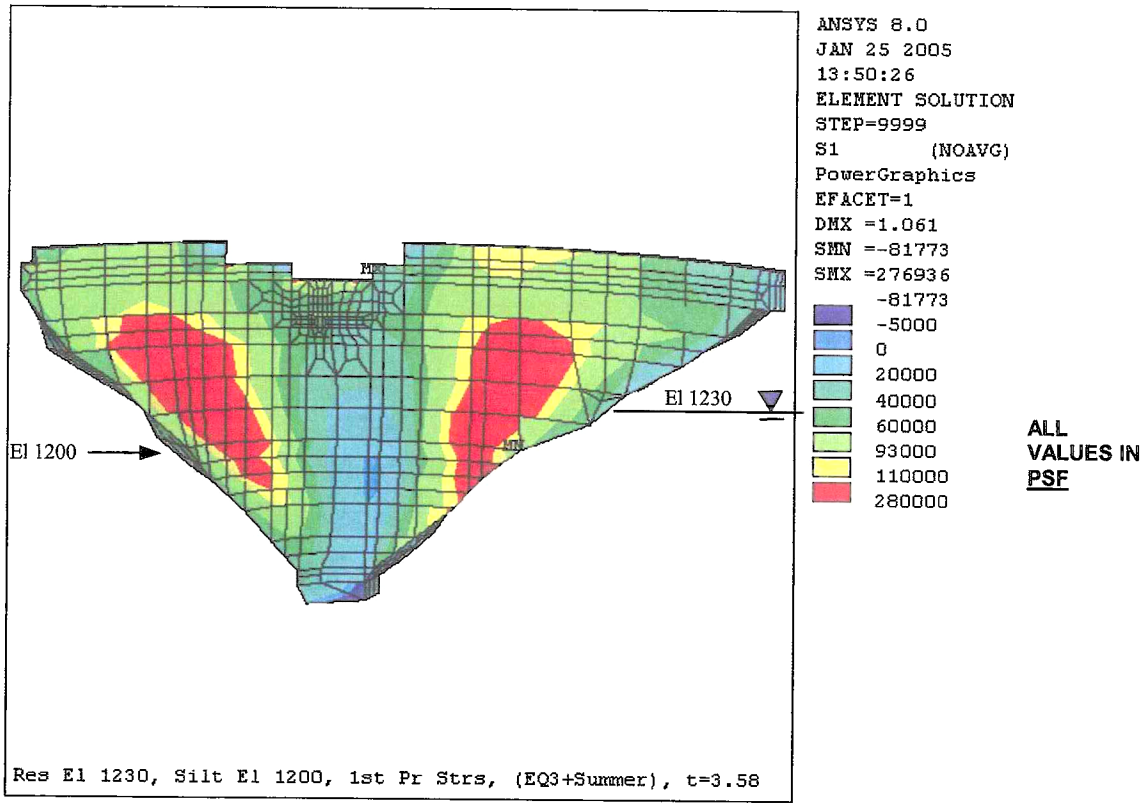


Node 723 on the Upstream Face, 36' from Node 721

Figure 3-59

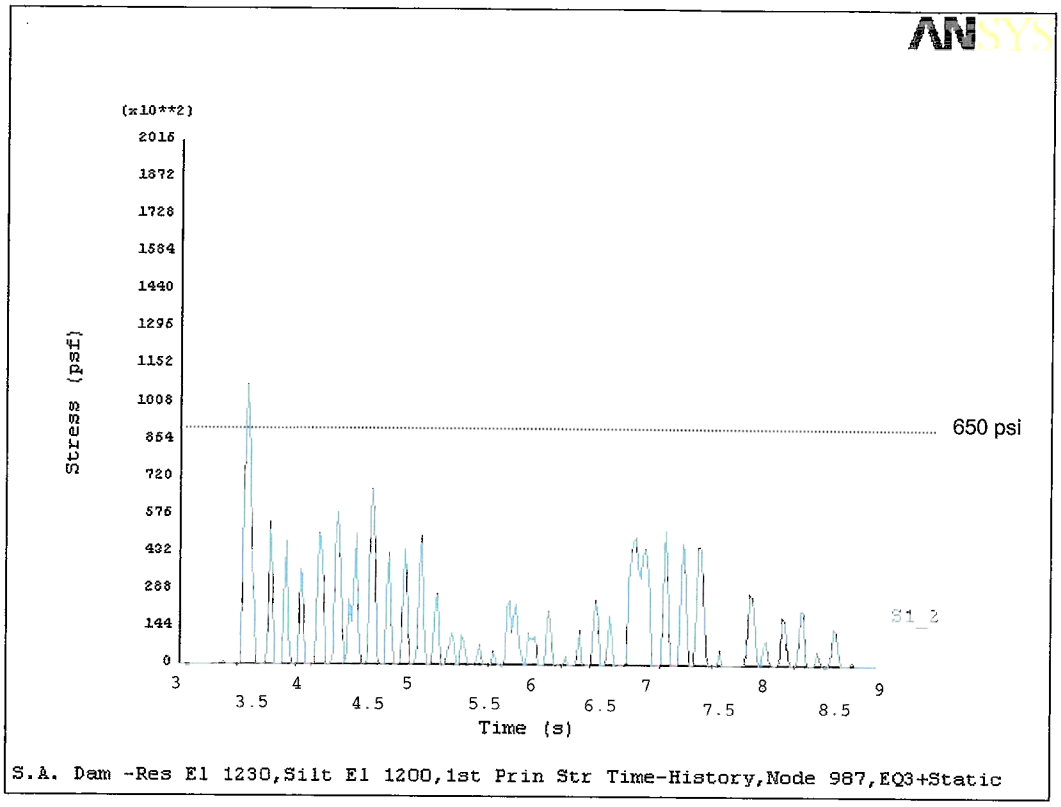


Upstream Face

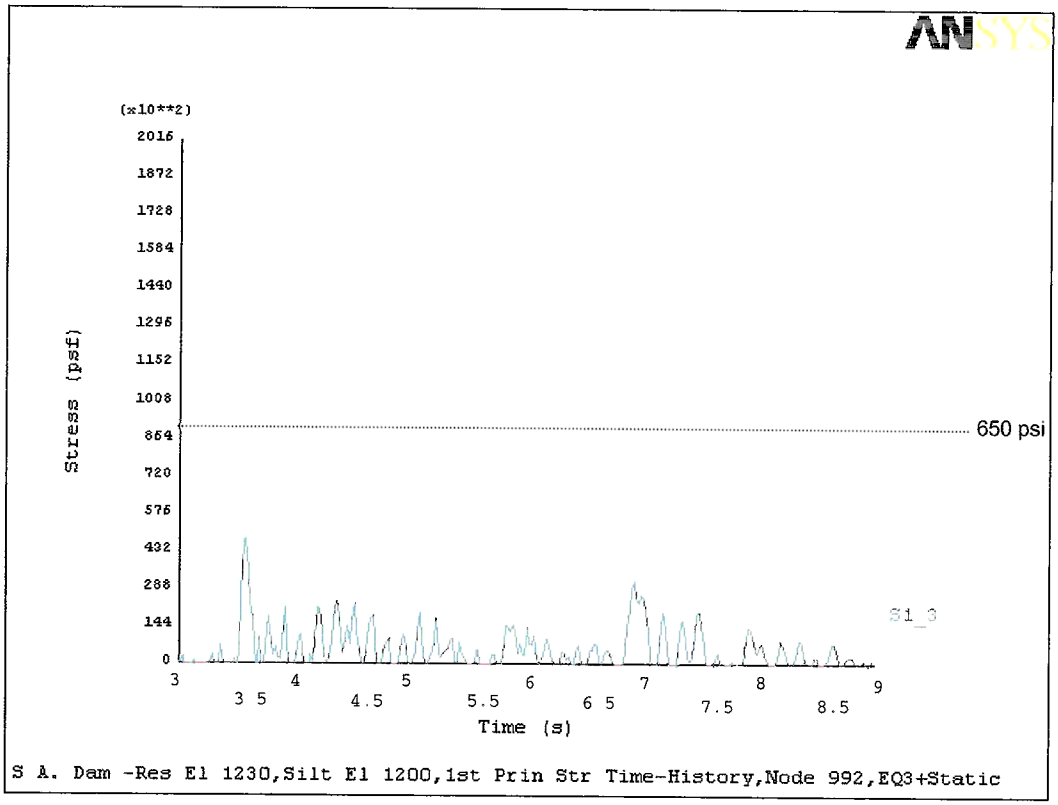


Downstream Face

Figure 3-60

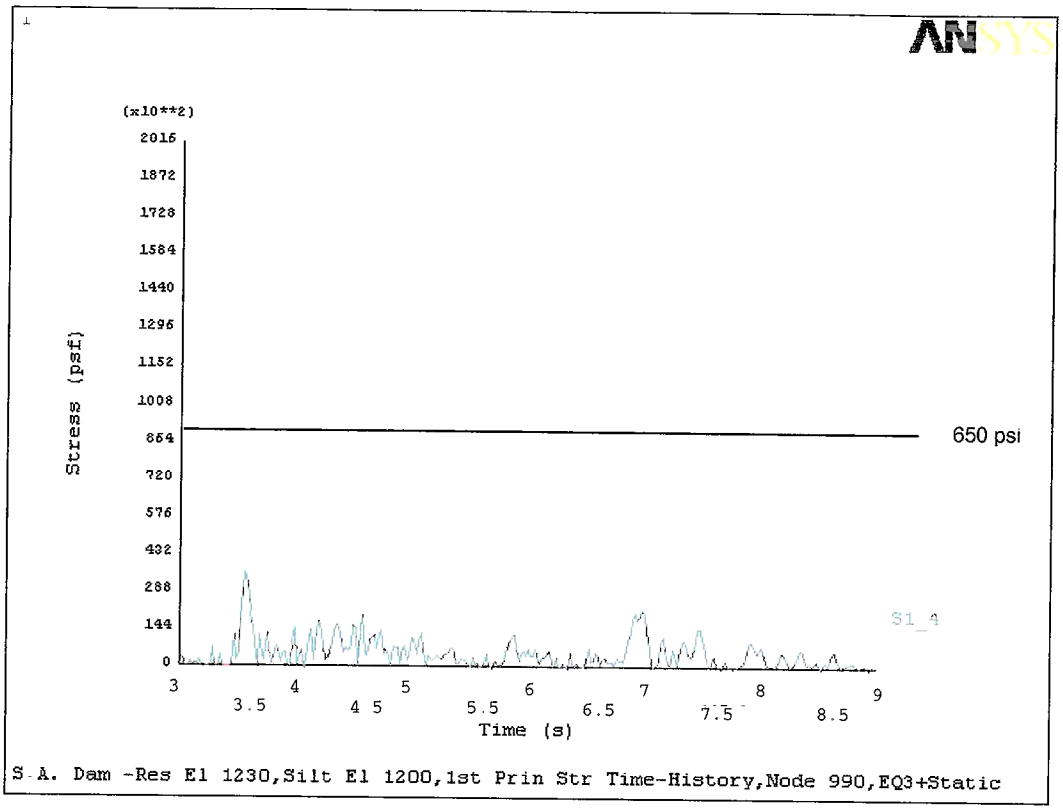


Downstream Face – Node 987

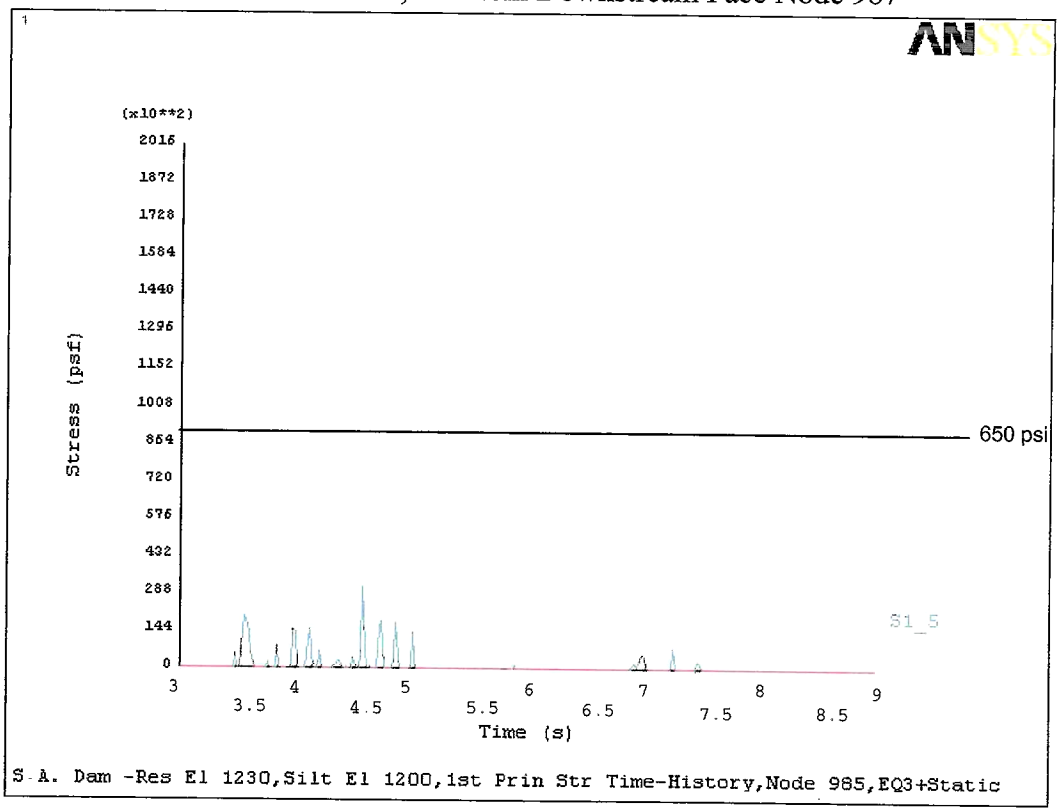


Node 992, 13' from Downstream Face Node 987

Figure 3-61

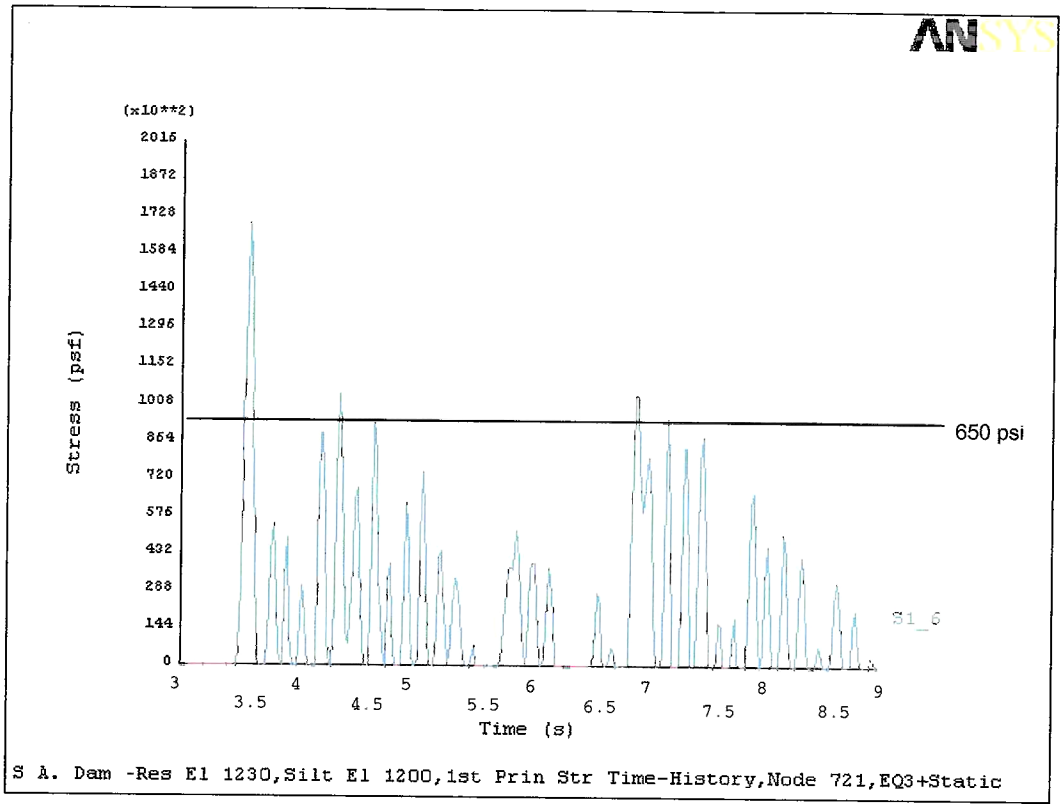


Node 990, 22' from Downstream Face Node 987

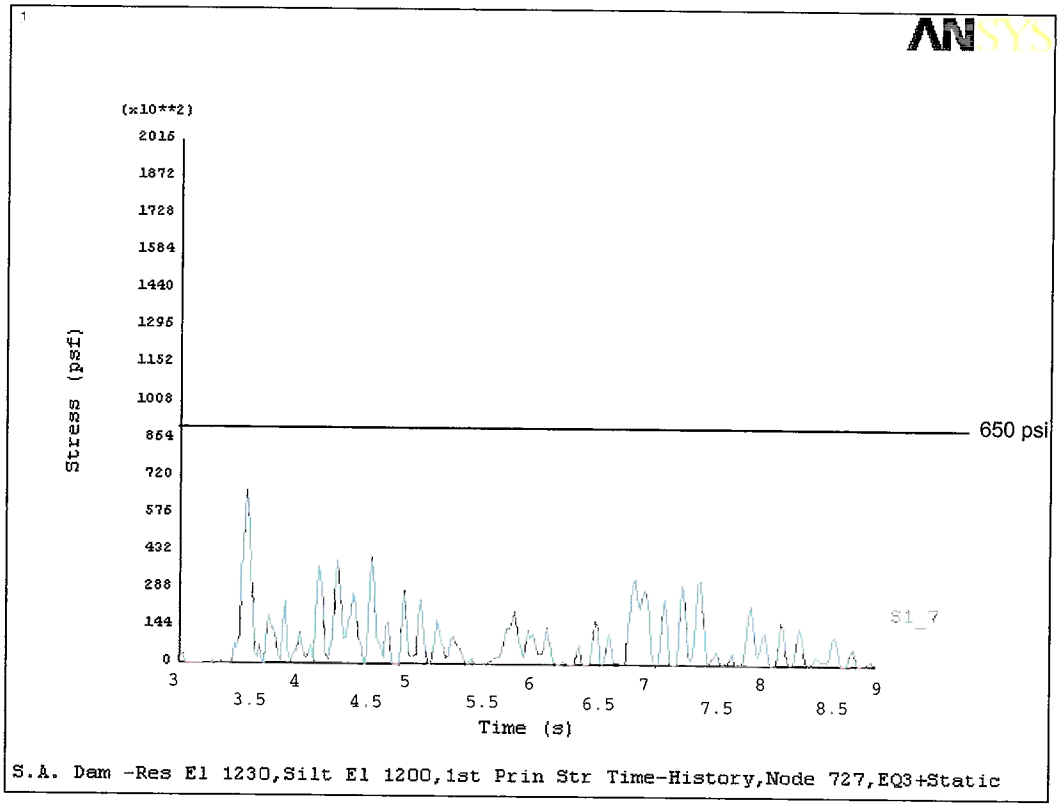


Node 985 on the Downstream Face, 33' from Node 987

Figure 3-62

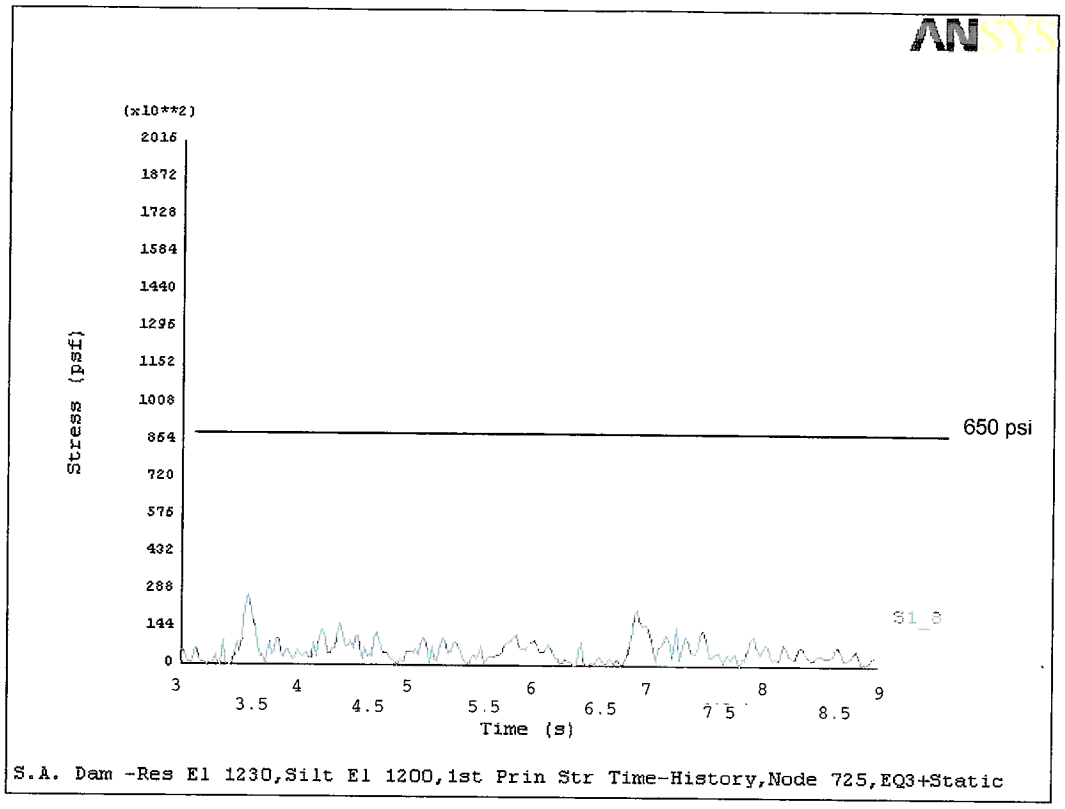


Downstream Face

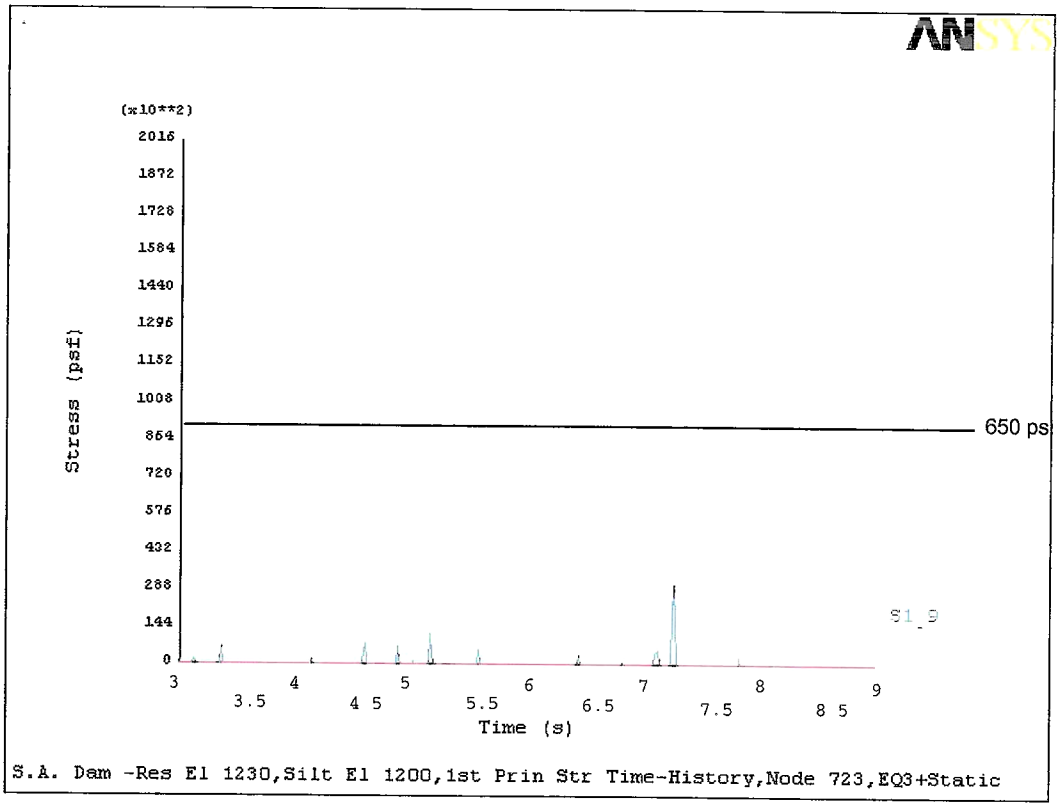


Node 727, 12' from Downstream Face Node 721

Figure 3-63



Node 725, 24' from Downstream Face Node 721



Node 723 on the Downstream Face, 36' from Node 721

Figure 3-64

Table 4-6 also shows the duration of the reservoir level above El 1231.0. Because the intake riser crest is at El 1230 and valves were hot used in the flood routing runs, the reservoir level approaches El 1230 asymptotically

4.10 Spill Trajectory and Impact Area

4.10.1 *Partial Rehabilitation Concept*

A primary consideration in the hydraulic design of the spillway for the Partial Rehabilitation Concept is that the spill should clear the base of the buttress. Spill trajectory is also a consideration for determining impact zones and areas for armoring or of potential scour. Reinforced concrete armoring will minimize scour in downstream spill impact areas. Spill trajectories for the maximum water level during the PMF (reservoir El 1325) and for 1-foot of head on the upper spillway level (reservoir El 1311) are presented on Figure 4-10. The spill trajectories as presented on the Figure all assume no impact on canyon walls. Impact areas would be where the spill trajectory intersects topographic features. The trajectory of spill will vary with water level in the reservoir and flow rates in the spillways. The spill trajectory methodology was as presented in "Design of Small Dams" (USBR, 1987, pg. 376).

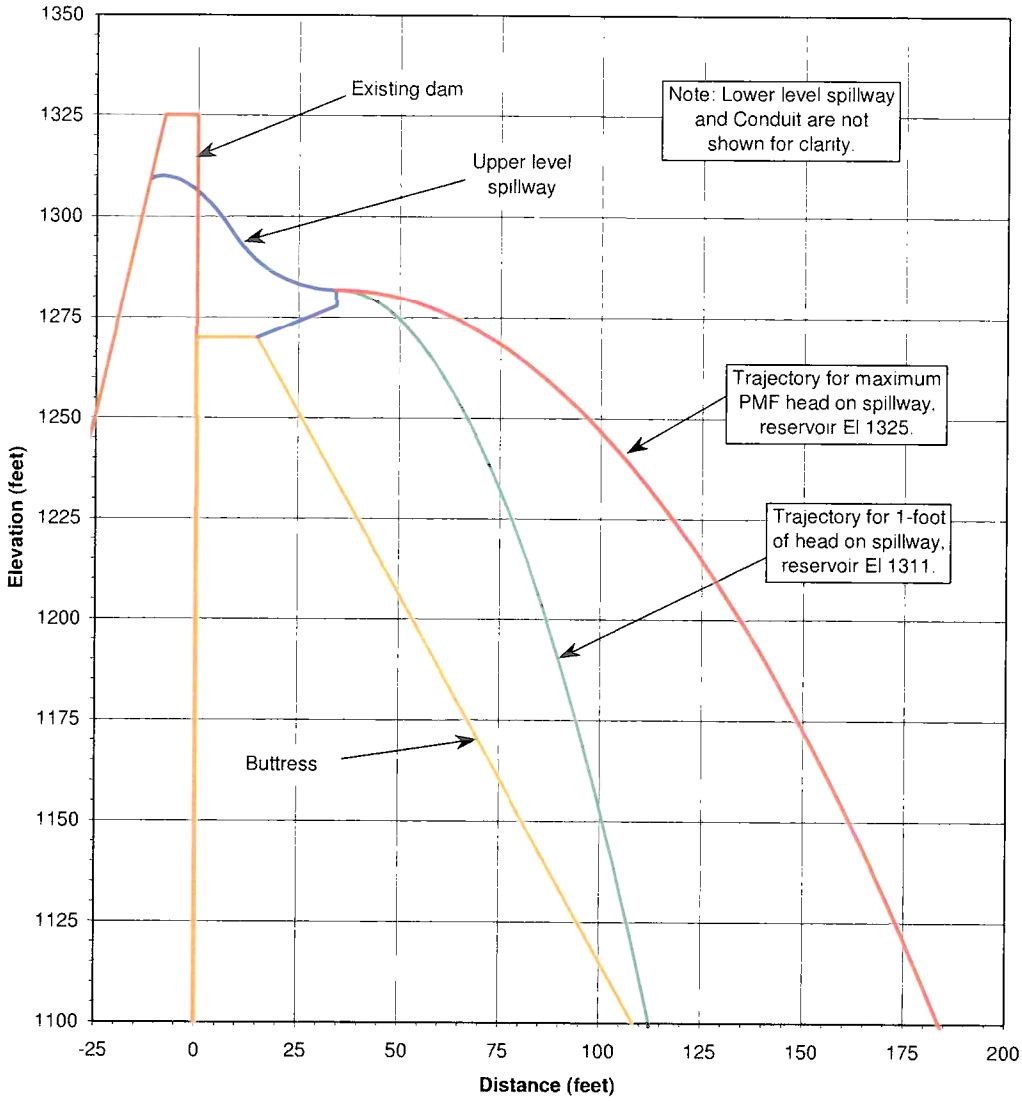


Figure 4-10· Partial Rehabilitation Concept – Upper Level Spillway Trajectories

Figure 4-11 includes similar spill trajectory information for the lower level spillway For 1-foot of head on the lower spillway level, the reservoir level would be at El 1301

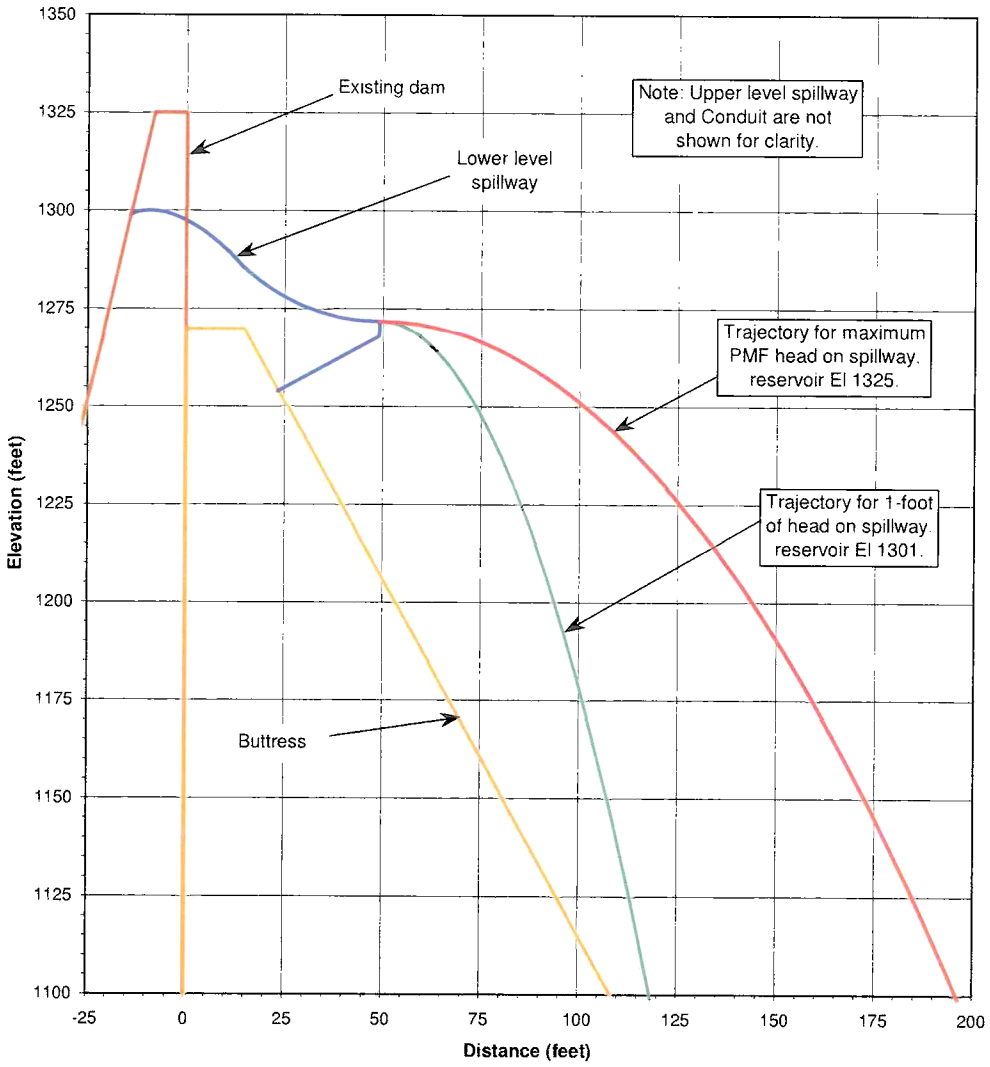


Figure 4-11 Partial Rehabilitation Concept – Lower Level Spillway Trajectories

The lower level of the notch would carry flow most frequently and would carry the most concentrated flow during extreme floods. The impact point at the bottom of the canyon (El 1100) would be about 120 feet downstream of the existing dam for the 1-foot of overtopping and about 195 feet downstream from the existing dam toe at the peak of the PMF

The trajectory of the Partial Rehabilitation Concept octagon orifice assumes that it is cut at a steep slope through the existing dam and curved to discharge horizontally as shown on Figure 4-1 As shown in Table 4-3, with a reservoir level at El 1271 (1-foot of head on the orifice) the orifice outflow would be only 8 cfs. As shown on Figure 4-12, if this low flow did not break-up into a spray, it would impact at about the level of armoring at the downstream toe of the buttress. The spill trajectory for the octagon orifice with reservoir water level at El 1325 assumes no interference with flow from the notch spillway If the octagon orifice were

constructed underneath the notch spillway, the trajectory would be shortened by impact with the notch spillway discharge

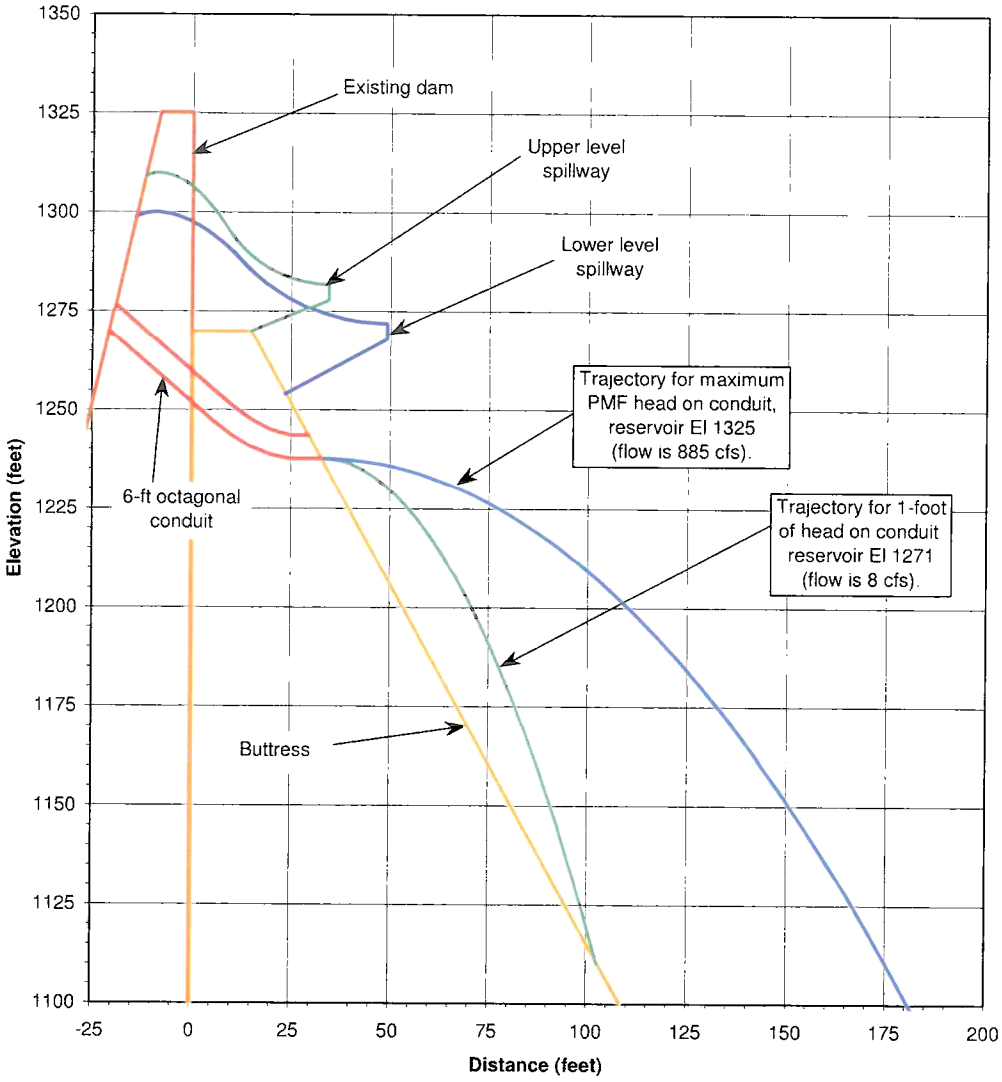


Figure 4-12. Partial Rehabilitation Concept – 6-ft Octagon Conduit Trajectories

Canyon sidewall and floor armoring will be extended as shown on Drawing 2-1 Drawing 2 1 also shows the limits of the expected spill impact areas for both the PMF and the Capital Flood.

4 10.2 *Intake Riser Modification Concept*

For the Intake Riser Modification Concept as shown on Figure 4-13, there is no need for the spill to clear a buttress. The trajectory and impact areas for spill from the upper and lower level notch for peak PMF levels would be closer to the toe of the existing dam than for the Partial Rehabilitation Concept At the peak of the PMF, with a reservoir level at El 1325, the upper and

lower spillway levels would have impact points about 110 feet and 135 feet downstream from the toe of the existing dam, respectively.

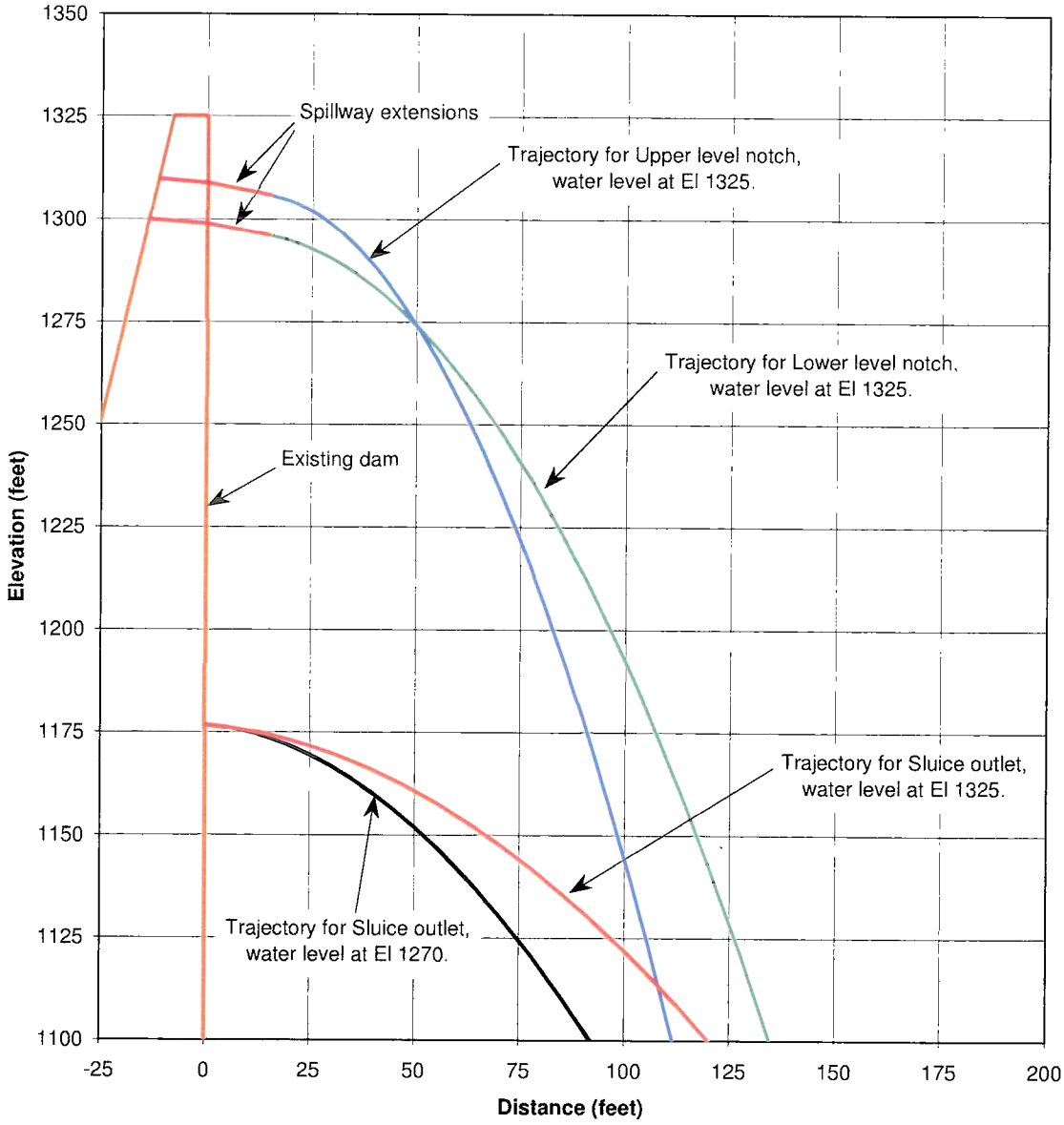


Figure 4-13 Intake Riser Modification Concept –Spill Trajectory for Notch Spillway and Sluice Outlet

For the PMF peak with a reservoir level at El 1325, the sluice outlet would have an impact point that is about 120 feet downstream without considering the effects of tailwater For a reservoir level at El 1270, where tailwater would be low enough to have little effect on the jet trajectory, the impact point would be about 90 feet downstream.

5.0 CONSTRUCTABILITY AND OPINION OF PROBABLE CONSTRUCTION COSTS

A constructability review and opinion of probable construction cost estimates were performed for the two alternatives presented in this report. The review and cost estimates were developed as follows:

- Identification of the main components of the modifications in terms of design and construction;
- Preliminary quantity take-off for various major construction items;
- Estimation of construction unit costs based on estimated published unit costs, then modified for the project area and site difficulty using judgment; and
- Computation of total construction costs using further judgment based on site and access difficulty.

It should be noted that, at this phase of study, the project layouts, quantities and costs are rough and carry a high contingency factor.

5.1 Opinion of Probable Construction Costs

Opinion of Probable Construction Costs (OPCC) for both concepts are included in Tables 5-1 and 5-2. The following assumptions were made in developing the OPCC:

- LACDPW's cost of lowering the reservoir and loss of revenue due to water loss are not included in these construction cost estimates.
- Estimate is classified as a Class IV cost estimate as defined by the Association for the Advancement of Cost Engineering (AACE), with an accuracy range of -20 to +35%.
- Construction Costs are as of 4th quarter 2005. Inflation has been added at the rate of 1.6%/per quarter to the MPC. CCI = 8860 LA County
- Estimate assumes a 1 year construction schedule.
- No post hurricane Katrina/Rita/Wilma inflation impacts have been considered.
- Single shift basis - 5 days per week. Minimal overtime included.
- Competitive market conditions prevail at time of bid tender – assumes 3+ bidders per trade.
- Construction start - 1st quarter 2007.
- Costs of obtaining permits and unknown consequences excluded.
- NPDES permit discharge restrictions not known or considered.
- All excavated materials and demolished concrete may remain on-site.
- Salvage value for existing dam equipment equals demolition costs.
- Owner shall provide a source of construction water and power near the dam.
- All permits will be in place prior to Contractor's notice to proceed (NTP).
- Builder's Risk Insurance is included at 1% of costs. If Builder's Risk is not available to the General Contractor due to the scope or magnitude of this project, increased bid margins can be expected.

- The costs of reasonable measures to minimize environmental impacts associated with construction activities have been included. Provisions for costs of environmental impacts not associated with normal construction practices have been excluded.
- A sufficient water reservoir will be maintained on the upstream side of the dam to support a crane raft system.
- Existing penstock valves will be reused.

5.2 Constructability

The constructability issues for the Partial Rehabilitation Concept and Intake Riser Modification Concept are presented below

5.2.1 Partial Rehabilitation Concept

The main constructability issues for both concepts are cutting and removing of the central portion of the dam for a length of approximately 112 feet (80-feet for partial rehabilitation) down to a depth of approximately 15 feet from the crest, a length of 40-feet down an additional 10-feet, rounding the spillway corners, and then constructing a spillway platform extension (intake riser modification) or flip-bucket (partial rehabilitation). Performance of this work from the top of the dam would require specialized techniques, such as lifting of the cut-up blocks of concrete via a crane on the lowered reservoir or a suspended cable above the dam and staging area. Also, constructing the spillway platform extension or flip bucket (e.g., forms, reinforcing steel, concrete placement, etc.) on the downstream edge of the notch crest would involve specialized installation procedures.

Anchors installed in the downstream erosion protection area, used to anchor protective concrete of the splashpad extension and lower abutment armoring, provide construction difficulty due near-vertical canyon walls downstream of the dam. In addition, splashpad/armoring concrete forms and/or shotcrete placement along near-vertical canyon walls provide possible construction difficulty

Cutting of the 6-ft diameter orifice (partial rehabilitation) will involve specialized wire sawing techniques (diamond-impregnated wire saw). Jacking equipment would be used to remove the core from the resulting orifice. The core would need to be cut into pieces as it is removed to allow for handling and removal of the concrete blocks. Afterwards, installing a large 6-ft diameter steel pipe inside the orifice would pose difficulty on the vertical dam face at El 1270.

5.2.2 *Riser Modification Concept*

In addition to the similar constructability issues shared with Partial Rehabilitation Concept, such as notching and armoring, the Riser Modification Concept will require difficult construction of an extended riser and trashracks on the near vertical upstream face of the dam. This will require construction of forms, placement of steel reinforcement, and pouring of concrete at heights up to 70 feet above the silt elevation (El. 1180). Moreover, relocation of the large steel gate and associated operating equipment will be performed under similar conditions. However, it is

anticipated that the same equipment (e.g., crane) used to remove the concrete blocks from the spillway notch will be used to assist in construction and placement of these structures.

Table 5-2
30% Opinion of Probable Construction Cost - Intake Riser Modification Concept
Santa Anita Dam
Seismic Rehabilitation & Spillway Modification Project

Item	Description	Quantity	UOM	Unit Cost	Total Cost	Comment
1	Mob/Demob & Contractor Field GCs	1	ls	$ 540,000	$ 540,000	10.0% of the Subtotal of Items 2-8
2	Temporary Controls/Logistics	1	ls	$ 940,000	$ 940,000	Allowance for Temp Measures, Hoisting, Staging
1	Temp Erosion Controls	1	ls	$ 25,000	$ 25,000	Silt fence, hay bales, CL & orange fencing, spill controls
2	Temp Power/Light/Communications	1	ls	$ 35,000	$ 35,000	Establish/run temp power and lights
3	Flexi-Float Barges Rent/Buy	12	mos	$ 22,000	$ 264,000	At US side for crane platform, 75x75' platform area
4	Mobe/Erect/Demobe Flexi-Floats & Crane at Base	1	ls	$ 100,000	$ 100,000	Incls erection, helicopter time, assembly, demobe
5	Rent/Opr 100-150 Ton Crane at Dam Base	1,200	hr	$ 350	$ 420,000	Rebar/from hoisting; 6 mos
6	Rigging Platform	1	ls	$ 50,000	$ 50,000	Staging/Assembly
7	Misc	1	ls	$ 50,000	$ 50,000	Crew access to dam base, safety, working platforms
3	Diversion & Care of Water	1	ls	$ 25,000	$ 25,000	Allowance for Routing Storm Event Water Over Works
4	Thicken Arch - Dam Mods	1	ls	$ 2,700,000	$ 2,700,000	Buttress/Orifice & Misc. Construction
1	Relocate Jib Crane	1	ls	$ 6,000	$ 6,000	Move/Store/Reinstall
2	Wire Saw Cut/Demo 7' Dia Orifice	36	cy	$ 5,000	$ 180,000	Wire saw, jack/cut blocks for crane pick; Penhall quote 10/03
4	Wire Saw Cut Notch/Rem Blocks To US Dam Side	1,000	cy	$ 2,000	$ 2,000,000	Qty incls aux bowl, parapet wall and emergency sw extension
5	Misc Demo	1	ls	$ 15,000	$ 15,000	Railings/access ways/misc metals/piping/utilities
6	Sandblast/Prep (E) Dam Surface	1,000	sf	$ 10.00	$ 10,000	Sand/water blast/grout/patch for contact area
7	New Riser - Concrete Forms	850	sf	$ 35.00	$ 29,750	Purchase/fab/erect custom steel forms for riser section
8	New Riser - Doweling	1	ls	$ 10,000.00	$ 10,000	Allowance for IF dowels
9	New Riser - Rebar	6	tn	$ 2,100	$ 11,550	at 300#/cy
10	New Riser - Concrete CIP	35	cy	$ 1,500	$ 52,500	F/P/S/F circular cone section, pumped from transit trucks
11	New Riser - Sluice Gate / Hoisting Cylinder	1	ea	$ 5,000.00	$ 5,000	Move (E) 6'x6' with operator
12	New Riser - Move (E) Trash Rack Section	1	ea	$ 7,500.00	$ 7,500	30' circular section, dismantle/re-install
13	New Riser - New Trash Rack Section	1	ea	$ 15,000	$ 15,000	16' conical section, fab & install; 200 sf at 15#/sf
14	Retrofit (E) Trash Racks on 30" Intakes	300	sf	$ 25.00	$ 7,500	Rehab with trash rack w/ smaller wire interior screen
15	Extend (E) Trash Rack	200	sf	$ 50.00	$ 10,000	Extend trash rack at 2 risers, install wire screen
16	Dam Misc Metals	1	ls	$ 15,000	$ 15,000	Railings/walkways/gratings/stairs/ladders
17	Dam Misc CSI 4-15	1	ls	$ 15,000	$ 15,000	Allowance for misc items
18	Concrete Abutments/Ramps at Pedestrian Bridge	12	cy	$ 1,500	$ 18,000	F/P/S/F Incls rebar and anchors
19	Purchase/Install Pedestrian Bridge w/ Ramp HRs	900	sf	$ 300	$ 270,000	Prefab, 112' span - 8' wide, wt unknown; place in 3 sec w/ lagging
20	Electrical/Instrumentation/Valve Controls	1	ls	$ 25,000	$ 25,000	Minor Scope
5	Valve House/Penstock Mods	1	ls	$	$	No Scope
6	Spillway Mods	1	ls	$ 310,000	$ 310,000	New Spillway Notch Extensions
1	Build/Install SW Ext Forms	2,500	sf	$ 20.00	$ 50,000	Bottom/side forms
2	Temp Support for SW Ext Concrete	1	ls	$ 50,000	$ 50,000	Form bracing system; scope TBD, safety work platform
3	SW Notch Extensions Concrete - Placed	160	cy	$ 850	$ 136,000	P/S/F, pumped from transit trucks, 350'
4	SW Notch Extension Rebar - Placed	20	tn	$ 2,200	$ 44,000	at 250#/cy
5	Drill/Grout Misc Dowels	1	ls	$ 30,000	$ 30,000	Factored from Big T
7	Abutment/Spillway Armoring	1	ls	$ 1,210,000	$ 1,210,000	Protect Cliff Walls at (E) Spillway/Splashpad
1	Cliff Wall Armour - Shotcrete	1,500	cy	$ 650	$ 975,000	pumped, fiber based, 4000 psi
2	Cliff Wall Armour - Rebar	75	tn	$ 2,000	$ 150,000	at 100#/cy
3	Drill/Grout 15' #10 Bars	80	ea	$ 1,000	$ 80,000	at 10' OC
8	Contract Allowances/NPIs	1	ls	$ 170,000	$ 170,000	Misc. Allowances
1	Build/Maintain US Dam Haul Road	1	ls	$ 35,000	$ 35,000	
2	Labor Support for Inspection Work	1	ls	$ 50,000	$ 50,000	
3	Rem/Repair Cracked Dam Concrete	1	ls	$ 75,000	$ 75,000	
4	Move/Relocate Structures in Parking/Staging Area	1	ls	$ 5,000	$ 5,000	
				Subtotal Direct Costs:	$ 5,895,000	
a.	Materials Sales Tax	1	ls	$	$ 292,000	8.25% on 60% of Subtotal of Direct Costs
b.	Escalation Allowance	1	ls	$	$ 715,000	1.65%/Qtr to Midpoint Construction (2nd Qtr '07)
c.	Subcontractor Bond Costs	1	ls	$	13,000	1.25% on 15% of Subtotal of Running Costs
d.	Genl Liability Insurance	1	ls	$	$ 35,000	0.50% on Running Subtotal
e.	GC Overheads & Profit	1	ls	$ -	1,043,000	15% on Running Subtotal; High Risk
f.	Performance & Payments Bonds	1	ls	$	68,000	0.85% on Running Subtotal
g.	Bldrs Risk Insurance	1	ls	$	$ 81,000	1.0% on Running Subtotal
h.	Contingency	1	ls	$	1,628,000	20% on Running Subtotal; High Risk, Delivery/Access Methods TBR
				Subtotal Indirect Costs:	$ 3,875,000	
				Grand Total Project Capital Costs.	$ 9,770,000	

I.	Engineering & Administration Costs	1	ls		$ 1,954,000	20% of Grand Total Project Capital Costs
II.	Geotechnical Investigation	1	ls		$	Not Required
				Grand Total Program:	**$ 11,700,000**	

Estimating Disclaimer - Engineer's Opinion of Probable Cost:

The estimate of costs shown and any resulting conclusions on the project financial, economic feasibility or funding requirements have been prepared from guidance in the project evaluation and implementation from the information available at the time the estimate was prepared. The final Costs of the project and resulting feasibility will depend on actual labor and material costs, competitive market conditions and other variable factors. Accordingly, the final project costs may vary from the estimate. Project feasibility, benefit/cost analysis, risk and funding must be reviewed prior to making specific funding decisions and establishment of the project budget.

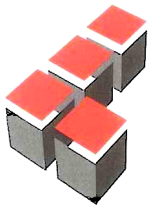
GEOTECHNICAL EVALUATION
SANTA ANITA SPREADING GROUNDS
EXISTING LEVEES WITHIN BASIN 12 AND BORROW PIT
1900± HIGHLAND OAKS DRIVE
ARCADIA, CALIFORNIA

Prepared For:

COUNTY OF LOS ANGELES
DEPARTMENT OF PUBLIC WORKS
GEOTECHNICAL AND MATERIALS ENGINEERING DIVISION
900 South Fremont Avenue, Fourth Floor
Alhambra, California 91803-1331

October 17, 2006

Project No. 601471-001



Leighton Consulting, Inc.
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October 17, 2006

Project No. 601471-001

County of Los Angeles Department of Public Works
Geotechnical and Materials Engineering Division
900 South Fremont Avenue, Fourth Floor
Alhambra, California 91803-1331

Attention Mr Amir M. Alam, P.E.
Senior Civil Engineer

Subject: Geotechnical Evaluation
Santa Anita Spreading Grounds
Existing Levees at Basin 12 and Borrow Pit
1900± Highland Oaks Drive
Glendora, California

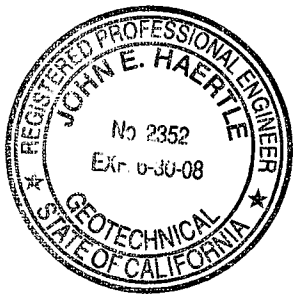
In accordance with our June 19, 2006 proposal, authorized on July 6, 2006, Leighton Consulting, Inc. presents results of our geotechnical evaluation of selected levees within Basin 12 and the Borrow Pit within the Santa Anita Spreading Grounds in Arcadia, in Los Angeles County

Results of this study indicate the factors-of-safety for the existing embankments analyzed along the western side of Basin 12 (Section A-A') and the western side of the borrow pit (Section B-B') exceed the typical minimum values of 1 5 for static conditions, and 1 1 for pseudo-static conditions under hydrostatic water conditions. However, the factors-of-safety during rapid drawdown conditions are below the minimum values.

The topographic survey indicates the depressions that exist within Basin 12 and the Borrow Pit have resulted in an increased slope height and inclination, thereby reducing the factor-of-safety The stability of the western embankment along Basin 12 as well as the extension of this embankment within the Borrow Pit can be improved by alteration of the slope inclination to a gradient of at 2.1 (H V) or flatter with grading conducted within the bottom of Basin 12 to establish a relatively consistent elevation of 668 feet MSL.

17781 Cowan ▪ Irvine, CA 92614-6009
949.253.9836 ▪ Fax 949.250.1114 ▪ www.leightonconsulting.com

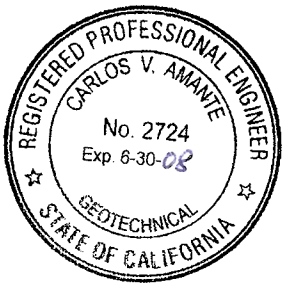
We appreciate the opportunity to be of service to the County once again. If you have any questions, or if we can be of further service, please call us at your convenience in our Irvine or Rancho Cucamonga offices at (949) 253-5922, extension 239, or (909) 484-2205, extension 125, respectively



Respectfully submitted,

LEIGHTON CONSULTING, INC.

John E. Haertle, GE 2352
Senior Project Engineer



Carlos V. Amante, GE 2724
Associate Engineer

JEH/CVA/
Distribution: (6) Addressee

Leighton

greater depths was not possible with a drilling rig using rotary drilling techniques and conventional hollow stem augers due to refusal on probable cobble and boulder material The relative density of the sediments that underlie the facility appeared to be medium dense to dense based on field testing, but these field results are expected to have been influenced by the presence of oversize material Evaluation of the liquefaction potential will required alternate exploration techniques utilizing specialized drilling equipment such as air rotary techniques or down-hole shear wave velocity measurements.

2.6.2. Seismically Induced Settlement: During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, unsaturated granular soils, which is not liquefaction. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement. The potential for seismically induced settlement is anticipated to be low

2.7 Slope Stability and Seepage Analyses

The slope stability of the western levee at selected locations along Basin 12 and the Borrow Pit of the facility was performed considering water levels within Basin 12 at the operational high water elevation and under rapid drawdown for static and pseudo-static (k_h=0.15) conditions. Review of the Operational Map for the facility indicates the operational water level for Basin 12 is at El. 673.0 feet, MSL, no water level was specified for the Borrow pit. Due to the lack of a specified water level, analysis was conducted for the Borrow Pit with water at El. 668 feet under static and pseudo-static conditions as was conducted for Basin 12 considering hydrostatic and rapid drawdown conditions. The slope stability analysis was performed using the SLOPE/W (GeoSlope, 2004) software package. Stability analyses were conducted considering rotational failure surfaces. Additional details of analyses as well as graphical output are presented in Appendix C.

Our models of the levee cross-sections were compiled from the boring data in conjunction with the topographic data presented in the topographic survey of the facility provided by the County Two cross sections were analyzed. Section A-A' through the western slope of Basin 12, and Section B-B' through the western slope of the Borrow Pit.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 Conclusions

Results of this study indicate the factors-of-safety for the existing embankments analyzed along the western side of Basin 12 (Section A-A') and the western side of the borrow pit (Section B-B') exceed the typical minimum values of 1.5 for static condtions, and 1.1 for

pseudo-static conditions under hydrostatic water conditions. However, the factors-of-safety are below the minimum values when subjected to rapid drawdown conditions.

Analyses of the existing slope and the recommended 2 1 (H V) slope configuration for both sections indicated a factor-of-safety less than 1 0 for pseudo-static conditions under rapid drawdown. Consequently, if Basin 12 (or the Borrow Pit) were to be filled to operational level at the time of a local major earthquake, conceivably on the Sierra Madre Fault just south of this site, and a levee failure were to occur, a rapid drawdown condition would result, exacerbated by ground shaking.

The potential for liquefaction to occur could not be evaluated due to the difficulty in extending the borings below a depth of 18 feet that was presumably due to the presence of oversize cobble and boulder material The presence of oversize material is considered to reduce the potential for liquefaction, but the actual liquefaction susceptibility depends upon the relative density of such materials. As previously discussed, exploration to allow liquefaction evaluation will require alternate exploration methods and techniques such as down-hole shear wave velocity measurements.

3.2 Earthwork

The topographic survey indicates a depression exists within Basin 12 and the Borrow Pit which resulted in an increase in slope height and inclination, thereby reducing the factor-of-safety The stability of the western embankment along Basin 12 as well as the extension of this embankment within the Borrow Pit can be improved by alteration of the slope inclination to a gradient of at least 2:1 (H.V) or flatter with grading conducted within the bottom of Basin 12 to an elevation of 668 feet MSL.

All earthwork should be performed in accordance with the *Earthwork and Grading Guide Specifications* presented in Appendix D, unless specifically revised or amended below or by future review of project plans.

3.2.1 Subgrade and Slope Preparation. Preparation of the subgrades prior to fill placement should include removal of any vegetation, disturbed soils, and other soils that are not considered to be suitable to serve as the subgrade for fill placement. Fill placement within the embankment(s) should be preceded by proper benching as described in the *Earthwork and Grading Guide Specifications*

3.2.2. Material Placement and Compaction. The prepared slopes and adjacent subgrades with the basins should be properly backfilled by the placement of suitable material that is free of debris, organic or other types of deleterious materials. The soils should be low in

expansion potential (EI $\leq$20). Soils to be placed as fill, whether onsite or import material, should be reviewed by Leighton Consulting, Inc., and tested as necessary The surface of the embankment should be scarified and recompacted as necessary to develop a uniform subgrade.

All fill soil should be placed in thin, loose lifts, sufficiently and uniformly moisture-conditioned at or within 1 to 2 percent above optimum moisture, and compacted to a minimum of 90 percent relative compaction. Relative compaction should be measured using the ASTM D 1557-02$^{\varepsilon 1}$ laboratory maximum-density test-method. Aggregate base should be compacted to a minimum of 95 percent relative compaction. The face of new slopes should be established by backcutting after overbuilding the slope by approximately 3 feet to allow compaction of the fill to be performed to planned slope face, or the actual face of slope should be specifically compacted using appropriate compaction equipment.

4.0 CONSTRUCTION CONSIDERATIONS

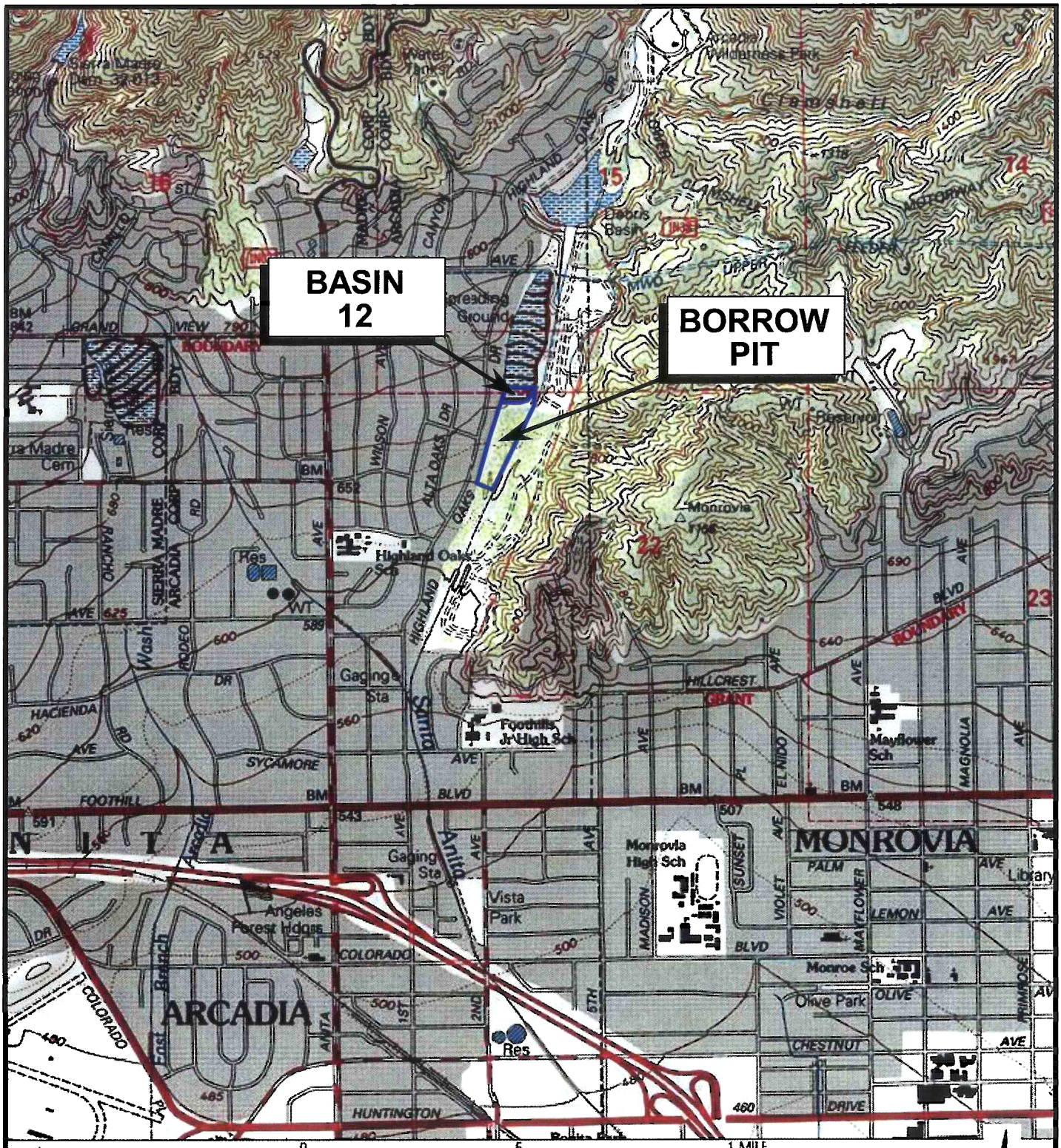
4.1 Temporary Excavations

The contractor is responsible for site safety, including all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing and benching should be performed by the contractor in accordance with the 2003 Edition of the *California Construction Safety Orders*, Article 6, or more current.

During construction, soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor is responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Soil types will vary, but Type C soils can be expected to prevail at the site. Close coordination between the competent person and Leighton Consulting, Inc. should be maintained to facilitate construction while providing safe excavations. Construction vehicles should not be allowed to drive within a horizontal distance from the top-of-slope less-than-or-equal-to ($\leq$) the height of the cut slope. For preliminary planning purposes, excavations are recommended to be sloped at a maximum inclination of 1½ 1 (horizontal·vertical).

4.2 Additional Geotechnical Services

Geotechnical recommendations presented in this report are based on subsurface conditions, as interpreted from limited subsurface explorations and limited laboratory testing. Leighton Consulting, Inc. should review the wall plans and specifications, when available, to comment on geotechnical aspects. Our recommendations should be revised, as necessary, based on future plans and incorporated into final-design plans and



Base Map: Mount Wilson, California 7.5-Minute Quadrangle

SITE LOCATION MAP

Levee Evaluation Basin 12 and Borrow Pit
Santa Anita Spreading Grounds
Arcadia, California

Project No.	601471-001
Scale (approx)	As Shown
Engr./Geol.	JEH/ELB
Drafted By	BQT
Date	9/06

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Figure 1

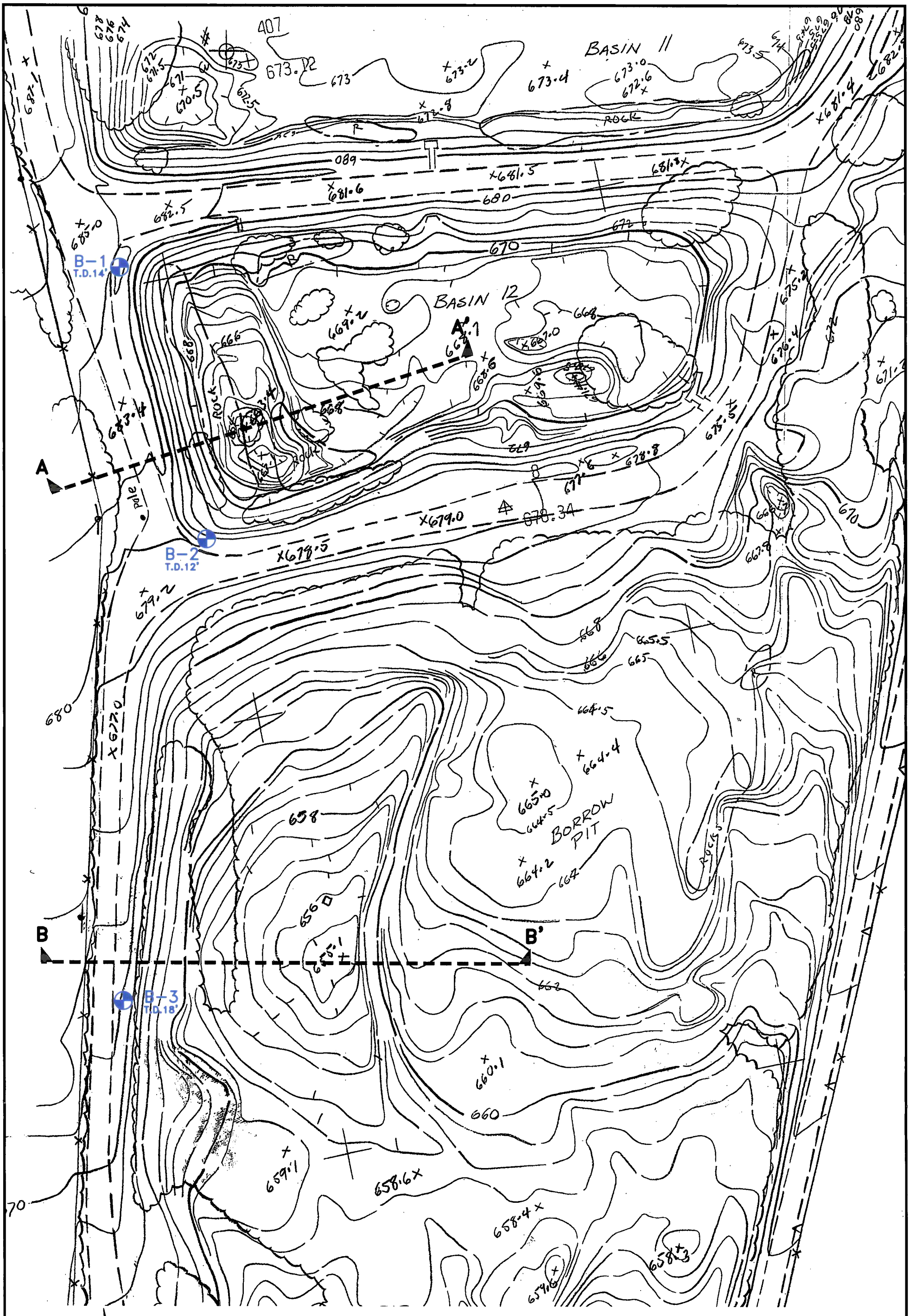

FIGURE 2

NOTE:
BORING MAP ADAPTED FROM TOPOGRAPHIC SURVEY (MAP 36–T30) PROVIDED BY LOS ANGELES COUNTY, DEPARTMENT OF PUBLIC WORKS (OCTOBER 31, 1986)

BORING LOCATION MAP
LEVEE EVALUATION BASIN 12 AND BORROW PIT
SANTA ANITA SPREADING GROUNDS
ARCADIA, CALIFORNIA

Leighton

Proj: 601471–001	Scale: 1"=40'	Date: 9/06
Eng/Geol: JEH	Drafted By: BQT	CP By: BQT

P:\DRAFTING\601471\001\OF_06-09-11\FIGURE2.DWG (12-01-06 9:04 20AM) Plotted by: btran

REPORT

WORK PLAN

SANTA ANITA DEBRIS DAM

Prepared For

Los Angeles County
Department of Public Works
900 South Fremont Avenue
Alhambra, California 91803

April 28, 2009

URS

URS Corporation
2020 East First Street, Suite 400
Santa Ana, California 92705-4032
Project No. 29871486

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Appendix

A Proposed Geotechnical Investigation Work Plan

Acronyms and Abbreviations

Department	Los Angeles County Department of Public Works
DSHA	deterministic seismic hazard analysis
DSOD	California Division of Safety of Dams
ft/s	feet per second
ft^3/s	cubic feet per second
km	kilometer
MCE	maximum credible earthquake
NGA	Next Generation of Ground-Motion Attenuation
RCC	Roller Compacted Concrete
URS	URS Corporation
USACE	U.S. Army Corps of Engineers, Los Angeles District

1.0 INTRODUCTION

1.1 PURPOSE, SCOPE AND ORGANIZATION

URS Corporation (URS) prepared this work plan for the Los Angeles County Department of Public Works (Department) to serve as a baseline plan for engineering studies required to investigate conditions at the damsite to the satisfaction of the California Division of Safety of Dams (DSOD), to identify and evaluate deficiencies and to develop alternatives to mitigate deficiencies.

The Table of Contents presents the overall report organization. A list of acronyms and abbreviations used in this document follow the Table of Contents and precede this section. After this introductory section, Section 2 provides a description of the facility, and Section 3 describes pertinent geologic and foundation conditions. Section 4 reviews DSOD's assessment of potential deficiencies and Section 5 presents a plan to address these deficiencies. References are listed in Section 6.

1.2 AUTHORIZATION

The preparation of this work plan was authorized by the Department by notice to proceed dated March 5, 2009 and Agreement "Design Services for the Big Dalton, Sawpit and Santa Anita Debris Dams Supplemental 2 of Contract No. PW70778" dated February 24, 2009

1.3 PREVIOUS STUDIES

Several seismic studies have been performed for the Santa Anita Debris Dam. The first assessments were performed by the U.S. Army Corps of Engineers Los Angeles District (USACE) in 1983 and 1987 Woodward-Clyde Consultants performed additional studies beginning in 1990, which were summarized in a 1994 report (Woodward-Clyde Consultants, 1994). Woodward-Clyde Consultants' study included field investigations and laboratory testing, and the analyses performed were simplified and conservative. In 2002, as part of the subject contract, URS was retained to provide additional investigation and design of remedial measures for Big and Little Dalton, Santa Anita and Sawpit debris dams. Based on the DSOD's concern about the integrity of the over-the-embankment spillway after an earthquake, a study of relocating the Santa Anita spillway was performed. This study looked at relocating the spillway to the left abutment. The DSOD reviewed this study and concluded the following with regard to using the basin for water conservation.

1 The field observation that led to the 1994 conclusion that the fault exposed on the left abutment does not "significantly displace" alluvium needs to be documented. The fault should be classified in accordance with the DSOD's Fault Activity Guidelines.

2. The previous slope stability analysis was based on limited data. A more thorough soil investigation to adequately assess the stability of the dam is needed. The potential for liquefaction of the embankment and foundation alluvium must be evaluated due to saturation from long-term storage in the reservoir

The Department has evaluated the feasibility of the spillway relocation and found that the negative environmental impacts are significant. Based on this, the Department wants to explore leaving the spillway at its current location and modifying it to make it acceptable for water conservation.

2.0 FACILITY LOCATION AND DESCRIPTION

The following information is provided in this section:

- Description of the facility location.
- General description of the facility, its history and current operation.
- A more detailed description of the embankment, spillway and outlet tower.

2.1 LOCATION AND COMMUNITY SETTING

Santa Anita Debris Dam is located near the mouth of Santa Anita Canyon on the south slope of the San Gabriel Mountains, in the City of Arcadia. The drainage area at the dam is 12.5 square miles.

The land to the west of the debris basin and dam is in the City of Arcadia and has been developed residentially, as shown on Figure 1, and the land to the east is undeveloped. This area consists of densely vegetated hills and canyons. There are two distinct fills near the dam. There is a larger fill about 900 feet downstream from the left (east) abutment of the dam that may be muck from construction of a tunnel along Metropolitan Water District of Southern California's Upper Feeder, or debris from Santa Anita debris basin. A smaller fill is located at the left abutment of the dam, extending over that abutment, apparently providing access to the debris basin from the downstream.

2.2 OVERALL DESCRIPTION AND OPERATION

Figure 2 presents a plan of Santa Anita Debris Dam Figure 3 presents the centerline profile and typical section of the dam. The dam has a maximum height of 56 feet and a length of approximately 955 feet. The spillway consists of a rectangular open channel founded within the embankment near the left abutment. The debris basin storage capacity is 116 acre-feet.

Santa Anita Debris Dam was designed and constructed by the USACE in the 1958-60 timeframe. The local sponsor was the Los Angeles County Flood Control District, now part of the Department, which assumed ownership of the dam in 1959. The primary function of the dam was debris retention, however, the Department has historically operated the facility for both debris retention and water conservation. The Santa Anita Spreading Grounds were constructed downstream of the dam for groundwater recharge.

Under the ownership of the Department, Santa Anita Debris Dam came within the jurisdiction of the DSOD. In 1982, the DSOD issued a Certificate of Approval for operation of the dam, contingent upon the USACE completing a satisfactory seismic safety evaluation.

The USACE conducted a Phase I safety evaluation of Santa Anita Debris Dam in 1983-85 as part of the National Dam Safety Assurance Program (USACE, 1985). The USACE concluded that, given the Department's then-current operating procedure, the dam embankment and foundation were not susceptible to general liquefaction or to significant loss of strength due to earthquake shaking. The USACE also concluded it was highly improbable that earthquake forces on the dam's appurtenant

structures would cause a catastrophic structural failure that would endanger lives or property downstream of the dam.

In their review of the USACE report, the DSOD did not fully concur with the USACE's conclusions, in part because data were inadequate. In response to DSOD comments, the USACE performed additional Phase I studies (USACE, 1987). However, the DSOD continued to have concerns regarding the adequacy of the data used by the USACE in developing its conclusions. Specifically, the DSOD remained concerned about the potential for seismically induced deformation of the embankment and spillway, and seismically induced failure of the intake tower.

From 1991 to 1994, Woodward-Clyde Consultants performed a seismic evaluation of Santa Anita Debris Dam. This seismic evaluation included performing additional field exploration and laboratory testing, developing design-level seismic ground motion parameters, performing analyses and developing remedial measures as necessary. The seismic evaluation considered using the dam and basin for water conservation purposes, which requires water storage, as distinguished from storm detention. The conclusions of that seismic evaluation are summarized as follows:

- The potential for surface fault rupture at the dam site appears to be low.
- There is a potential for seismically induced liquefaction of the dam embankment. The consequences of liquefaction would be limited cyclic mobility; that is, the response of the material would be a temporary reduction in shear strength to undrained residual values but would not result in flow-type behavior
- The potential for liquefaction of the foundation alluvium at the Santa Anita Debris Dam site is relatively low
- Post-seismic stability analyses of the dam indicate a minimum factor of safety of 1.6.
- Simplified and conservative seismic deformation analyses of the dam indicate seismic deformations of less than 1 foot at the dam crest.
- Post-liquefaction settlement of the dam embankment is estimated to be on the order of 6 inches.
- There is a potential for differential settlement of the spillway slab due to liquefaction-induced settlement of the underlying embankment. However, the magnitude of differential settlement is estimated to be less than 6 inches, or less than half the minimum spillway thickness, and the spillway would be expected to pass limited flows without additional damage following a seismic event.
- The spillway walls do not have sufficient moment capacity under seismic conditions.
- The intake tower is not stable under seismic loading.

In response to these findings, in 1995 the DSOD restricted the Department from continued operation of the project for water conservation. Alternative concepts were developed for mitigating the seismic hazards to the spillway and intake tower, with the goal to remove the imposed storage restriction.

2.3 EMBANKMENT

Figure 2 presents an embankment plan and Figure 3, a centerline profile and typical section. Santa Anita Debris Dam has a crest elevation of 796 feet and a maximum height (as defined by DSOD) of 56 feet. The dam has approximate crest length and width of 955 feet and 20 feet, respectively The upstream and downstream slopes are inclined at 2:1 (horizontal vertical). The dam is a zoned earth embankment with a central core, transition zones, shells and an outer upstream rock layer An exploratory trench was excavated along the core footprint; the exploratory trench side slopes were inclined at 2:1, in addition, the original ground surface under the remainder of the embankment was stripped to a depth varying from about 0.5 to 4 feet.

The core zone is 12 feet wide and vertical and extends from the stripped original ground surface to 5 feet below the crest. The core zone consists of silty and clayey sand that was borrowed from an area on the right bank of the debris basin within the older alluvium.

The transition zones are also 12 feet wide and vertical and extend over the top of the core as well as flanking it on both sides. The transition and shell zones consist of sandy and gravelly alluvial materials borrowed from within the debris basin.

The outer upstream rock layer has a minimum thickness of 3 feet measured perpendicular to the face. The material for this layer is described simply as "rock"

2.4 SPILLWAY

The spillway was designed and constructed by the USACE (1955a, 1955b, 1958). It consists of an ungated, concrete-lined, rectangular open channel, founded within the embankment near the left abutment, as shown on Figures 2 and 3. The spillway is approximately 160 feet wide at the spillway crest. The spillway crest elevation was determined by the debris storage requirement (374,000 cubic yards) and is at approximately Elevation 774.31 feet. The spillway was designed to pass the design flood peak discharge of 38,000 cubic feet per second (ft^3/s) with a peak water surface at Elevation 792.8 feet (USACE, 1955a). The dam crest being at Elevation 796.0 feet, the minimum freeboard is 3 feet. The approach channel is riprap-lined and the spillway chute is lined with concrete. The spillway slab is underlain by pervious materials consisting of a drain layer over a filter layer. The select pervious zone is 2.5 feet thick upstream of the core zone and 4 feet thick downstream of the core zone. The core zone extends to the bottom of the spillway slab to control seepage under the spillway

For analysis and preliminary design purposes, the configurations and dimensions of the spillway shown on the plans prepared by the USACE (1955b, 1958a, 1959) and summarized above will be assumed, unless more recent survey data indicate different as-built conditions. Spillway remediation designs will

be based on the original maximum probable flood estimates (38,000 ft^3/s) and minimum residual freeboard of 1.5 feet.

2.5 INTAKE TOWER

The low-level outlet works consists of a gated intake tower, a gated outlet conduit and a diversion structure that were as designed and built by the USACE (USACE, 1955a, 1958b, 1958). The intake tower is 31.6 feet high and has a 4-foot by 5-foot rectangular interior opening. The 355.4-foot long, 48-inch inside diameter outlet conduit has a design capacity of 260 ft^3/s. The outlet conduit elevation is 748.9 feet at the tower intake and 737.9 feet at the spillway

The outlet works were designed to: (1) completely drain the debris basin after each storm and (2) divert clear water into downstream spreading grounds. For draining the debris basin the outlet conduit discharges through openings in the spillways walls.

For final analysis and design purposes, the configurations and dimensions of the outlet works shown on the plans prepared by the USACE (1955a, 1958b, 1958) will be assumed, unless more recent survey data indicate different conditions. If it is necessary to remediate or replace any element of the outlet works system, designs would be based on the original design criteria and the rated capacities of the systems, stated above. Remediation or redesign of an intake tower would include provisions for restricting debris from entering the outlet system.

3.0 GEOLOGIC AND FOUNDATION CONDITIONS

3.1 REGIONAL GEOLOGY

Santa Anita Debris Dam is located at the southern margin of the San Gabriel Mountains, which is part of Southern California's Transverse Ranges physiographic and geologic province. This province, which is one of the most seismically active regions in California, is characterized by tectonic compression across east-west-trending reverse and strike-slip faults. This tectonic setting is believed to be caused by the bend in the San Andreas fault zone north of Los Angeles.

The Sierra Madre fault, a reverse fault that is considered capable of producing an earthquake with a maximum moment magnitude of 7.5, is located about 300 meters (1000 feet) south of the dam, with the dam being on the hanging wall. The San Andreas fault, a strike-slip fault that is considered capable of producing an earthquake with a maximum moment magnitude of 8.0, is located about 33 kilometers northeast of the dam. These and other active faults near the dam are considered capable of producing significant seismic shaking at the dam site. Many other faults are located within 200 kilometers of the site, but based on the distance and maximum moment magnitude of these two faults and other faults in the region, one of these two faults will give the controlling ground motion parameters for seismic analyses using ground motion parameters derived from a deterministic seismic hazard analysis.

3.2 LOCAL GEOLOGY

The Santa Anita Debris Dam is located at the mouth of the Santa Anita Canyon. The canyon walls are composed of granitic bedrock. Near surface bedrock is moderately to highly weathered and very closely fractured by joint sets and numerous minor shears. Alluvial materials overlie the granitic bedrock along the canyon floor Alluvial materials consist of older terrace deposits and recent streambed deposits of the Santa Anita Wash.

3.3 FOUNDATION CONDITIONS

As shown on Figure 3, the dam embankment is founded on alluvial materials across the streambed and on the right abutment, and on granitic bedrock on the steep left abutment. The right abutment of the dam is a gently sloping alluvial terrace composed of older alluvium. The terrace alluvium consists chiefly of reddish brown silty sand and clayey sand with occasional gravel (USACE, 1985, Woodward Clyde Consultants, 1994).

The central (stream channel) portion of the embankment, in the vicinity of the spillway, is founded on recent alluvium. The recent alluvium consists primarily of poorly graded sand with silt and gravel, poorly graded gravel with silt and sand, and silty sand with gravel (USACE, 1985, Woodward Clyde Consultants, 1994).

The left abutment of the dam is a steeply sloping bedrock ridge. The ridge is mostly covered by colluvial debris and thick vegetation. The bedrock is typically slightly to moderately weathered and very closely

fractured by multiple joint sets. The USACE (1985) mapped a fault in the granitic rock of the left abutment. The fault was described as a narrow zone of gouged and crushed rock near the south edge of the rock spur The fault has a reported strike of N40W and a dip of 60 degrees northeast. Woodward-Clyde (1994) concluded that the fault in the abutment was not active; this fault will not be considered in the deterministic seismic hazard analysis.

The depth to bedrock beneath the dam ranges from zero at the left abutment to an unknown depth at the right abutment. Based on seismic refraction surveys and borehole data, Woodward Clyde Consultants (1994) suggested that the top of bedrock ranges from 0 to 50 feet deep beneath the dam and the upper bedrock is characterized by shear-wave velocities ranging from 2000 to 2900 ft/s and compression-wave velocities ranging from 5000 to 12,500 ft/s.

The groundwater within the alluvium varies seasonally The USACE (1985) concluded that the groundwater table is at or near the ground surface within the basin during the wet season (November through April) and up to 18 feet deep during the dry season (May through October). Rotary wash boring RW-SA2 (Woodward-Clyde Consultants, 1994) indicated that the elevation of the groundwater measured on May 24, 1991 was 2 feet below the dam/alluvial foundation interface, or at Elevation 745.5 feet.

4.0 DSOD Assessment of Potential Deficiencies

DSOD has issued four letters that are relevant to current status of Santa Anita Debris Dam.

4.1 1987 DSOD Letter

In a letter to the Department dated February 26, 1987 (DSOD, 1987), the DSOD stated that it had reviewed 1985 USACE report and concluded that there was insufficient data provided in the investigation to rule out liquefaction or severe deformation under a Maximum Credible Earthquake (MCE). Further, the USACE report did not address the consequence of a catastrophic failure of the appurtenant structures under the MCE event with a full reservoir The DSOD requested a phased investigation program, and suggested the Department "look into" using a Becker Hammer for the investigation and evaluation of the gravelly silty sand in the dam and dam foundation.

4.2 1994 DSOD Letter

In a letter to the Department dated September 29, 1994 (DSOD 1994), the DSOD stated:

> *We concur with WCC's [August 1994] recommendation for the reinforcement of the outlet tower and spillway walls. We recommend the use of the response spectra method for analysis of the tower. Please submit the final selected response spectra curves to this Division for review prior to the design of the repair.*

Thus, the DSOD envisions the Department providing seismic parameters for design and recommends use of a response spectrum method for the tower analysis. After approval of the seismic parameters, the DSOD expects the Department to prepare a plan to reinforce the intake tower and the spillway walls.

4.3 1995 DSOD Letter

In a letter to the Department dated June 7, 1995 (DSOD, 1995), the DSOD stated:

> *Our contact with the Corps of Engineers and Mr. Sree Kumar of your staff indicated that the repair of Santa Anita Debris Basin Dam is unlikely for the next several years. The dam safety deficiencies reported by Woodward-Clyde Consultants [report dated August 3, 1994] necessitate a change in the present mode of operation.*

In this letter DSOD is less specific than in the previous letter and refers simply to "dam safety deficiencies reported by Woodward-Clyde Consultants" Those deficiencies are:

- There is a potential for failure in bending at the base of the spillway walls due to seismic shaking loads. However, the potential failure can be mitigated with buttresses without loss of spillway capacity

- There is a potential for moment-induced failure of the intake tower due to seismic shaking loads, as well as sliding and overturning of the tower foundation block. As a result, there is

likely to be a loss of control of flow through the outlet conduit following a seismic event. However, the potential tower failure and the concerns for potential foundation sliding and overturning can be mitigated with buttresses.

Woodward-Clyde Consultants also reported two other minor deficiencies that were judged unlikely to lead to an uncontrolled release of reservoir storage and hence did not appear to require remediation. These minor deficiencies are:

- The potential for liquefaction of the alluvial foundation materials at the dam site is low.
- There is a potential for liquefaction-induced deformations of the dam embankment during a seismic event. However, the potential deformations of the embankment are estimated to be less than 1 foot, and are not likely to cause a catastrophic failure of the embankment.
- There is a potential for differential settlement of the spillway slab due to liquefaction-induced settlement of the underlying embankment. However, the magnitude of differential settlement is estimated to be less than 6 inches, or less than half the minimum spillway thickness, and the spillway would be expected to pass limited flows without additional damage following a seismic event.

Woodward-Clyde Consultants also concluded that the potential for fault rupture to intersect the dam embankment, spillway or outlet works appeared unlikely

Although DSOD did not appear to take issue at the time with Woodward-Clyde Consultants' conclusions about the fault in the left abutment, it does so in a letter a decade later, which is discussed in the following section.

4.4 2005 DSOD LETTER

In a meeting with the DSOD on July 7, 1998 the DSOD, Department and URS discussed URS' design criteria report (URS 1998). The DSOD expressed concern about settlement/deformation and the integrity of the spillway Specifically, the DSOD indicated that the spillway would be severely damaged as a result of the embankment deformation during the MCE and that this would lead to a dam failure after a small flood. URS was tasked with performing a study to investigate spillway relocation concepts to satisfy DSOD concerns (URS 2004).

In a letter to the Department dated February 8, 2005 (DSOD, 2005), DSOD review comments on the relocation study stated:

> *The following items need to be addressed if the basin is [to be] converted to permanent storage:*
>
> *A) The field observation that led to the 1994 conclusion that the fault exposed on the left abutment does not "significantly displace" alluvium needs to be documented. The fault should be classified in accordance with DSOD's Fault Activity Guidelines.*

B) The previous slope stability analysis was based on limited data. A more thorough soil investigation to adequately assess the stability of the dam is needed. The potential for liquefaction of the embankment and foundation alluvium must be evaluated due to saturation from long-term storage in the reservoir.

The Department has since evaluated the spillway relocation concept and found that from an environmental standpoint it is not feasible. Therefore, the Department has tasked URS with performing studies to address DSOD concerns, specifically the concern that the spillway would be severely damaged as a result of the embankment deformation during the MCE and that this would lead to a dam failure after a small flood.

4.5 SYNTHESIS OF DSOD LETTERS

It is URS's understanding that the outstanding DSOD issues at this time are:

1. Stability of the spillway walls.
2. Stability of the intake tower
3. DSOD classification of fault observed in the left abutment.
4. Material characteristics, especially the potential for liquefaction of the granular soils making up the embankment and its foundation.
5. Stability of the embankment; deformation potential.
6. Integrity of the spillway slab, under the assumption that a significant flood could flow through the spillway before any damage could be repaired and that this would result in a dam failure.

Of these issues, this work plan essentially addresses the last four The first two issues will depend somewhat on the conclusions drawn from this study and are therefore deferred at this time. It is anticipated, however that the spillway walls and intake tower will be remediated by structural buttresses similar to what was done at Little Dalton Debris Dam.

The Department intends to address the fault, the data limitation and embankment stability and deformation by performing additional investigations and analyses. The Department intends to address the spillway integrity issue by developing either structural or ground improvement mitigation of the existing spillway to maintain the integrity of the spillway during and after the design earthquake.

5.0 PLAN TO ADDRESS DSOD CONCERNS

5.1 GENERAL

This work plan is intended to serve as a baseline plan for engineering studies required to investigate conditions at the dam site to the satisfaction of the DSOD, to identify and evaluate deficiencies and develop alternatives to mitigate deficiencies. The Department seeks to adopt a phased approach to the work, so that the DSOD has an opportunity to review the engineering at each stage. The major stages are: criteria, data collection and basic data evaluation, engineering analyses, conception and evaluation of mitigation measures, selection of preferred mitigation measures and preliminary design.

5.2 ADDITIONAL DATA COLLECTION

The Geotechnical Investigation Work Plan in Appendix A presents a work plan to collect additional geologic and geotechnical information.

5.3 ANALYSIS CRITERIA AND METHODS FOR EMBANKMENT

This section presents criteria and analysis methods for· water levels, fault parameters, level of shaking (median or 84th percentile), minimum factors of safety, allowable deformation of embankment and spillway, overturning for outlet tower

5.3.1 Reservoir Water Surface and Phreatic Surface

The County wishes to operate the debris dam for water conservation purposes and thus seeks the flexibility to maintain the normal water surface elevation at the spillway crest whenever practical; therefore, for the conceptual and preliminary design analyses, the water surface elevation in the debris basin will be assumed to be at the spillway crest.

The previous studies performed by Woodward-Clyde Consultants included the development of a steady-state phreatic surface in the dam embankment, assuming the reservoir water surface to be at the spillway crest. Figure 3 shows this phreatic surface profile at a typical section in the stream channel. Geotechnical analyses performed as part of conceptual and preliminary design will also be based on the previously developed phreatic surface profile.

5.3.2 Ground Motion Analysis and Fault Parameters

As required by the DSOD, ground motion parameters will be developed in a deterministic seismic hazard analysis (DSHA). Considering the maximum credible earthquakes (moment magnitudes) for the many faults within 200 kilometers of the dam, and the distances of these faults from the dam, two faults may yield the controlling ground motion parameters for seismic analyses using ground motion parameters derived from a deterministic seismic hazard analysis. These two faults and relevant fault parameters are summarized in the following table and will be used in the DSHA.

6.0 REFERENCES

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Chiou, B. and Youngs, R., 2008." An NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra" *Earthquake Spectra 24* (1), February, pp. 173-215.

DSOD (Division of Safety of Dams), 1987 "Santa Anita Debris Basin, No. 32-29, Los Angeles County" Letter from J.J. Doody, Chief, DSOD, to T.A. Tidemanson, Director, Los Angeles County Department of Public Works. February 26.

Initial and more refined analyses will be performed and preliminary cost estimates prepared to evaluate the different options. The analyses, reasoning and preferred option will be presented to the DSOD for review

5.4.2 Analyses of Spillway Mitigation Concepts

The data from the proposed additional exploration will be combined with the previous existing data to create a geotechnical model of the conditions beneath the spillway slab. In order to validate and refine the preliminary spillway mitigation concepts discussed, advanced analyses will be undertaken using numerical modeling software, FLAC. FLAC is a two-dimensional[2] explicit finite difference program for geotechnical engineering and rock mechanics computations. It offers a wide range of capabilities to solve complex problems in geomechanics, including slope stability, groundwater flow, nonlinear static and dynamic stress-strain analysis of soil continua and soil-structure interaction. FLAC has been thoroughly verified against closed-form solutions, physical models and field-testing. The specific benefit of FLAC for the proposed analyses is that the spillway structure and soil conditions can be modeled to obtain a more realistic assessment of the behavior of the spillway-soil system. For example the spillway walls, slab, spillway wall buttresses, slab thickening, piles and improved soils can be modeled in FLAC to evaluate the integrity of the spillway both during and after the earthquake. In this way the stress, strain, deformation and cracking of the slab and can be evaluated.

The seismic deformations will be addressed with fully nonlinear effective-stress analyses. Time domain, nonlinear analyses will be performed using a Mohr-Coulomb (linear elastic/perfectly plastic) soil model coupled with an empirical porewater pressure generation scheme. Porewater pressures are generated in response to shear stress cycles, following Seed's cyclic-stress approach. However, unlike the standard cyclic-stress approach where liquefaction potential is assessed as a post-processing step, porewater-pressure generation is fully integrated with the dynamic analysis.

Porewater pressures are continuously updated for each element in response to shear stress cycles. Effective stresses decrease with increasing porewater pressure and a state of liquefaction may be approached for frictional materials. As the available shear strength decreases, increments of permanent deformation are accumulated. In addition, the plastic strains generated as a result of increased porewater pressures significantly contribute to the internal damping of the modeled earth structure. The simultaneous coupling of porewater pressure generation with the stress analysis results in a more realistic dynamic response of the model.

This framework, based on the Mohr-Coulomb soil model coupled with incremental porewater pressure generation, has been employed on many projects involving dynamic deformation analysis of dams, analysis of dynamic soil-structure interaction of wharf structures, and predictive modeling of dynamic centrifuge tests.

Embankment settlements due to volumetric strains induced by strong earthquake shaking will be estimated directly within FLAC using procedures such as those by Tokimatsu and Seed (1987). FLAC's macro-programming language, FISH, will be utilized for this purpose.

[2] A three-dimensional version of FLAC, FLAC3D, also exists. However, three-dimensional effects are not expected to be significant for this project.

In the case of Santa Anita Debris Dam, the buttress concept is feasible for the upstream slope of the embankment. Buttressing the downstream slope may be feasible but would require reconstruction of the spillway and a portion of the downstream chute. The upstream buttress could include details such as an impermeable layer to lower the phreatic surface within the embankment. In this regard the upstream buttress could mitigate both the seismic-induced deformations and limit the post-seismic settlement by reducing the potentially liquefiable soils (by lowering the phreatic surface). It may be possible to remove some of the upstream embankment materials and recompact them during the buttress construction, further reducing the liquefaction potential. However, even with the reduction in phreatic surface it may not be possible to limit the downstream deformations without a downstream buttress. Buttress materials can be earth fill, RCC or soil cement. The use of RCC is unlikely because the foundation conditions would be alluvium.

Soil improvement is another feasible option. Typical soil improvement options for liquefaction include dynamic deep compaction, vibro-replacement, vibro-compaction, compaction grouting, jet grouting and permeation grouting. The materials requiring soil improvement are very coarse grained and include gravel, cobbles and boulders, they are also relatively dense with $(N_1)_{60}$ values of 25 Because of this, dynamic compaction, vibro-replacement, vibro-compaction and jet grouting are considered infeasible. Permeation grouting and limited compaction grouting may be feasible. The benefit of permeation grouting would be to mitigate the liquefaction potential but also increase the strength of the material, potentially further limiting seismic-induced deformations. Permeation grouting could be performed by drilling vertical holes, installing sleeve-port grout pipes and then grouting with microfine cement grout under low pressures. Compaction grouting is also an option in finer-grained zones. The probable zone of permeation grouting is illustrated on Figure 4.

Various methods of controlling the phreatic surface include an upstream impermeable blanket and a slurry cutoff wall. It appears that the central impermeable core zone is functioning properly; therefore, focus would be on providing an upstream blanket coupled with a slurry cutoff wall at the upstream toe. This could be effective in lowering the phreatic surface and therefore mitigating the liquefaction potential in the embankment. Because of the depth and coarseness of the alluvium, a slurry wall may have to be deep, and would be difficult to construct. Another limitation is that even mitigating the potential for liquefaction may not reduce the seismic deformations to acceptable levels because of the very high acceleration that will likely be required for design.

Another option is to provide structural elements such thickening the spillway slab or adding piles below the spillway slab to firm materials. The concept of thickening the spillway slab and adding piles is illustrated on Figure 5. Because of the coarse nature of the soils, pile driving is not considered feasible. It may be possible to construct cast-in-drilled-hole piles. They would need to be designed to support the spillway slab and the down drag due to liquefaction. The piles can also limit deformation if considered as reinforcing elements in the dam.

Based on the discussion above, two mitigation concepts are recommended for further consideration. These are the soil improvement by permeation grouting option shown on Figure 4 and structural mitigation with slab thickening and piles as shown on Figure 5

zoning, remaining freeboard, and embankment slopes would all be considered before final decisions on whether a 5 to 10 foot deformation is acceptable without structural modifications.

Deformations greater than ten feet are considered to be in nearly the same class as embankments with post-seismic factors of safety less than unity. DSOD does not consider that there is precedent for predicting the final configuration or embankment integrity for dams sustaining deformations in excess of ten feet. Structural modifications will in all likelihood be required.

It should be noted that the preceding DSOD guidance was issued before nonlinear analyses became well developed and the engineer's ability to evaluate the effects of large ground deformations is certainly more developed now than it was in 1996.

For Santa Anita Debris Dam, URS recommends as an acceptance criterion that the estimated permanent seismically induced deformation be less than or equal to 3 feet; for comparison, the normal freeboard and residual freeboard are nominally 23.25 and 3.25 feet, respectively

Post-liquefaction settlement of the embankment will be evaluated using the procedure of Idriss and Boulanger (2008), Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992).

5.3.6 Geotechnical Parameters

Geotechnical parameters have been evaluated in the past for use in previous analyses (Woodward-Clyde Consultants, 1994). For this study URS will reevaluate geotechnical parameters using previous data and data obtained in the current study (Section 5.2, Appendix A) and new correlations that were not available at the time the previous parameter evaluation was completed. The geotechnical parameters include moist and saturated unit weight, static shear strength and post-liquefaction residual shear strength (for liquefied zones only).

5.4 Evaluation and Analyses of Spillway Mitigation Concepts

5.4.1 Preliminary Evaluation of Spillway Mitigation Measures

The DSOD has indicated that the primary concern regarding the over-the-embankment spillway is that the spillway will be severely damaged as a result of embankment deformations caused by the MCE event and that subsequent floods over the severely damaged spillway would cause failure of the dam. The embankment deformations and severity of damage would be exacerbated by the potentially liquefiable materials identified in the embankment, below the spillway slab. The following paragraphs discuss ground improvement methods to mitigate the liquefaction potential and reduce deformations as well as structural mitigation measures to ensure the integrity of the spillway to operate after a design earthquake.

Historic approaches to mitigating potential liquefaction-induced instability of embankment dams have included: buttressing the embankment; soil densification (or other soil improvement); various means of controlling the development of a phreatic surface within the embankment; removing the dam from service (i.e., breaching the dam) and replacing the dam. The following paragraphs discuss these options in more detail.

- Pseudostatic analysis of failure in both the upstream and downstream directions to obtain the yield acceleration[1] for use in simplified deformation evaluation. The phreatic surface will be the same as in the steady seepage analysis.

- The stability analysis will include circular trial failure surfaces analyzed using Bishop's Modified Method of Slices. When non-circular trial failure surfaces are analyzed, one of the methods of slices that considers all three equilibrium conditions (Spencer, Janbu's Generalized Method, Morgenstern and Price, Fredlund's Generalized Limit Equilibrium Method) will be used.

- In all cases, pore-water pressure will be based on simple distance below phreatic surface, thus neglecting seepage pressures.

5.3.4 Liquefaction

The potential for embankment materials and foundation alluvium to liquefy will be evaluated using the procedure developed by Idriss and Boulanger (2008). The moment magnitude for the analysis is as given in Section 5.3.2 and the peak horizontal ground acceleration will be calculated in the deterministic seismic hazard analysis in a future task.

5.3.5 Seismic Deformation and Post-liquefaction Settlement

Seismic performance of the dam will be evaluated using the Makdisi-Seed method (Makdisi and Seed, 1978) or Bray-Rathje method (Bray and Rathje, 1998). The yield acceleration will be computed using the critical trial failure surface from the static analysis (Seismic Slope Stability Task Force, 1988) and the dynamic yield strength [80 percent of the static shear strength of the soil] (Kramer, 1996). If any zones of the embankment or foundation are evaluated as being potentially liquefiable, post-seismic undrained residual shear strength will be used for those zones. The undrained residual shear strength will be evaluated using Idriss and Boulanger (2007, 2008).

DSOD (Babbitt and Verigin, 1996) has issued some guidance for allowable embankment deformations:

> *Conservative judgment is exercised by DSOD in reviewing all deformation analyses including those done in-house. Deformations of 0 to 5 feet are considered sustainable provided the deformation is not too large a percentage of the total dam height and do not seriously compromise freeboard. A 0 to 5 foot deformation computation on a large dam could actually be a prediction of nothing more than structural cracking under heavy seismic loading. Computations will almost always predict some deformation if a dam is subject to peak ground accelerations exceeding about 0.35g.*
>
> *Deformations of 5 to 10 feet are considered serious. As the number approaches ten feet DSOD does not believe all related structural behavior is predictable. Transverse cracking, especially at the abutments, is one concern that has been observed but not effectively calculated. Other types of embankment cracking and local slumping are also deemed possible. Freeboard, crest-width,*

[1] The yield acceleration is the acceleration that results in a factor of safety equal to 1.0 in a pseudostatic analysis

characteristics (moment magnitude, style of rupture, depth of rupture, directivity effects and distance from the site) that are as similar as possible given the existing database of recorded events. The recorded acceleration-time histories will be spectrally matched to the average response spectrum using the approach described by Silva and Lee (1987). It involves combining a Fourier amplitude spectrum (which is generated by matching a target spectrum) with a phase spectrum from an actual strong motion record. The software program RSPMATCH Version 2.2 (or later) written by Norm Abrahamson, which implements this method, will be used.

Moment magnitude for Sierra Madre fault - A MCE is defined by the California Geological Survey as "the largest earthquake that is capable of being produced from a source, structure, or region, under the currently known tectonic framework. It is a rational and believable event which can be supported by all known geologic and seismologic data. An MCE is determined by judgment considering the geologic evidence of past movement and the recorded seismic history of the area." We adopt this definition in this evaluation.

The Sierra Madre fault system forms an approximately 60 kilometer long, east-west trending, north dipping structural zone along the south side of the San Gabriel Mountains, along which the mountains have been uplifted. Earlier studies estimated that the MCE that could occur on the Sierra Madre fault zone was moment magnitude 7, based on the grounds that the fault probably ruptures in approximately 15 km long segments (Crook et al., 1987). However, more recent studies (Dolan et al., 1995, Rubin et al., 1998, Zucker and Dolan, 2001, Anderson et al., 2003), have suggested that the Sierra Madre fault is capable of generating larger than magnitude 7 earthquakes involving rupture of multiple segments of the fault. Based on our review of all these studies we believe that the Sierra Madre fault is capable of generating a MCE of moment magnitude 7.5.

5.3.3 Conventional Stability Analyses and Minimum Factors of Safety

Once the additional field data are collected (see Section 5.2), the subsurface conditions and geotechnical models will be updated. Stability of the embankment will be evaluated using conventional limiting equilibrium stability analysis procedures. The analyses will be performed using the SLOPE/W component of GeoStudio version 7 13 (or later) from Geo-Slope International and/or UTEXAS4 from Shinoak Software. The analyses and required minimum factors of safety are:

- Static analysis of failure in the downstream direction assuming the steady seepage phreatic surface. Minimum required factor of safety is 1.5.

- Static analysis of failure in the upstream direction assuming a rapid drawdown from the normal water surface elevation (spillway crest) to Elevation 750.0 feet. Minimum required factor of safety is 1.25.

Fault	MCE[1] Moment Magnitude, Mw	Depth to Top of Rupture Surface (km)	Fault Type	Dip of Fault Surface (degrees)	Down-dip Rupture Width (km)	Closest Horizontal Distance from Top Edge of Rupture (km)	Joyner-Boore Distance[2] (km)
Sierra Madre[3]	7.5	0	Reverse	45	18	0.3	0
San Andreas	8	0	Strike-Slip	90	12	33	33

Notes: (1) MCE means maximum credible earthquake. (2) The Joyner-Boore distance, Rjb, is the closest horizontal distance from the dam to the projection of the rupture surface onto the ground surface. (3) The dam is on the hanging wall of the Sierra Madre fault.

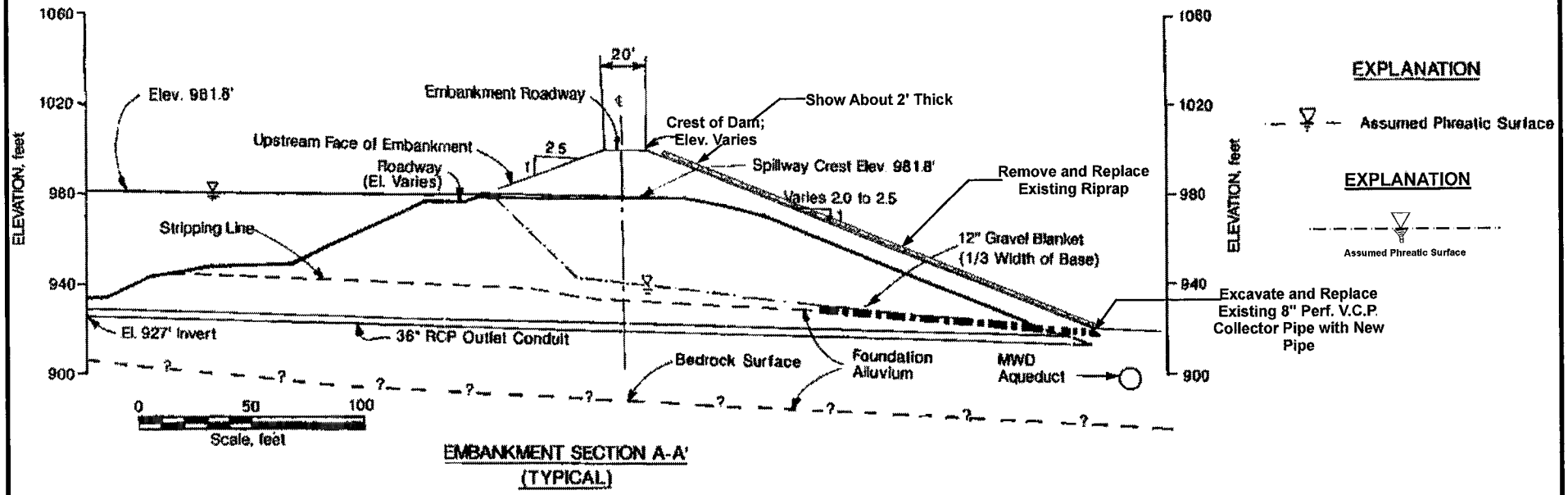
The median plus one standard error term (84th percentile) ground motion parameters are considered appropriate to assess the performance of the dam because of its high hazard classification if the basin is used for water conservation. Although two faults would be considered in the DSHA, it is expected that the Sierra Madre fault will yield the controlling MCE rather than the San Andreas fault due to the proximity of the former to the dam (closest inclined distance of 0.3 km versus 33 km) and expected higher ground motion levels. For this reason, if the Sierra Madre pseudoacceleration response spectrum is markedly higher than the San Andreas', acceleration-time histories would be developed only for the Sierra Madre fault MCE.

The peak horizontal ground acceleration and pseudoacceleration response spectrum with 5 percent damping will be developed by using five Next Generation of Ground-Motion Attenuation (NGA) relations developed by Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008), Chiou and Youngs (2008) and Idriss (2008). An average pseudoacceleration response spectrum for use in analyses will be computed by averaging the spectral ordinates at each period and connecting the ordered pairs of spectral pseudoacceleration and period with a straight line (on either a semilogarithmic or logarithmic plot).

Spectral Adjustment for Rupture Directivity Effects At periods longer than about 0.5 second, near-fault ground motions can be strongly influenced by the effect of rupture propagating along the fault. This rupture directivity effect causes ground motions to be larger in the direction normal to the fault (fault-normal component) and smaller in the direction parallel to the fault (fault-parallel component) (Somerville et al., 1997).

The Sierra Madre fault MCE would occur at a small distance from the dam. However, for reverse faulting (which is the case of the Sierra Madre fault), the effects of rupture directivity are already implicitly included in the ground motion models for the average horizontal component since all sites along the top edge of the fault will be subject to this effect. Thus, additional adjustments for rupture directivity would not be necessary unless direction/component-specific ground motion levels are required. If directivity is considered, it will be done using the procedure developed by Somerville et al. (1997).

Acceleration-Time Histories – To support the FLAC analyses, a minimum of three acceleration-time histories will be developed by spectrally matching appropriate records from earthquakes with relevant

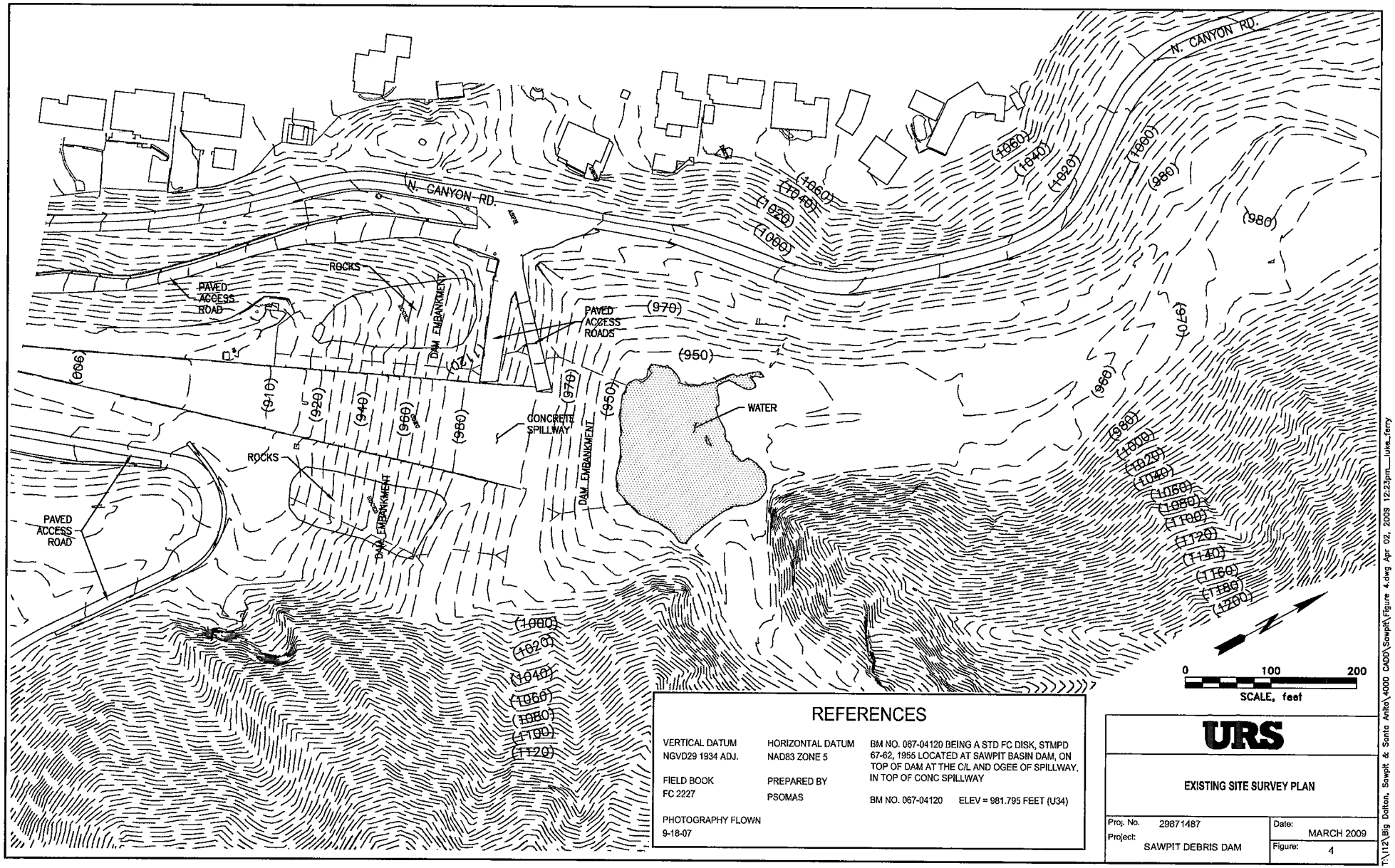


EMBANKMENT SECTION A-A'
(TYPICAL)

URS

SECTION SHOWING RIPRAP AND BLANKET DRAIN COLLECTOR PIPE

Project No. 29871487 10000	Date April 2009
Project: SAWPIT DEBRIS DAM	CAD ID:
	Figure: 13



REFERENCES

VERTICAL DATUM NGVD29 1934 ADJ.	HORIZONTAL DATUM NAD83 ZONE 5	BM NO. 067-04120 BEING A STD FC DISK, STMPD 67-62, 1955 LOCATED AT SAWPIT BASIN DAM, ON TOP OF DAM AT THE C/L AND OGEE OF SPILLWAY. IN TOP OF CONC SPILLWAY
FIELD BOOK FC 2227	PREPARED BY PSOMAS	BM NO. 067-04120 ELEV = 981.795 FEET (U34)
PHOTOGRAPHY FLOWN 9-18-07		

URS

EXISTING SITE SURVEY PLAN

Proj. No. 29871487	Date: MARCH 2009
Project: SAWPIT DEBRIS DAM	Figure: 4

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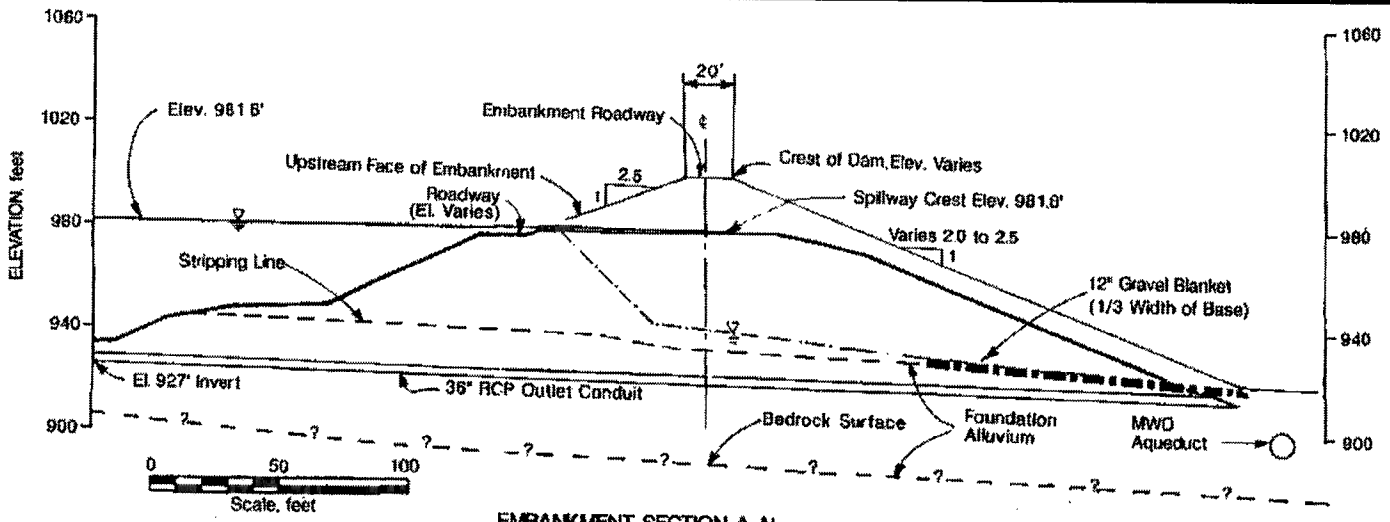
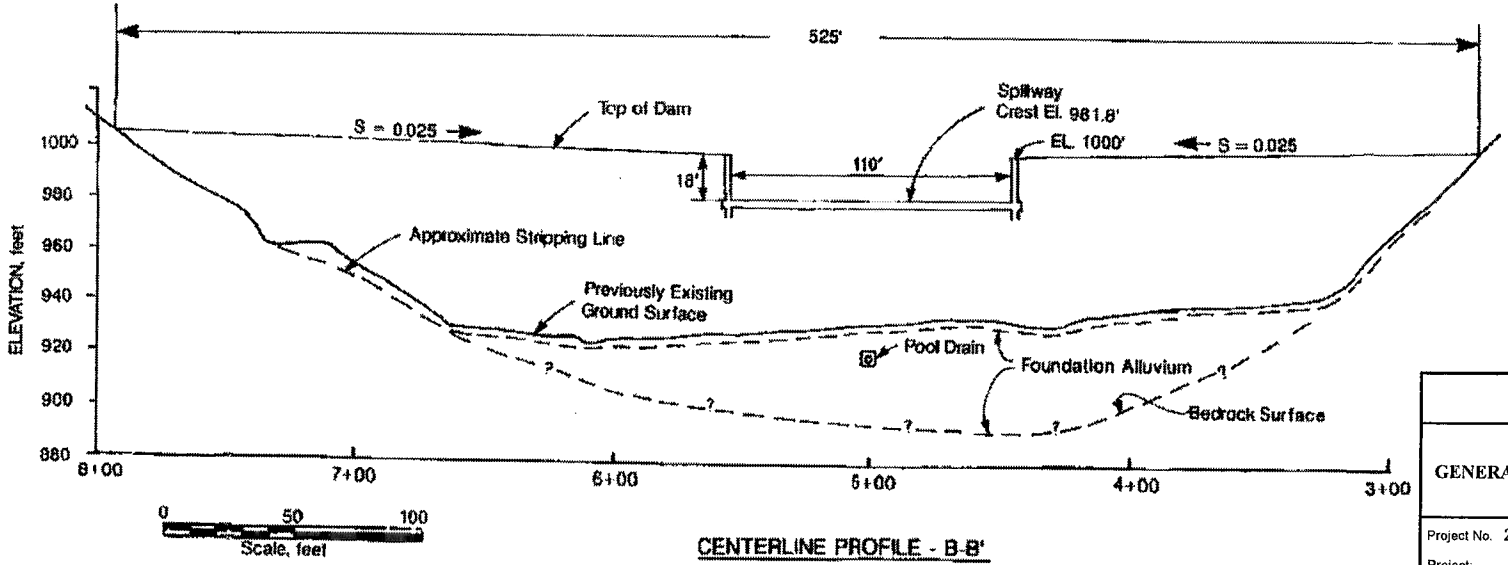


EMBANKMENT SECTION A-A'
(TYPICAL)

EXPLANATION

Assumed Phreatic Surface

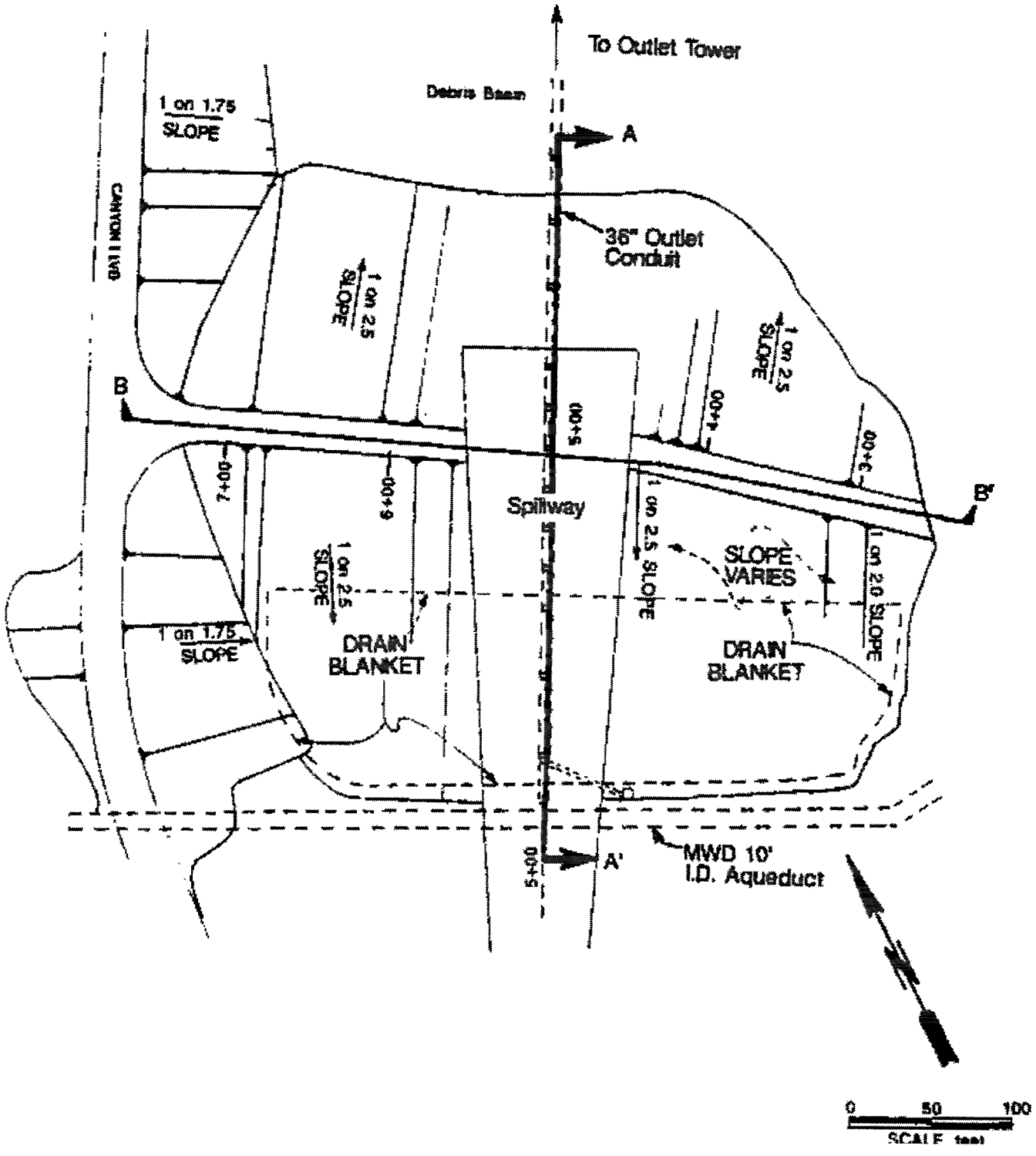


CENTERLINE PROFILE - B-B'
LOOKING UPSTREAM

URS

GENERALIIZED SECTION AND PROFILE

Project No.	29871487 10000	Date	Dec 2008
Project:	SAWPIT DEBRIS DAM	CAD ID:	
		Figure:	3



For Section A-A' and B-B', See Figure 3

Source: I:\Big_Dalton_Sawpit_wa\mxd\Sawpit_Plan_Embankment.mxd

PLAN OF EMBANKMENT				URS
Project No 29871487.10000	Date December 2008	Project	SAWPIT DEBRIS DAM	Fig. 2

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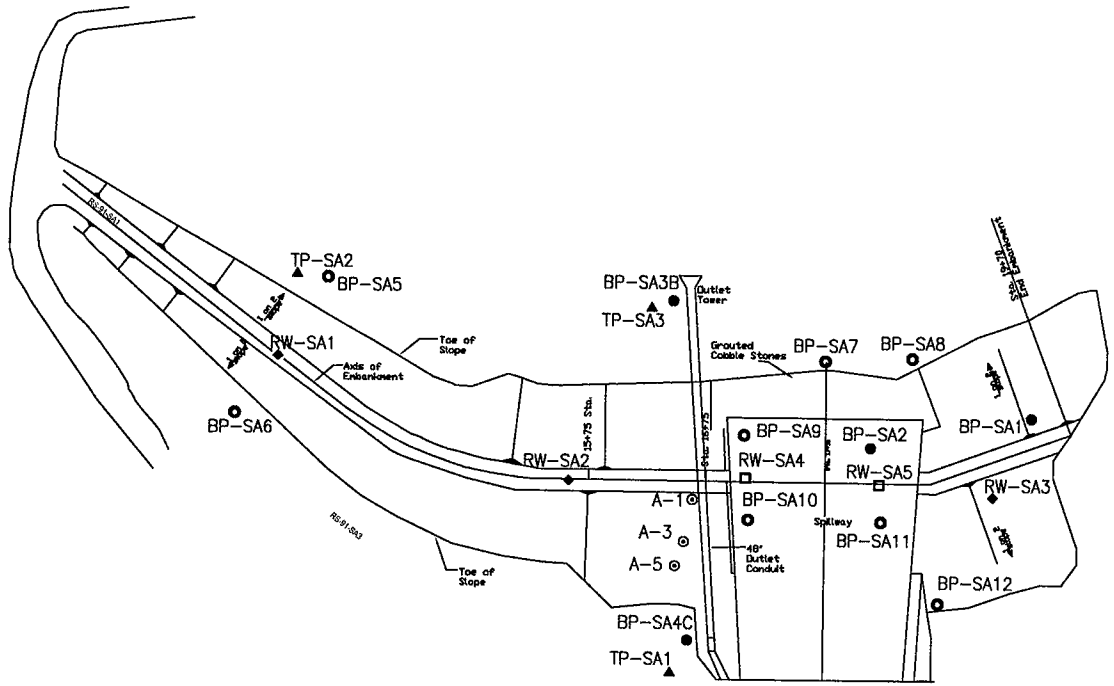
Appendix A

Geotechnical Investigation Work Plan



LEGEND

- ● BP-SA3B (Approximate Location of Previous Becker Penetration Test)
- ▲ TP-SA2 (Approximate Location of Previous Test Pit)
- ◆ RW-SA2 (Approximate Location of Previous Rotary Wash Boring)
- ⊙ A-1 (Approximate Location of Previous Existing Standpipe Piezometer)

Proposed Exploration

- ○ BP-SA7 (Approximate Location of Proposed Becker Penetration Test)
- □ RW-SA4 (Approximate Location of Proposed Rotary Wash Boring)

0 100 200

SCALE, feet

URS

PROPOSED SUBSURFACE FIELD EXPLORATION

Proj. No. 29871486
Project: SANTA ANITA DEBRIS DAM
Date: December 2008
Figure: A-2

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Proposed Scope of Work for Aerial Photograph Analysis, Geologic Field Mapping, Drilling, and Laboratory Testing

The geotechnical investigation will include an aerial photograph analysis, a geologic field mapping program, a field drilling program, and laboratory testing.

Aerial Photograph Analysis & Geologic Field Mapping (Fault Investigation)

During construction of the dam the Army Corps of Engineers (COE) mapped a fault in the Cretaceous age granitic rock that comprises the left abutment (Figure A-1). The fault, which had a reported strike of N40°W and a dip of 60° northeast, was not mapped beyond the immediate exposure of the left abutment, and does not appear on published geologic maps of the area. In its 1994 report, Woodward Clyde Consultants concluded that this fault was not active based on its observation that older fluvial terrace deposits, probably of late Pleistocene age, overlie the fault with no significant displacement. Concerning this conclusion, Woodward Clyde Consultants offered no supporting evidence for this observation. As noted above, the DSOD requested that the conclusion that this fault does not significantly displace alluvium be documented and that the fault be classified in accordance with DSOD's fault activity guidelines.

URS believes that trenching investigations to substantiate the conclusion that the fault does not cut the fluvial terrace deposits would not be feasible because the potentially promising trench sites are located in residential developments that have been graded with artificial fill. URS therefore proposes to perform a noninvasive investigation consisting of aerial photograph analysis, and geologic mapping. URS believes that these investigations will enable it to more rigorously document the geologic observation that older alluvium overlies the fault. URS has in its library high quality stereographic photographs of the project vicinity from 1928, 1938, and 1945. These images, which were obtained from the Fairchild Aerial Photograph Collection, show the area prior to residential development and are therefore ideal for assessing the geomorphic expression (or lack thereof) of the fault. In particular the 1945 photographs, which are at a scale of 1 inch = 600 feet, are particularly high quality photographs of the project area. URS will review these photographs, as well as 1930s vintage topographic maps, and will perform geologic reconnaissance/mapping of the area to assess whether or not the fault has any geomorphic expression in the Quaternary deposits that are west of the dam. URS will also be inspecting exposures along slopes and roadcuts within the area of geologic reconnaissance shown on Figure A-1, to assess whether the left abutment fault is part of a through-going map scale fault, or just a minor bedrock shear with limited lateral extent typical of the fractured rockmass that exists along the San Gabriel Mountain front.

Based on the presence or absence of geomorphic expression in the Quaternary deposits, URS will provide their opinion regarding the activity of the fault. URS will classify the fault as active, conditionally active or inactive, in accordance with the DSOD fault activity classification guidelines. URS will document its findings in a report signed by a State of California Certified Engineering Geologist. The report will include geologic

maps and annotated aerial and land-based photographs to substantiate its principal conclusions.

Drilling Investigations

In order to obtain additional subsurface information and perform supplemental liquefaction and slope stability analyses, the following field explorations, at the locations shown on Figure A-2, are proposed.

A minimum of eight Becker Penetration Test (BPT) sounding (BP-SA5 through BP-SA12) to depths of 80 feet or refusal, whichever comes first, with installation of standpipe piezometers in borings BP-SA6 and BP-SA12 (located at the downstream toe). The BPT tests will be performed within shell zones and penetrate into native alluvium. URS will obtain required permits from County of Los Angeles DHS for piezometer construction.

Two rotary wash borings (RW-SA4 and RW-SA5) will be drilled to a maximum depth of 80 feet (or refusal, whichever comes first) with Standard Penetration Tests (SPTs) and Modified California samples at 2-to 5-foot intervals. These borings will be drilled through the core and into the underlying native alluvium, and will be sampled and tested in accordance with appropriate ASTM standards.

In preparation of this work plan the following assumptions were made:

• URS will core through the spillway slab at locations shown on Figure A-2 to allow the Becker Penetration Test soundings and rotary wash borings to be made through the spillway slab.

• The drill cuttings will be left at the site, at a location designated by the Department, and all the borings will be backfilled with a bentonite/cement slurry backfill. The top of the borings made on the spillway slab will be backfilled with concrete.

• Permission to access the site, and clear access for our personnel and equipment. In addition, it is our understanding that weekend work is not required. Furthermore, clearance for access to the points of exploration for drill rigs will be provided by Department personnel and equipment. Based on our recent field visit some roads are in poor condition and some dozer work will be required to repair the roads.

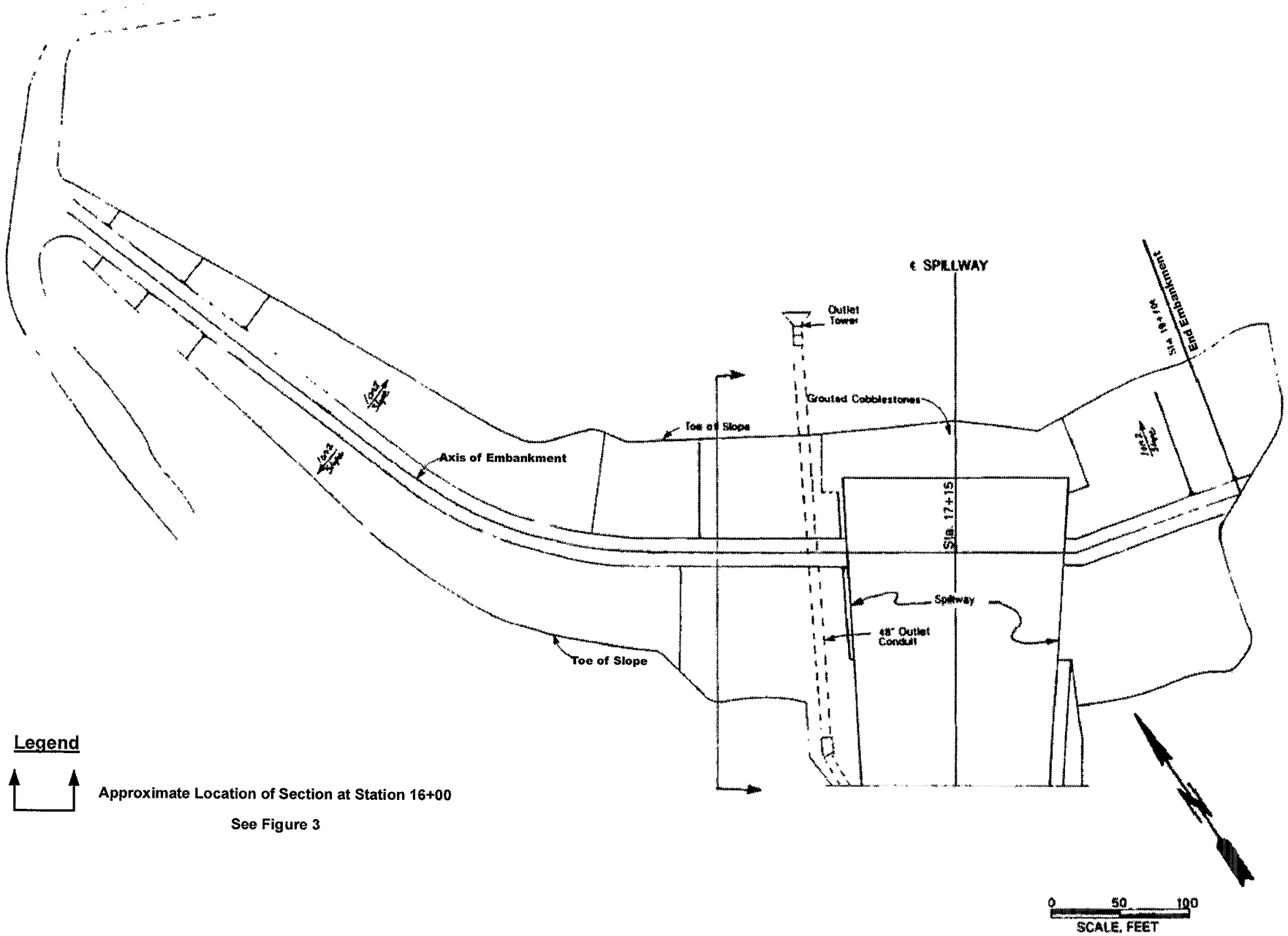
• URS will make reasonable efforts to locate surface and subsurface utilities in an effort to prevent accidental damage to utility lines during subsurface exploration activities. However, due to the location of the site, Underground Service Alert may not properly identify the locations of the utility lines; therefore, it is assumed that the location of all exiting utility lines will be identified by the Department. The Department assumes all responsibility for any utilities damaged during our field investigation provided URS drills at locations that have been approved by the County

• URS will plan to perform the field work during dry weather to minimize damage to embankment surface from traffic. However, if DSOD has concerns that the "traffic" from the drilling equipment will damage the lower portion of the embankment during the rainy season, then URS will postpone the field drilling program until there is a dry period based on weather forecast.

• URS' scope of investigation is limited to the engineering properties of the onsite soils and this scope excludes investigation or treatment of hazardous materials/wastes. In preparation of this proposal URS has assumed that such materials are not present at the site and that evaluation, treatment, and special handling of any such materials, if encountered, are not included in our scope of services.

Laboratory Testing

Particle size distribution analyses, Atterberg limits, direct shear, in-situ density and water content tests will be performed on selected samples obtained from the rotary wash borings.



EMBANKMENT PLAN

URS

Project No: 29871486.10000	Date: Jan 2009	Project: SANTA ANITA DEBRIS DAM	Figure: 2

Source: Topographic Map from Imagis Connect Mar 2008

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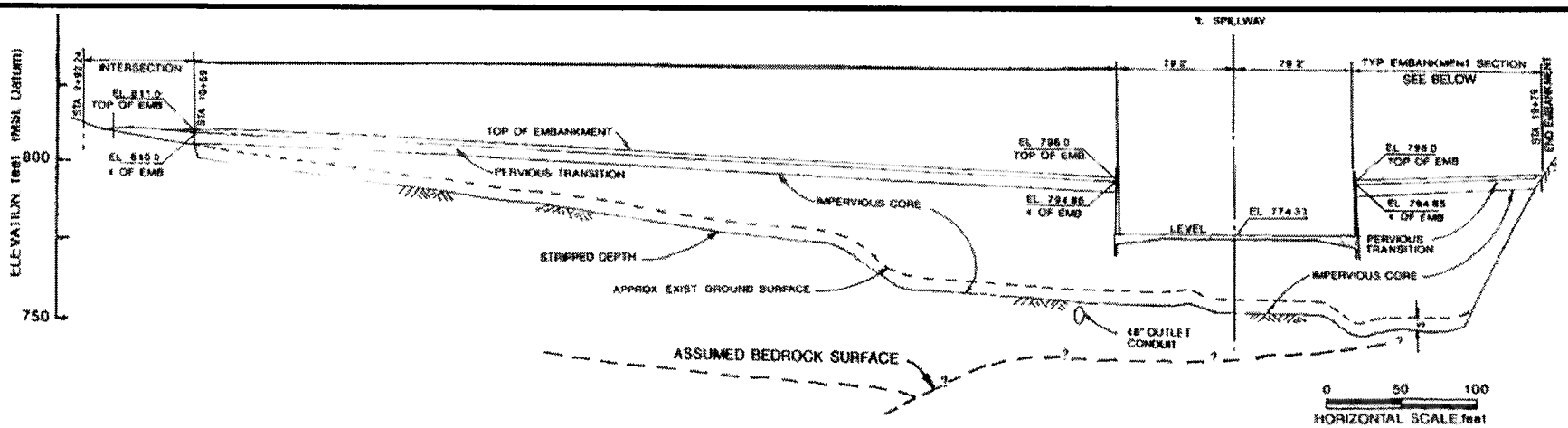
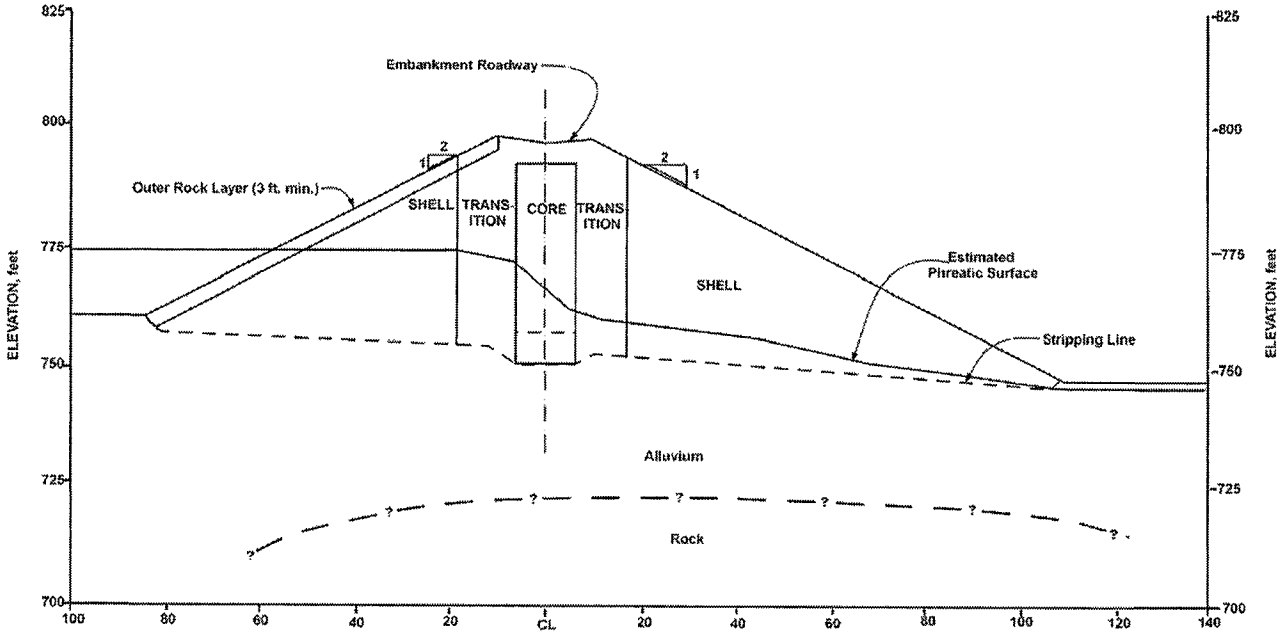


PROFILE ON AXIS OF EMBANKMENT LOOKING UPSTREAM

These Scales Apply to the Above Profile



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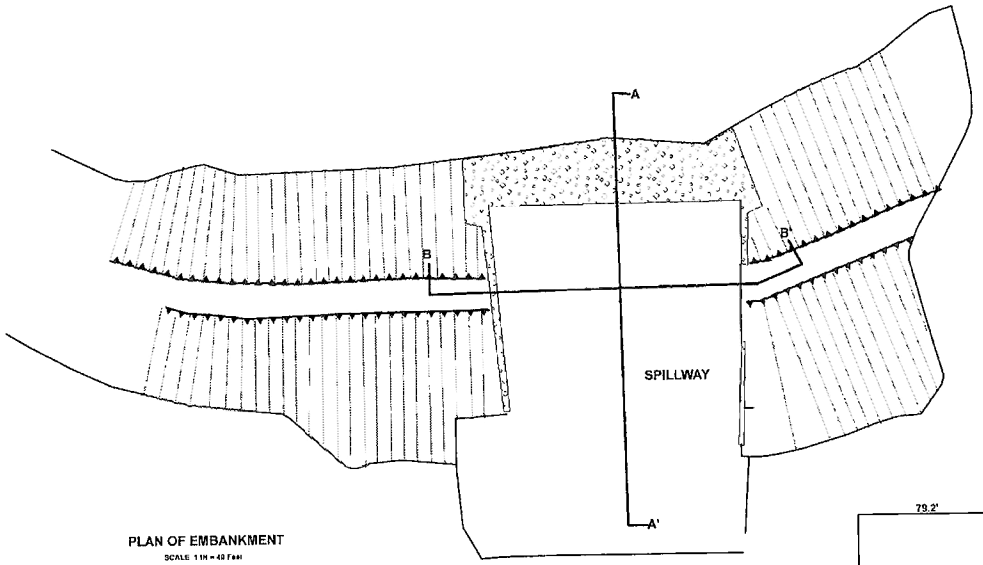
CENTERLINE PROFILE AND TYPICAL SECTION

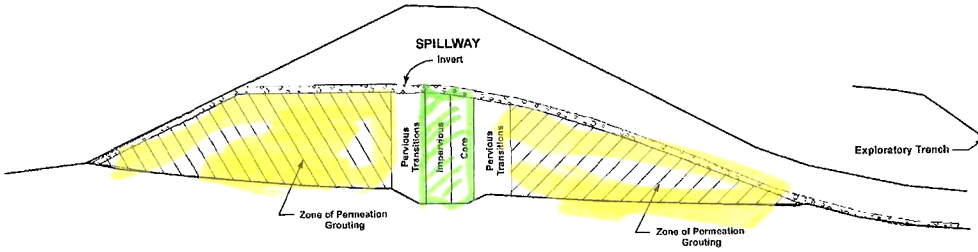
Project No. 29871486.10000	Date April 2009
Project: SANTA ANITA DEBRIS DAM	CAD ID:
	Figure: 3



PLAN OF EMBANKMENT

SCALE 1 IN = 40 Feet

40 0 40 80 Feet

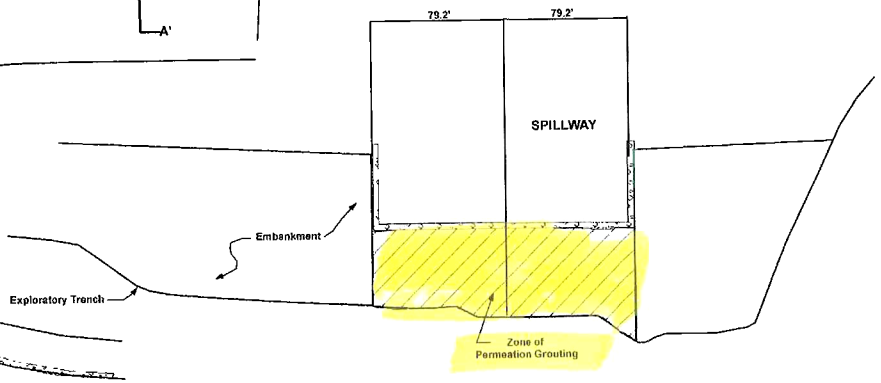


SECTION ON CENTER LINE OF SPILLWAY

SCALE 1 IN = 20 Feet

20 0 20 40 Feet

SECTION A - A'



PROFILE ON AXIS OF EMBANKMENT

SCALE: HOR 1 IN = 40 Feet
VERT 1 IN = 20 Feet

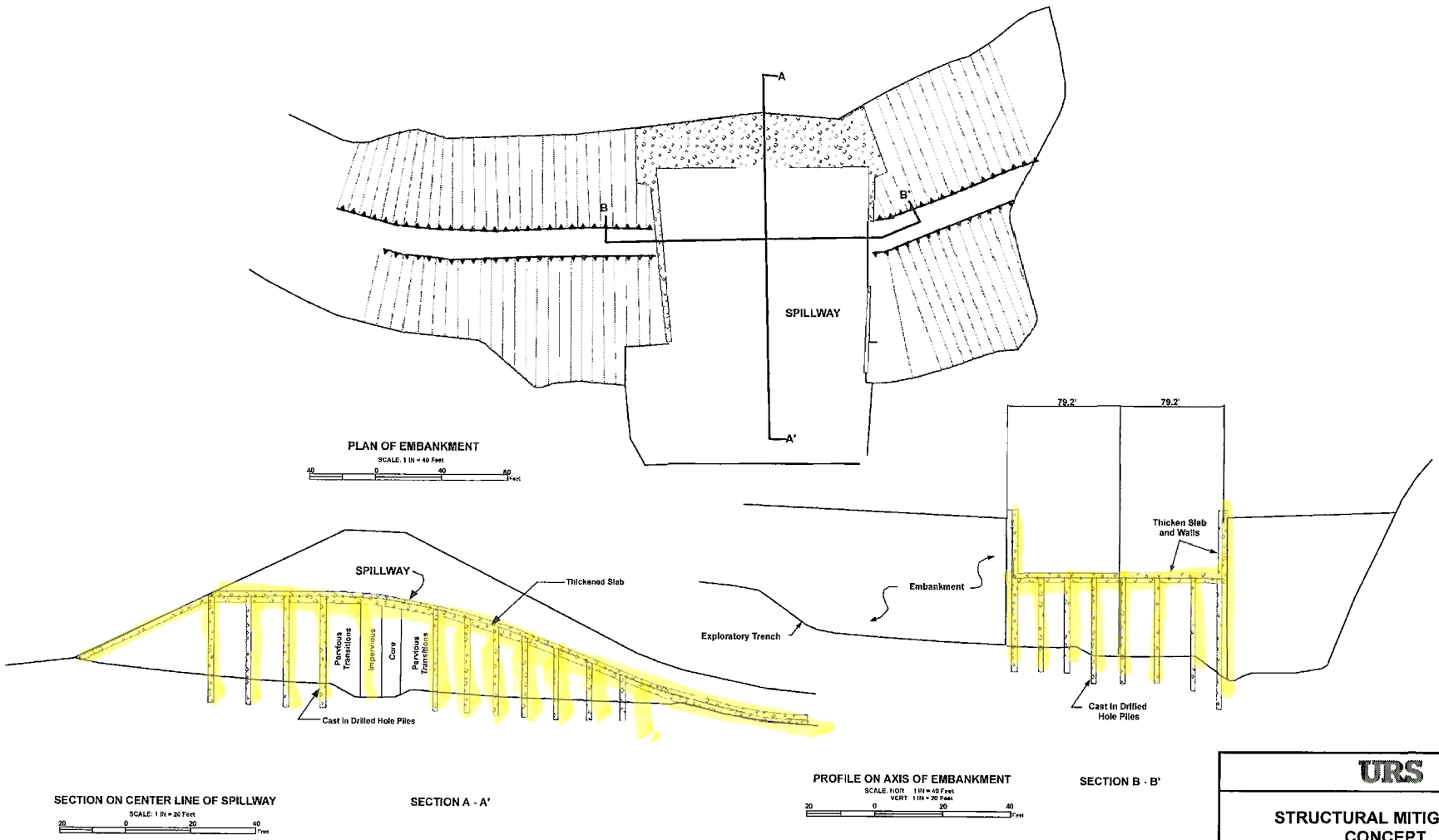
20 0 20 40 Feet

SECTION B - B'

URS

GROUND IMPROVEMENT MITIGATION CONCEPT

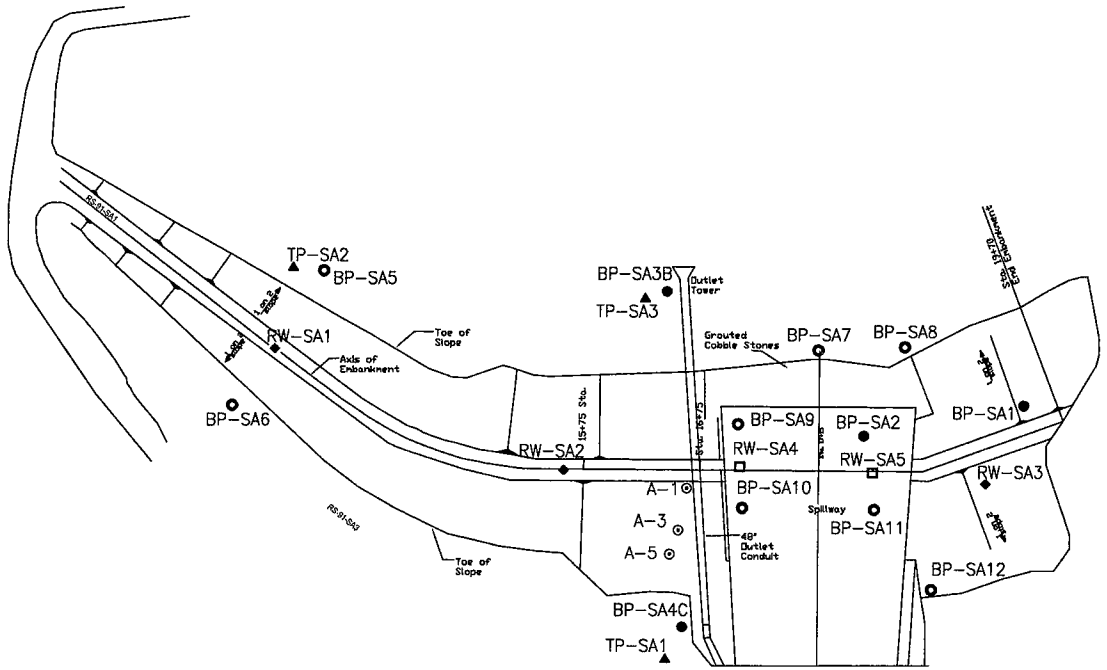
Project No. 29871486.10000	Date April 2009
Project SANTA ANITA DEBRIS DAM	CAD ID:
	Figure. 4



URS

STRUCTURAL MITIGATION CONCEPT

Project No.: 29871486.10000	Date: April 2009
Project: SANTA ANITA DEBRIS DAM	CAD ID
	Figure: 5



LEGEND

- ● BP-SA3B (Approximate Location of Previous Becker Penetration Test)
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- ⊙ A-1 (Approximate Location of Previous Existing Standpipe Piezometer)

Proposed Exploration

- ○ BP-SA7 (Approximate Location of Proposed Becker Penetration Test)
- □ RW-SA4 (Approximate Location of Proposed Rotary Wash Boring)

0 100 200

SCALE, feet

URS

PROPOSED FIELD EXPLORATION

Proj. No.	Date: June 2008
Project: SANTA ANITA DEBRIS DAM	Figure: 1

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Scope of Work

Task 1 – Work Plan

The work plan will include a map showing the locations of proposed Becker Penetration Tests (BPTs) and rotary wash borings, a figure showing the approximate location of the fault identified by the COE and the intended area for the field reconnaissance, proposed models and loading assumptions, and scope and schedule of work. After review by the Department, it will be submitted and presented to the DSOD. The plan will be updated, modified or changed to address DSOD comments.

Task 2 – Field Investigation

The field investigation will consist of a geological field mapping program follow by a field drilling program.

2a) Geologic Field Mapping:

During construction of the dam the Army Corps of Engineers (COE) mapped a fault in the Cretaceous age granitic rock that comprises the left abutment. The fault, which had a reported strike of N40°W and a dip of 60° northeast, was not mapped beyond the immediate exposure of the left abutment, and does not appear on published geologic maps of the area. In its 1994 report, Woodward Clyde Consultants concluded that this fault was not active based on its observation that older fluvial terrace deposits, probably of late Pleistocene age, overlie the fault with no significant displacement. Concerning this conclusion, Woodward Clyde Consultants offered no supporting evidence for this observation. URS will further evaluate and define what was meant by "no significant displacement" in their conclusions as well as fully explain other deductions from their observations (e.g. if Pleistocene age deposits are displaced, it indicates movement within Pleistocene time). However, in their 2004 report titled "Santa Anita Debris Dam, Spillway Relocation Concepts" URS again reiterated and advocated this conclusion that that the fault was not active. URS will document their findings in a report signed by a State of California Certified Engineering Geologist. The report will include geologic maps and annotated aerial and land-based photographs that satisfy the DSOD's requirements which were stated on Feb 8, 2005 letter to substantiate its principal conclusions.

URS believes that trenching investigations to substantiate the conclusion that the fault does not cut the fluvial terrace deposits would not be feasible because all the potentially promising trench sites are located in residential developments. We therefore are proposing to perform a noninvasive investigation consisting of geologic mapping and aerial photograph analysis, which will enable URS to more rigorously document the geologic observation that older alluvium overlies the

fault. URS has in their library high quality stereographic photographs of the project vicinity from 1928, 1938, and 1945. These images, which were obtained from the Fairchild Aerial Photograph Collection, show the area prior to residential development and are therefore ideal for assessing the geomorphic expression (or lack thereof) of the fault. URS will review these photographs, as well as 1930s vintage topographic maps, and will perform geologic reconnaissance of the area to assess whether or not the fault has any geomorphic expression in Quaternary deposits. Based on the geomorphic expression of the fault in Quaternary deposits, URS will provide their opinion regarding the activity of the fault. URS will classify the fault as active, conditionally active or inactive, in accordance with the DSOD fault activity classification guidelines.

In order to obtain additional subsurface information and perform supplemental liquefaction and slope stability analyses, the following field explorations (at the locations shown on attached Figure 1) are proposed for material characterization:

2b) Core drilling through spillway slab at locations of 5 borings that are proposed within the spillway as detailed in items 2c and 2d below

2c) A minimum of eight Becker Penetration Tests (BPTs) holes (BP-SA5 through BP-SA12) to depths ranging from 80 feet or refusal, whichever comes first) with installation of standpipe piezometers in borings BP-SA6 and BP-SA12, which will be located along of downstream slope. The BPT tests will be performed within shell/transition native alluvium. URS will obtain required permits from Los Angeles County of Los Angeles DHS for piezometer construction.

2d) Two rotary wash borings (RW-SA4 and RW-SA5) to a maximum depth of 80 feet (or refusal, whichever comes first) with Standard Penetration Tests (SPTs) at 2-to 5-foot intervals. These borings will be drilled through the core and into the underlying native alluvium, and will be sampled and tested in accordance with appropriate ASTM standards.

In preparation of this proposal and cost estimate, the following assumptions were made:

• This investigation is subject to Prevailing Wage Law as set forth under the California Labor Code, and the costs associated with drilling by subcontractors are based on this assumption.

• The drill cuttings will be left at the site and all the borings will be backfilled with a bentonite/cement slurry backfill.

• We have permission to access the site, and clear access for our personnel and equipment will be provided In addition, it is our understanding that a permit is required for drilling, and that weekend work is not required. Furthermore,

clearance for access to the points of exploration for drill rigs will be provided by County personnel and equipment. Based on our recent field visit some roads are in poor condition and some dozer work will be required to repair the roads.

• We will make reasonable efforts to locate surface and subsurface utilities in an effort to prevent accidental damage to utility lines during subsurface exploration activities. However, due to the location of the site, Underground Service Alert may not properly identify the locations of the utility lines, therefore, it is assumed that the location of all exiting utility lines will be identified by your staff. The Department assumes all responsibility or any utilities damaged during our field investigation provided URS drills at locations that have been approved by the County.

• We will plan to perform the field work during dry weather to minimize damage to embankment surface from traffic. However, if DSOD has concerns that the "traffic" from the drilling equipment will damage the lower portion of the embankment during rainy season, then we will postpone the field drilling program; until there is a dry period based on weather forecast.

• Our scope of investigation is limited to the engineering properties of the onsite soils and this scope excludes investigation or treatment of hazardous materials/wastes. In preparation of this proposal we have assumed that these materials are not present at the site and evaluation, treatment, and special handling of these materials are not included in our scope of services.

Task 3 – Laboratory Testing

Particle size analyses, Atterberg limits, direct shear, and water content tests will be performed on selected disturbed samples obtained from the rotary wash borings.

Task 4 – Analysis

Before performing the proposed analysis, URS will present and discuss in details of choosing the best alternative analysis for DSOD approval based on the results of the above field explorations and laboratory testing. URS will substantiate and document their opinion in selecting the best analysis based on their years of geotechnical expertise on dam rehabilitation. URS will address and confirm that the analysis of embankment stability is applicable regardless of the final concept for the spillway (even if the concept will not be approved by DSOD and the spillway must be relocated off the embankment).

4a) Seismicity and Conventional Analyses: Deterministic scenario ground motions will be computed for the maximum earthquakes that will generate the most significant ground shaking at the dam site. As recognized in previous

studies, the two most significant seismic sources are the Sierra Madre fault and the San Andreas Fault. Per DSOD criteria, median and 84th percentile, 5%-damped acceleration response spectra will be calculated using PEER's (Pacific Earthquake Engineering Research) Next Generation Attenuation ground motion prediction equations. A Vs30 (average shear-wave velocity in the top 30 m) for the dam site will be estimated based on available geotechnical and geologic data. The dam site is located in the hanging wall of the Sierra Madre fault and so hanging wall effects will be incorporated into the ground motions. The response spectra will be adjusted for average fault rupture directivity using recently developed model(s). Separate spectra will be computed for use on sections parallel and perpendicular to the axis.

Conventional Analyses: Prior to performing this analysis, URS will obtain approval from DSOD for using this conventional analysis to support any conclusion. Our conventional analyses will include the evaluation of liquefaction potential and associated calculated settlements utilizing the NCEER 1997 method, limit equilibrium stability analyses of embankment, and seismic stability analyses of embankment and associated calculated embankment deformation using simplified methods. In addition, URS will use piezometric surfaces in performing seismic stability analysis that are representative for the case of "full" reservoir condition and for the case of rapid drawdown condition. If conventional analysis shows that performance of the embankment is acceptable to DSOD, no further analyses will be performed.

If results of this analysis are unacceptable to DSOD, URS will discuss with the Department and DSOD the potential for performing the FLAC analyses to support the final design.

4b) FLAC Analyses: The FLAC analyses will consist of the following tasks:

• Input Ground Motions: We propose to analyze the dam for three MCE-level ground motions. Acceleration histories representing these events will be developed by matching/scaling representative earthquake records to a site-specific design spectrum. Separate ground motions will be computed for use on the sections parallel and perpendicular to the axis.

• Analyses of Existing (As-Built) Spillway Structure: The seismic performance of the spillway structure will be evaluated by performing nonlinear soil-structure interaction analyses for the most critical cross section with FLAC (Itasca, 2005). FLAC and its predecessor codes have been widely utilized and verified in the last 30 years against closed-form solutions, physical models, and case histories in the field (Roth et al., 2008 and 2004, Roth and Dawson, 2003; and Dawson et al., 2001).

Soils will be represented by an effective-stress based linear elastic/perfectly plastic Mohr-Coulomb constitutive model coupled with an empirical pore-

pressure generation scheme. Pore pressures will be updated every half shear-stress cycle, with an incremental version of the cyclic-stress approach developed at UC Berkley by Seed and co-workers. The spillway slab and walls will be modeled with elasto-plastic beam elements with strength degradation related to bending-curvature. These structural elements will be interacting with the soil mesh of the embankment through interface elements allowing potential gaps to open behind the spillway walls as they suffer shaking-induced permanent/plastic deformations in bending.

Analysis results will be provided in graphical format and will include vector and contour plots of permanent shaking-induced deformations; contour plots of shaking-induced excess pore pressures; profiles of soil pressure on the spillway wall at various times during shaking; post-shaking moment and shear diagrams for spillway walls and slab; shaking-deformed mesh plots at exaggerated scale; and envelops of maximum bending moments, shear forces and plastic rotations of structural elements contained in the model.

• Analysis of Embankment Section: The seismic performance of the embankment will be evaluated by analyzing the most critical cross section. This section will be analyzed and results presented in the same manner as discussed above.

In addition to the dynamic analysis runs, we will also perform static slope-stability analyses to evaluate the slope stability of the embankment before and after shaking. These analyses will also be performed with FLAC using the strength-reduction technique. In this procedure, the soil shear strength is reduced in stages until slope failure occurs, and the factor of safety is defined as the ratio of the soil's actual shear strength to the reduced strength at failure. Dawson et al. (1999) demonstrated that this definition of a safety factor is identical to that used in conventional limit-equilibrium analyses with trial slip circles or failure planes.

Task 5 – Conceptual Mitigation Alternatives

Conceptual mitigation alternatives will be presented based on the results of the analysis. This task will include identifying all areas and structures that require mitigation, including those that may be adversely affected from liquefaction of the embankment and foundation alluvium. Mitigation alternatives will address each specific area and structure that is subject to potential settlement and deformation.

Special attention will be made to address differential settlement of the spillway These alternatives will be evaluated based on the model as well as the better definition of liquefaction zones within dam embankment and underlying alluvial soils. Based on DSOD's previous comment, mitigation alternatives will include evaluations with saturation from long-term storage in the reservoir In addition, the effect of seismic forces on the spillway and outlet tower structures will be evaluated and appropriate conceptual mitigative measures will be provided.

Task 6 – Report

The results of our investigation will be provided in a draft report that will be submitted to the County and the DSOD for review and comment. The contents of this report will be addressed in a meeting with DSOD representatives in Sacramento. Our draft report will include the following:

- A description of site conditions and geotechnical factors that may impact the dam site.

- A summary of project requirements and design criteria known to us at the time of our field exploration and report preparation.

- A summary of subsurface conditions as encountered during our field exploration.

- A geotechnical map depicting the locations of borings and Becker Penetration Tests.

- Logs of borings.

- A description of laboratory test procedures and test results.

- Regional and site-specific faulting and seismicity

- Response spectra and acceleration-time histories.

- Evaluation of seismic and static stability and deformation analysis of the embankment slopes during the seismic events.

- Evaluation of seismic forces on spillway and outlet tower structures.

- Mitigation concepts for embankment slopes, spillway and outlet tower structures and associated approximate costs.

- Conclusions and recommendations. After completion of review of the draft report by the Department and the DSOD, the final report will be issued that will also include the responses to review comments on the draft report by the DSOD.

Task 7 – Preliminary Analyses and 30% Completion of Design Effort

Following the approval of the final report, we will coordinate with representatives of the Department to select a conceptual mitigation measure for further modeling and analyses. Appropriate and more detailed FLAC analyses will be preformed for the selected mitigation measure at this stage. It is assumed that a single mitigation measure, such as embankment improvement for the spillway mitigation will be analyzed using FLAC. A preliminary design report (PDR) will be prepared that will summarize the results of the FLAC analyses. The PDR will also include preliminary design plans, and preliminary cost estimate for the selected mitigation measure. The plans will include a site layout, cross sections and preliminary dimensions for estimating costs. This task does not include preparing detailed design criteria, or technical specifications. This task would correspond to 30 percent of the design effort, but not 30 percent plans.

Schedule

We expect to begin our investigation immediately upon receiving the written notice to proceed. Our estimated schedule that includes the estimated review time of work plan and draft report by the DSOD is presented on Table 1 We anticipate that the draft report will be submitted to the DSOD 8 weeks after the receipt of written notice to proceed. This assumes that Task 4b (FLAC) is required. Assuming that the DSOD will complete its review of the draft report in 12 weeks, the final report will be submitted 42 weeks after the receipt of written notice to proceed. It may be updated, modified, or changed upon LACDPW's requests.

Estimated Cost

URS proposes to charge for our services on a time and materials basis in accordance with our rate schedule provided in Attachments A and B and our existing Contract with the Department (70778). We understand that the Department may elect to perform some of the investigation tasks internally or through other contract vehicles. The cost for each task is provided in Table 2. Depending on the outcome of the investigation and analyses, some of the listed tasks may not be required. URS would not exceed these estimates without notifying the Department in advance. The breakdown details of estimated costs for different tasks are provided in attached Table 2 and 3.

Proposed Staffing

Mr Michael Smith will be URS' project manager. Mr Smith has over 22 years experience in geotechnical engineering and dam engineering in Southern California. Mr Smith worked on the original 1990s studies for the Santa Anita

Debris Dam and helped develop the recent spillway relocation concept. Mr. Michael Luebbers will provide peer reviews. Mr. Luebbers has over 30 years of dam and geotechnical expertise. He was the lead dam designer for the East Dam of the Diamond Valley Lake Project. Mr Wolfgang Roth will be the numerical analysis lead. Mr Roth has over 30 years experience in dam, reservoir and tunnel design. Mr Roth has extensive expertise in foundation and earthquake engineering and numerical modeling. We welcome the opportunity to discuss this request for change order with you if there are questions or comments regarding the proposed scope or methodology. The proposed scope of the work will be performed in accordance with the existing contact utilizing the fee rates provided in Attachments A and B.

FOOTHILL COMMUNITIES WATER SUPPLY RELIABILITY STUDY (FCWSRS)

Program Implementation Plan (PIP)

Final

June 2009

Prepared for:

U.S. Army Corps of Engineers
Los Angeles District
South Pacific Division

In Association With:

FWC Foothill Water Coalition

Prepared by:

TETRA TECH

Determine projected capital and operational costs for each of the alternatives investigated. Define breakdown of operating costs between agencies involved. Perform a cost/benefit analysis.

7. **PLAN FORMULATION & EVALUATION**

Provide an evaluation of each alignment including a conclusion/recommendation. Prepare a plan for the continuation of the project as necessary.

2. SAN GABRIEL/RAYMOND BASIN FEEDER

A. Project Background: Presently the existing San Gabriel Valley Municipal Water District (SGVMWD) Devil Canyon Azusa Pipeline terminates in Azusa at the San Gabriel Canyon Spreading Grounds. This serves the San Gabriel Valley by transporting untreated State Water Supply Project (SWP) water to the area but excludes the Raymond Basin and the Pasadena area, located further to the west, where supplemental water is in greatest need.

B. Project Benefits: The benefits of the San Gabriel and Raymond Basin Feeder include:

1. The San Gabriel and Raymond Basin Feeder will provide a new source of water to the Raymond Basin area through groundwater recharge and additional groundwater recovery.
2. Groundwater reliability, through additional groundwater recharge, is increased.
3. Increased groundwater pumping may also significantly increase the local supply of water that might be shared among agencies during a planned or unplanned shut down of imported water facilities.
4. Groundwater recharge in the Eaton Wash Spreading Grounds will improve declining groundwater levels in the Pasadena Subarea of the Raymond Basin. These declining groundwater levels have been identified as a significant groundwater basin management issue in the Raymond Basin that may lead to a phased curtailment of groundwater production. The San Gabriel and Raymond Basin Feeder would make imported water available for groundwater recharge to supplement recharge with local water.
5. Groundwater recharge in the Santa Anita and Sierra Madre Spreading Grounds will benefit the Santa Anita Subarea of the Raymond Basin. Groundwater production from the City of Sierra Madre's wells is impacted when insufficient amounts of local water are available for groundwater recharge. The San Gabriel and Raymond Basin Feeder would make imported water available for groundwater recharge to supplement recharge with local water. This will improve the reliability of groundwater production for the City of Sierra Madre, the City of Arcadia and other groundwater producers in the Santa Anita Subarea.
6. Groundwater recharge in the Arroyo Seco Spreading Ground will benefit the Monk Hill Subarea of the Raymond Basin

7. SGVMWD currently does not have facilities to deliver imported water to the City of Sierra Madre, a member agency. The San Gabriel and Raymond Basin Feeder will allow SGVMWD to deliver imported water supplies to the City of Sierra Madre for groundwater recharge at the Sierra Madre Spreading Grounds.

C. Project Description: This project would extend the existing SGVMWD Devil Canyon Azusa Feeder Pipeline from its present terminus in Azusa to the Eaton Wash Spreading Grounds in Pasadena in order to serve the Raymond Basin. This project will permit delivery of untreated water to the existing Santa Anita and Sierra Madre spreading grounds (Phase I) and the Eaton Wash Spreading Grounds (Phase II) for groundwater recharge within the Upper San Gabriel and Raymond Basins respectively.

An additional construction phase (Phase III) has been suggested, which would extend the feeder to the Arroyo Seco Wash to provide additional water at the spreading grounds. It is likely that a booster pump station will be required to increase the hydraulic grade line to reach the Arroyo Seco Spreading Grounds. This could be constructed on County property at the Eaton Wash Spreading Grounds but will require approval and permits from L.A. County Department of Public Works (LACDPW). Other booster pump station locations would need to be addressed which are not on County property.

Critical to implementation of this project is completion of the "Alosta Connection Project" described above (Project 1) and the Water Facilities Agreement between MWD and SGVMWD, which will ensure compliance with DWR's State Water Contract. This permits the pipeline to draw on water from either the SGVMWD Devil Canyon Azusa Feeder and/or the MWD-Rialto Feeder.

D. Proposed Alignment: West from the terminus of the SGVMWD Devil Canyon-Azusa Feeder Pipeline at the San Gabriel Canyon Spreading Grounds in Azusa to the Eaton Wash Spreading Grounds in Pasadena. Actual route is to be determined in preliminary design report. A possible alignment is shown in Figure 3 but is subject to modification during the preliminary design phase.

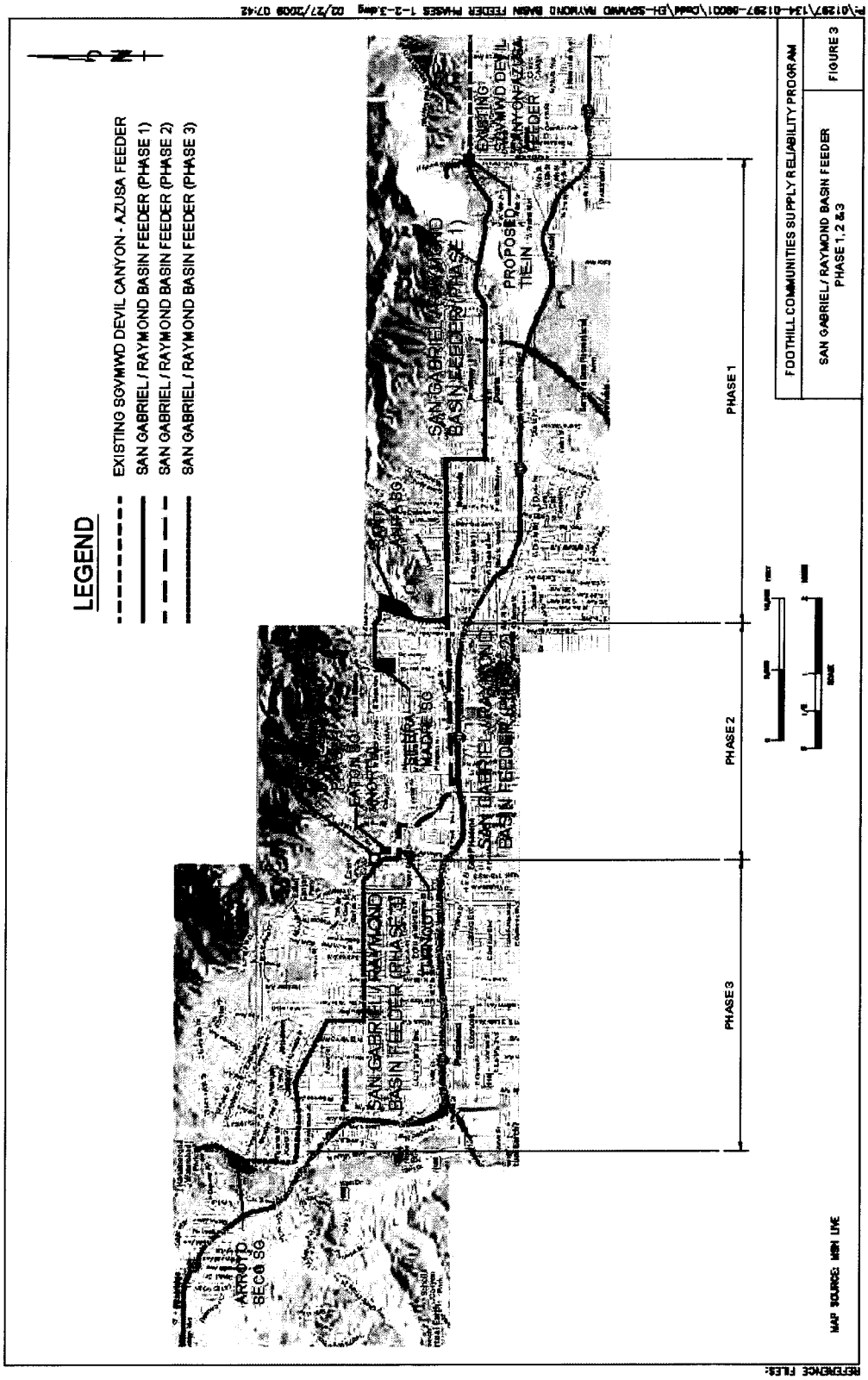
1. **Line Size and Capacity:** Line size and capacity will be determined based on the system hydraulics, the recommended points of delivery, the volumes of water to be discharged at each point of delivery and the mode of system operation. The mode of operation can have a great impact on the size and cost of the pipeline. If water must be delivered at all points at the same time, a much larger diameter pipe would be required as compared to a sequential delivery mode of operation.

2. **Approximate Pipe Distance:** 73,800-feet (approximately 14 miles). This will depend on the actual alignment selected.

3. **Facilities Required:**

    a. Turn out structure on Devils Canyon-Azusa Pipeline. This would probably be in the vicinity of Sierra Madre Ave. and North San Gabriel Ave.

b. Isolation and Control Valves at each discharge location to control the rate of flow. Tentatively, these would be at the Santa Anita Spreading Grounds, the Sierra Madre Spreading Grounds, and the Eaton Wash Spreading Grounds (North). An additional connection


LEGEND
EXISTING SGVMWD DEVIL CANYON - AZUSA FEEDER
SAN GABRIEL / RAYMOND BASIN FEEDER (PHASE 1)
SAN GABRIEL / RAYMOND BASIN FEEDER (PHASE 2)
SAN GABRIEL / RAYMOND BASIN FEEDER (PHASE 3)
PROPOSED TIE-IN
EXISTING SGVMWD DEVIL CANYON-AZUSA FEEDER
SAN GABRIEL / RAYMOND BASIN FEEDER (PHASE 1)
SAN GABRIEL / RAYMOND BASIN FEEDER (PHASE 2)
SAN GABRIEL / RAYMOND BASIN FEEDER (PHASE 3)
PHASE 1
PHASE 2
PHASE 3
FOOTHILL COMMUNITIES SUPPLY RELIABILITY PROGRAM
SAN GABRIEL / RAYMOND BASIN FEEDER
PHASE 1, 2 &3
FIGURE 3
MAP SOURCE: MRN LIVE
REFERENCE FILES:

could be located to serve the Eaton Wash. This turnout would be used to deliver water to the Eaton Spreading Basin (south). If the line is extended to the Arroyo Seco, discharge facilities would be required at that location too. Alignment and facilities on County properties will require approval and permits from LACDPW.

c. Metering facilities at each discharge point. It is important in the management of the system to monitor the volume of water delivered at each discharge point. As this is raw water, meters could be of venturi type or magnetic flux meters if higher accuracy is required.

d. A pumping facility will be required to deliver water to Phase III and possibly even for Phase II. This depends on the hydraulic grade line of the proposed system under dynamic conditions. It has been estimated that between 10 to 15 cfs of water could be delivered to the Santa Anita and Sierra Madre spreading grounds without additional pumping.

4. **Significant Crossings:** San Gabriel River. Tunneling is expected to cross the San Gabriel River. The tunnel may need to extend from West Sierra Madre Ave. in Azusa to Royal Oaks Drive in Duarte. This portion will need to be placed deep enough that flooding in the San Gabriel River will not affect the pipeline. There are a number of additional concrete drainage channels coming from the foothills that will also need to be crossed under.

5. **Easement/Permit Requirements:** Spreading agreements would be required from LACDPW for discharge to County operated Spreading Grounds and from the U.S. Corps of Engineers for discharge to existing channels. Easements or encroachment permits for pipeline construction within city streets may be required from the following:

a. City of Azusa

b. City of Duarte

c. City of Monrovia

d. City of Arcadia

e. City of Sierra Madre

f. City of Pasadena

g. SGVMWD

E. Water Availability: Imported water delivered through the proposed San Gabriel/Raymond Basin Feeder project would be provided by SGVMWD and MWD as available as discussed in the Introduction above. It would also be possible for SGVMWD to provide water for groundwater recharge to its member city, Sierra Madre.

F. Regulatory Issues: Regulatory and institutional issues associated with the San Gabriel and Raymond Basin Feeder include:

1. Coordination with MWD and SGVMWD for use of water supplies and pipeline facilities. In addition, MWD and SGVMWD can only serve imported water within their respective boundaries.
2. Establishment of ownership and rights to capacity.
3. Coordination with the Raymond Basin Management Board to develop a groundwater recharge program in the Raymond Basin, including compliance with the Raymond Basin Management Board's Draft Criteria for Delivery of Supplemental Water for groundwater recharge.
4. Coordination with LACDPW, the City of Sierra Madre and the City of Pasadena for use of spreading grounds.
5. Spreading of imported water in the Arroyo Seco Spreading Grounds for groundwater recharge may require a review of the impact on the USEPA's Record of Decision regarding groundwater contamination in the vicinity.
6. Depending on pipeline routing, coordination with Caltrans for construction of pipeline facilities through Caltrans' roadways.
7. Coordination and permitting with local cities for construction of pipeline facilities.
8. Implementation of the San Gabriel and Raymond Basin Feeder project would need to comply with California Environmental Quality Act (CEQA) and potentially National Environmental Policy Act (NEPA) requirements.
9. Groundwater recharge with imported water has not historically been subject to permitting requirements by the Regional Water Quality Control Board (RWQCB). However, the RWQCB in the Lahontan region has recently required permitting for a proposed imported water groundwater recharge project in the Antelope Valley.
10. Depending on the pipeline routing, potential impacts on sensitive habitats or species may need to be investigated in the Eaton Canyon and Arroyo Seco areas.
11. Establish protocol for delivery of water and use of connections.

G. Agency Involvement

1. Raymond Basin Management Board (RBMB)
2. California Department of Public Health
3. Regional Water Quality Control Board (RWQCB)
4. LA County Department of Public Works (LACDPW)
5. San Gabriel Valley Municipal Water District (SGVMWD)
6. Corps of Engineers

H. Program Costs

The estimated construction costs for the San Gabriel/Raymond Basin Feeder (Phases I, II, and III), based on and Engineering Construction Cost index of 8549 are:

Phase I	
Construction Costs	$15,285,150
Other Related Costs	$ 9,094,660
Total Project Costs	$24,379,810
Phase II	
Construction Costs	$ 6,948,000
Other Related Costs	$ 4,134,000
Total Project Costs	$11,082,000
Phase III	
Construction Costs	$12,010,400
Other Related Costs	$ 7,146,200
Total Project Costs	$19,156,600

Refer to Tables 6 and 7 for a summary of all program costs and a breakdown of associated project costs.

I. Work Breakdown Structure

The following Work Breakdown Structure (WBS) indicates the various tasks and subprojects required to prepare the PFS for the "San Gabriel/Raymond Basin Feeder" project.

WBS#	Description
1	Engineering Appendix
1.A	Feas - Surveys and Mapping
	Utility Research
	Mapping
1.B	Feas - Geotechnical Appendix
1.C	Feas - Engineering & Design Appendix
	Hydraulics
	Alignment Alternatives
	Control & Metering
	`SCADA
2	Real Estate Appendix
2.A	R/W
2.B	Easements
3	Water Rights/Procedures
3.A	Legal Framework
3.B	Water Rights
3.C	Operational Procedures
4	Environmental Studies/Appendix
4.A	Traffic and Sediment Control
4.B	Plant and Wildlife

4.C	Flooding
5	Permits
6	Phasing Plan
7	Cost Estimates
8	Plan Formulation & Evaluation

J. Scope of Work:

The following scope of work defines each of the WBS tasks outlined above.

SAN GABRIEL/RAYMOND BASIN FEEDER

1. **ENGINEERING APPENDIX**

1.A **Feas-Surveys and Mapping**

Topographic Mapping: Obtain street plans or other suitable plans for the areas covering each alignment.

Utility Research: Evaluate existing street plans and available utility maps to determine congestion of utilities and possible location of new watermain.

1.B **Feas-Geotechnical Appendix**

Soils Study: Obtain previous soils reports from municipalities along each of the potential alignments. Coordinate with each municipality to identify known fatal flaws such as contaminated soil areas.

Obtain trench replacement requirements from each municipality along the recommended alignment.

Evaluate spreading ground recharge capacities and groundwater impacts. Conduct field percolation tests.

1.C **Feas-Engineering & Design Appendix**

Hydraulics: Investigate the system hydraulics to determine pipeline design capacity, water sources and establish the most cost effective pipe size. Investigate hydraulic grades at points of delivery and point of connection to the existing feeder. Pumping may be required to deliver water depending on system hydraulics, pipe size and elevation differences. Determine if pumping is required to deliver water to the final points of use and develop preliminary pumping criteria and location of pumping facilities if pumping is required. It is anticipated that pumping will be required for Phase III because of elevation at point of delivery. Analyze system for transient flow/water hammer conditions.

Alignment Alternatives: Evaluate alternative pipe routings and determine most cost effective pipe routing. Consider conflicts with other utilities, drainage crossings and traffic impacts. Also important is to be sure there is sufficient road width where the pipeline is aligned within existing road

right-of-ways. One piping alternative was suggested above. Identify significant highway and drainage crossings along route and establish requirements for each crossing. Evaluate locations and impacts of booster station placement if county properties are not available.

Prepare matrix to document significant issues with each alignment such as relative impacts on traffic, ease of construction, relative permit issues, relative environmental issues, costs, etc.

Determine recommended alignment.

Control & Metering: Identify points of delivery and recommended capacity at each delivery point. Determine control valve and metering requirements for each delivery point. Determine the size of the required structure (i.e. vault or building) to house the turnout and related equipment.

Identify location for control and metering structure.

SCADA (Supervisory Control And Data Acquisition): Investigate and recommend SCADA monitoring and reporting for agencies involved in operation and control of flows.

2. **REAL ESTATE APPENDIX**

Determine location of existing easements, properties, and rights-of-way impacted by each potential alignment. Identify all required land acquisitions, leases, and easement along with their estimated costs.

3. **WATER RIGHTS/PROCEDURES**

3.A Legal Framework: There is currently a draft "Water Facilities Agreement (Phase 1)" being considered between MWD and SGVMWD which defines the funding, ownership, implementation and operation of the Alosta Connection project. It also includes when and how much water can be transferred. Determine the impacts of that agreement on this pipeline. Identify other legal documents/agreements that may impact the conveyance and use of water under this project. MWD and SGVMWD can only serve imported water within their respective boundaries.

3.B Water Rights: Determine any water rights involved in transferring water between MWD and SGVMWD. Define ownership and rights to capacity of facilities.

3.C Operational Procedures: Define procedures for when and how much water can be transferred between the agencies.

4. **ENVIRONMENTAL STUDIES/APPENDIX**

Identify the appropriate NEPA/CEQA document (e.g., Categorical Exclusion/Exemption, Environmental Assessment/Initial Study, Environmental Impact Statement/Report) based on the likely impacts of any future actions including impacts to special status and/or endangered species.

4.A Traffic & Sediment Control: Identify construction traffic impacts along the recommended pipeline alignment. Delivery of construction materials, excavation of trenches and installation of pipe and appurtenances will impact local traffic and generate noise and dust pollution. Identify requirements for construction sediment control to prevent project related sediments from entering waterways.

4.B Plant and Wildlife: Construction within the San Gabriel River flood plain will require addressing any impacts on plant and wildlife within the river bottom.

4.C Flooding: The proposed pipeline crosses the San Gabriel River. Evaluate the scouring potential during high flows and protection of the pipeline during flooding.

5. **PERMITS:**

Determine permitting agencies involved in the project and define the type of permit required and the procedures and time required for each permit. Identify substantial impacts of the required permits.

6. **PHASING PLAN:**

Develop a phasing plan to spread out the pipeline construction. The following construction phases have been recommended:

Phase I: Devil Canyon Azusa Pipeline to Santa Anita Ave. and connection to the Santa Anita and Sierra Madre Spreading Grounds.

Phase II: Sierra Madre Spreading Grounds to the Eaton Wash Spreading Grounds

Phase III: Eaton Wash Spreading Grounds to Arroyo Seco Wash to take advantage of the spreading grounds at the Devil's Gate dam. This segment may require pumping.

The phasing plan shall use the above phases or provide recommendations for other phasing. Impacts of breaking up the project into phases shall be evaluated.

7. **COST ESTIMATES:**

Determine projected capital and operational costs for each of the alternatives investigated.

Define breakdown of operating costs between agencies involved for the recommended project. Also, identify costs broken down by the phases.

8. **PLAN FORMULATION & EVALUATION**

Provide an evaluation of each alignment including a conclusion/recommendation. Prepare a plan for the continuation of the project as necessary.

APPENDIX 3-B

Labor Compliance Plan

2009

Labor Compliance Program

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Labor Compliance Program

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SECTION I - PUBLIC WORKS SUBJECT TO PREVAILING WAGE LAWS

State prevailing wage rates apply to all public works contracts as set forth in Labor Code Sections 1720 et seq. and include, but are not limited to, such types of work as construction, alteration, demolition, repair, or maintenance work. The Division of Labor Statistics and Research (DLSR) predetermines the appropriate prevailing wage rates for particular construction trades and crafts by county.

A. Types of Contracts to Which Prevailing Wage Requirements Apply

As provided in Labor Code Section 1771 et seq., a Labor Compliance Program as described in subdivision (b) of section 1771.5 of the Labor Code shall be initiated and enforced on all applicable public works projects. Upon approval by the Director of the Department of Industrial Relations, this LCP shall apply to all statutes that require Awarding Bodies to have a LCP to which the payment of prevailing wages apply as a condition of project authorization, project funding, or use of specified contracting authority. The Department of Industrial Relations' website shall maintain a list of statutes requiring LCP oversight.

B. Applicable Dates for Enforcement of the LCP

The applicable dates for enforcement of awarding body Labor Compliance Programs is established by Section 16422 of the California Code of Regulations. Contracts are not subject to the jurisdiction of the Labor Compliance Program until after the program has received approval.

SECTION II - COMPETITIVE BIDDING ON PUBLIC WORKS CONTRACTS

All bid advertisements (or bid invitations) and public works contracts shall contain appropriate language concerning the requirements of the Labor Code. Notice of approval of an Awarding Body's Labor Compliance Program shall be given in the Call for Bids and in the contract or purchase order and shall also be posted at the job site. The Notice of an approved Labor Compliance Program shall contain, at the minimum, the effective date of the Director's initial or final approval, a statement whether the limited exemption from prevailing wages pursuant to Labor Code Section 1771.5(a) applies to contracts under the jurisdiction of the LCP, a telephone number to call for inquiries, questions, or assistance with regard to the LCP, and the name of the agent or office administering the LCP.

SECTION III - JOB START MEETING

After the award of a public works contract, and prior to the commencement of the work, a mandatory Job Start meeting (Pre-Job conference) shall be conducted by TSG with the contractor and those subcontractors listed in its bid documents or who are required to be identified or prequalified in a Design-Build Contract.

At that meeting, federal and state labor law requirements applicable to the contract will be discussed and copies of suggested reporting forms furnished, including prevailing wage requirements, the respective record keeping responsibilities, the requirement for the submittal of certified payroll records, and the prohibition against discrimination in employment.

The contractor and each subcontractor will be provided with a Checklist of Labor Law Requirements (presented in Part Three of this document) and will discuss in detail the following checklist items:

1. The contractor's duty to pay prevailing wages (Labor Code §1770 et seq.);

2. The contractor's duty to employ registered apprentices on public works projects (Labor Code §1777.5);

3. The penalties for failure to pay prevailing wages and to employ apprentices, including forfeitures and debarment (Labor Code §1775, §1777.7, and §1813);

4. The requirement to maintain and submit copies of certified payroll records, on a weekly basis, as required (Labor Code §1776), and penalties for failure to do so (Labor Code §1776(g)); the requirement includes and applies to all subcontractors performing work on projects even if their portion of the work is less than one half of one percent of the total amount of the contract.

5. The prohibition against employment discrimination (Labor Code §1735 and §1777.6; the Government Code; and Title VII of the Civil Rights Act of 1964, as amended);

6. The prohibition against taking or receiving a portion of an employee's wages (Labor Code §1778) (kickback);

7. The prohibition against accepting fees for registering any person for public works (Labor Code §1779) or for filing work orders on public works (Labor Code §1780);

8. The requirement to list all subcontractors that are performing one-half of one percent of the total amount of the contract (Public Contract Code Section 4100 et seq.);

9. The requirement to be properly licensed and to require all subcontractors to be properly licensed, and the penalty for employing workers while unlicensed (Labor Code §1021 and under California Contractors License Law. Also, see Business and Professions Code §7000, et seq);

10. The prohibition against unfair competition (Business and Professions Code §17200-17208);

11. The requirement that the contractor and subcontractor be properly insured for Workers' Compensation (Labor Code §1861);

12. The requirement that the contractor abide by the Occupational Safety and Health laws and regulations that apply to the particular public works project.

13. The requirement to secure proof of eligibility/citizenship from all project workers due to the federal prohibition against hiring undocumented workers.

14. The requirement to provide itemized wage statements to project employees under Labor Code §226.

The contractors and subcontractors present at the Job Start meeting will be given the opportunity to ask questions relative to the items contained in the Labor Law Requirements Checklist. The checklist will then be signed by the contractor's representative, a representative of each subcontractor, and a representative of the LCO.

At the Job Start meeting, the contractor will be provided with a copy of the LCP package which includes: a copy of the approved LCP, the checklist of Labor Law Requirements, applicable Prevailing Wage Rate Determinations, blank certified payroll record forms, fringe benefit statements, State apprenticeship requirements, and a copy of the Labor Code relating to Public Works and Public Agencies (Part 7, Chapter 1, §1720-§1861).

It will be the prime contractor's responsibility to provide copies of the LCP package to all listed subcontractors and to any substituted subcontractors.

SECTION IV – MONITORING PAYMENT OF PREVAILING WAGES

A. Certified Payroll Records Required

The contractor and each subcontractor shall maintain payrolls and basic records (timecards, canceled checks, cash receipts, trust fund forms, accounting ledgers, tax forms, superintendent and foreman daily logs, etc.) during the course of the work and shall preserve them for a period of three (3) years thereafter for all trades workers working on projects which are subject to the LCP. Such records shall include the name, address, and social security number of each worker, his or her classification, a general description of the work each employee performed each day, the rate of pay (including rates of contributions, or costs assumed to provide fringe benefits), daily and weekly number of hours worked, and actual wages paid.

1. Submittal of Certified Payroll Records

   The contractor and each subcontractor shall maintain weekly certified payroll records for submittal as required. The contractor shall be responsible for the submittal of payroll records of all its subcontractors. All certified payroll records shall be accompanied by a statement of compliance signed by the contractor or each subcontractor, indicating that the payroll records are correct and complete, that the wage rates contained therein are not less than those determined by the Director of the Department of Industrial Relations, and that the classifications set forth for each employee conform with the work performed.

   Certified payroll records required by Labor Code Section 1776 may be maintained and submitted electronically subject to the conditions set forth in CCR§16404.

   California Code of Regulations (Title 8, section 16000) defines "payroll records" as: all time cards, cancelled checks, cash receipts, trust fund forms, books, documents, schedules, forms, reports, receipts or other evidences which reflect job assignments, work schedules by days and hours, and the disbursement by way of cash, check, or in whatever form or manner, of funds to a person(s) by job classification and/or skill pursuant to a public works project. TSG may request payroll records at any time and the contractor is required to provide requested payroll records within 10 days following written receipt of the request.

2. Full Accountability

   Each individual, laborer or craftsperson working on a public works contract must appear on the payroll. The basic concept is that the employer who pays the trade worker must report that individual on its payroll. This includes individuals working as apprentices in an apprenticeable trade. Owner-operators are to be reported by the contractor employing them; rental equipment operators are to be reported by the rental company paying the workers' wages.

   Sole owners and partners who work on a contract must also submit a certified payroll record listing the days and hours worked, and the trade classification descriptive of the work actually done.

   The contractor shall provide the records required under this section within ten (10) days of each payday and make them available for inspection by the Department of Industrial Relations, and shall permit representatives of each to interview trades workers during working hours on the project site.

3. Responsibility for Subcontractors

   The prime contractor shall be responsible for ensuring adherence to labor standards provisions by its subcontractors. Moreover, the prime contractor is responsible for Labor Code violations of its subcontractors in accordance with Labor Code Section 1775.

4. Payment to Employees

Employees must be paid unconditionally, and not less often than once each week, the full amounts, that are due and payable for the period covered by the particular payday. Thus, an employer must establish a fixed workweek (Sunday through Saturday, for example) and payday (such as every Friday or the preceding day should such payday fall on a holiday). On each and every payday, each worker must be paid all sums due as of the end of the preceding workweek and must be provided with an itemized wage statement as required by Labor Code §226.

If an individual is called a subcontractor, whereas, in fact, he/she is merely a journey level mechanic supplying only his/her labor, such an individual would not be deemed a bona fide subcontractor and must be reported on the payroll of the prime contractor as a trade worker. Moreover, any person who does not hold a valid contractor's license cannot be a subcontractor, and anyone hired by that person is the worker or employee of the general contractor for purposes of prevailing wage requirements, certified payroll reporting and workers' compensation laws.

The worker's rate for straight time hours must equal or exceed the rate specified in the contract by reference to the "Prevailing Wage Determinations" for the class of work actually performed. Any work performed on Saturday, Sunday, and/or on a holiday, or portion thereof, must be paid the prevailing rate established for those days regardless of the fixed workweek. The hourly rate for hours worked in excess of 8 hours in a day and 40 hours in a workweek shall be premium pay. All work performed on Saturday, Sunday and holidays shall be paid pursuant to the Prevailing Wage determination.

B. Audit of Certified Payroll Records

Audits shall be conducted under the direction of the LCO, and shall also be conducted at the request of the Labor Commissioner to determine whether all trades workers on project sites have been paid according to the prevailing wage rates.

Payroll records, once furnished by the contractors, shall be reviewed by TSG staff as promptly as practicable after receipt, but in no case not more than 30 days after receipt. A review of payroll records consists of ensuring that all appropriate data elements identified in Labor Code 1776(a) have been reported; certification forms have been signed; correct prevailing wage rates have been reported as paid for each labor classification; and confirmation of payment has been corroborated by TSG staff. TSG staff will review independent sources of information in order to confirm that payments were made to the worker, which may include but not be limited to: employee interviews, worker's paycheck stubs, copies of bank certified cancelled checks, payroll registers, and/or Employer Payments trust fund accounting records.

For each month in which a contractor reports having workers employed on the project, TSG will undertake random confirmation of payroll records for at least one worker for at least one weekly period within that reporting month. Confirmation of payroll records is defined as an independent corroboration of reported prevailing wage payments. Confirmation will be accomplished through on-site worker interviews, examination of paychecks or paycheck stubs, direct confirmation of payments from third party recipients of "employer payments," or any other reasonable method of corroboration.

If a violation is discovered, TSG shall prepare a written report of the investigation/audit, which at minimum will contain the following information:

- Brief written narrative identifying the Bid Advertisement Date of the contract for public work and summarizing the nature of the violation and the basis upon which the determination of underpayment was made;
- Any penalties to be assessed under Labor Code Section 1775 and 1813;

- Public Works Investigation Worksheet, showing the actual hours worked, amounts paid, amounts due, and classifications of workers employed in connection with the public work;
- Public Works Audit Worksheet, summarizing the information contained on the Public Works Investigation Worksheet; and
- Prevailing Wage Determination Summary, which lists the work classifications involved in the audit and associated prevailing wage amounts.

C. On-Site Employee Interviews

Pursuant to California Code of Regulations §16432(d), in order to confirm the payment of prevailing wages for the appropriate work classification performed by workers on the project, TSG staff will conduct on-site project visits to interview project workers.

On-site visits will be undertaken during each week that workers are present at the job site and may be undertaken randomly or as deemed necessary by the Labor Compliance Program. All craft workers shall be interviewed at least once over the life of the project. While on the job site, a representative sample of all crafts should be interviewed. For example, if four carpenters are working on the job site on the inspection day, two should be interviewed. The interview should not be conducted under the immediate supervision of any project foremen, so as to provide the worker with privacy. Any suspicious behavior such as coaching by the supervisor, or a contractor's unwillingness to allow the required site interview to take place, will result in a more in-depth investigation to take place.

D. Worker Complaints

In addition to monitoring all certified payroll records provided by the contractors, worker complaints of underpayment of prevailing wage rates must be investigated. Employees will be informed that the filing of the complaint is confidential, and that his or her identity will be disclosed to the contractor or subcontractor only after an investigation has occurred and only within the context of an audit worksheet.

Once TSG is notified of a complaint, a written acknowledgement that the complaint has been received must be sent promptly to both the complaining party and the Awarding Body. The acknowledgement must state the name, mailing address and telephone number of the TSG staff assigned to the complaint.

Within 10 days of receiving the complaint, both the complaining party and the Awarding Body must be notified in writing of the resolution or of the status of the complaint and the reason the complaint has not been resolved. Further, the complaining party and the Awarding Body must be notified in writing at least once every 30 days of the status of an unresolved complaint.

If an employee of the contractor reports the violation(s), TSG staff will thoroughly investigate the charge by interviewing the affected employee(s) and analyzing the submitted documents. If it is determined that a violation has occurred, the TSG staff shall prepare a Public Works Audit Worksheet, to be sent to the contractor. Major components and tasks related to investigating worker complaints are: gather supporting documents from all available sources to analyze for authenticity; conducting a complete CPR and/or project audit; and review CPRs for errors, inconsistencies, discrepancies, falsification, misclassification, under-reporting and any other omissions that render the records inaccurate where needed by comparing the inspector of records' daily log with all available records.

SECTION V - APPRENTICE REQUIREMENTS

A. Use of Apprentices

Apprentices shall be permitted to work as such only when they are registered, individually, under a bona fide apprenticeship program registered and approved by the State Division of Apprenticeship Standards. The allowable ratio of apprentices to journeypersons in any craft/classification shall not be greater than the ratio permitted to the contractor as to its entire workforce under the registered program.

Any worker listed on a payroll at an apprentice wage rate who is not registered shall be paid the journey level wage rate determined by the Department of Industrial Relations for the classification of the work he/she actually performed. Pre-apprentice trainees, trainees in non-apprenticeable crafts, and others who are not duly registered will not be permitted on public works projects unless they are paid full prevailing wage rates as journeypersons.

Compliance with California Labor Code Section 1777.5 requires all public works contractors and subcontractors to:

1. Submit contract award information to the apprenticeship committees for each apprenticeable craft or trade in the area of the Project. Must send the DAS-140 form to all apprenticeship committees, unless the contractor is registered with a particular one;

2. Request dispatch of apprentices from the applicable Apprenticeship Program(s) and employ apprentices on public works projects in a ratio to journeypersons which in no case shall be less than one (1) hour of apprentice work to each five (5) hours of journeyperson work; and

3. Contribute to the applicable Apprenticeship Program(s) or the California Apprenticeship Council in the amount identified in the prevailing wage rate publication for journeypersons and apprentices. If payments are not made to an Apprenticeship Program, they shall be made to the California Apprenticeship Council, Post Office Box 420603, San Francisco, CA 94142.

If the contractor is registered to train apprentices, it shall furnish written evidence of the registration (i.e., Apprenticeship Agreement or Statement of Registration) of its training program and apprentices, as well as the ratios allowed and the wage rates required to be paid there under for the area of construction, prior to using any apprentices in the contract work. It should be noted that a prior approval for a separate project does not confirm approval to train on any project. The contractor/subcontractor must check with the applicable Joint Apprenticeship Committee to verify status.

B. Apprentice hours-to-Journeyman hours (Ratios)

Unless a collective bargaining agreement specifies a different ratio, the typical ratio of apprentice hours to journeyman hours is one to five; in other words, apprentices in an apprenticeable trade should typically work 20 percent of total hours worked. The Division of Apprenticeship Standards maintains a list of all trades and their applicable specified apprenticeship ratios.

If TSG determines that at the end of a contractor or subcontractor's work on the public works contract that the contractor was in violation of the apprentice-to-journeyman ratio, TSG will require that the regular prevailing wage rate be paid to any worker who is not a duly registered apprentice and for any hours in excess of the maximum ratio permitted under Labor Code Section 1777.5(g).

SECTION VI - REPORTING WILLFUL VIOLATIONS TO THE LABOR COMMISSIONER

If an investigation reveals that a willful violation of the Labor Code has occurred, the LCO will make a written report to the Labor Commissioner reflecting prevailing wage deficiencies for each underpaid worker, and including any penalties to be assessed under Labor Code sections 1775 and 1813, as determined by the LCP after consideration of the best information available as to actual hours worked, amounts paid, and classifications of workers employed in connection with the public work. Such available information may include, but is not limited to, worker interviews, complaints from workers or other interested persons, all time cards, bank certified cancelled checks, cash receipts, trust fund forms, books, documents, schedules, forms, reports, receipts or other evidences which reflect job assignments, work schedules by days and hours, and the disbursement by way of cash, check, or in whatever form or manner, of funds to a person(s) by job classification and/or skill pursuant to a public works project. An Audit using the Audit Record Worksheets found in Attachment E, when accompanied by a brief narrative identifying the Bid Advertisement Date of the contract for public work and summarizing the nature of the violation and the basis upon which the determination of underpayment was made, presumptively demonstrates sufficiency.

Labor Code Section 1777.1 states:

> "A willful violation occurs when the contractor or subcontractor knew or reasonably should have known of his or her obligations under the public works law and deliberately fails or refuses to comply with its provisions."

Six (6) types of willful violations are reported:

A. Failure to Comply with Prevailing Wage Rate Requirements

Whenever it is determined that a willful violation has occurred in failing to comply with prevailing wage rate requirements (as set forth in the Labor Code and public works contracts) by paying less than the stipulated basic hourly rate to trades workers, or if overtime, holiday rates, fringe benefits, and/or employer payments are paid at a rate less than stipulated, it shall be reported to the Labor Commissioner upon completion of an investigation and audit.

B. Falsification of Payroll Records, Misclassification of Work, and/or Failure to Accurately Report Hours of Work

Falsification of payroll records and failure to accurately report hours of work is characterized by deliberate underreporting of hours of work; underreporting the headcount; stating that the proper prevailing wage rate was paid when, in fact, it was not; clearly misclassifying the work performed by the worker; and any other deliberate and/or willful act which results in the falsification or inaccurate reporting of payroll records.

C. Failure to Submit Certified Payroll Records

Whenever it is determined that a willful violation has occurred by the contractors and/or subcontractors not having responded to the 10-day notification by the LCO to comply with the requirement of submittal of weekly certified payroll records and/or to correct inaccuracies or omissions that have been detected, it shall be reported to the Labor Commissioner upon completion of an investigation and audit.

D. Failure to Pay Fringe Benefits (Employer Contributions)

Fringe benefits are defined as the amounts stipulated for employer payments or trust fund contributions and are determined to be part of the required prevailing wage rate. Whenever it is determined that a willful violation has occurred by failure to pay or provide fringe benefits and/or make trust fund contributions on a timely basis, which is equivalent to payment of less than the stipulated wage rate, it shall be reported to the Labor Commissioner upon completion of an investigation and audit.

E. Failure to Pay the Correct Apprentice Rates and/or Misclassification of Workers as Apprentices

Failure to pay the correct apprentice rate or classifying a worker as an apprentice when he/she is not properly registered is equivalent to payment of less than the stipulated wage rate and shall be reported to the Division of Apprenticeship Standards as a willful violation, upon completion of an investigation and audit.

F. Taking or Receiving a Portion of an Employee's Wages (Kickbacks)

Accepting or extracting kickbacks from employee wages under Labor Code Section 1778 constitutes a felony and may be prosecuted by the appropriate enforcement agency.

After the Labor Compliance Program has determined that violations of the prevailing wage laws have resulted in the underpayment of wages and an audit has been prepared, prior to a determination of the amount of forfeiture by the Labor Commissioner, notification shall be provided to the contractor and affected subcontractor of an opportunity to resolve the wage deficiency pursuant to CCR§16432(f). The contractor and affected subcontractor may, within 10 days after notification of the wage deficiency, submit to the Labor Compliance Program for consideration exculpatory information consistent with the "good faith mistake" factors set forth in Labor Code Section 1775(a)(2)(A)(i) and (ii). The Labor Compliance Program shall not be required to request the Labor Commissioner for a determination of the amount of penalties to be assessed under Labor Code Section 1775 if, (a) based upon the contractor's submission, the Labor Compliance Program reasonably concludes that the failure to pay the correct wages was a good faith mistake, and has no knowledge that the contractor and affected subcontractor have a prior record of failing to meet their prevailing wage obligations, and (b) the underpayment of wages to workers is promptly corrected and proof of such payment is submitted to the Labor Compliance Program.

SECTION VII - ENFORCEMENT ACTION

A. Duty of the Awarding Body

The Solis Group, as the entity having an approved LCP, has a duty to the Director of the Department of Industrial Relations to enforce Labor Code Section 1720 et seq. and the procedural regulations of the Department of Industrial Relations in a manner consistent with the practice of DLSE and regulations found at Title 8, California Code Regulations, Section 16000 et seq.

B. Withholding Contract Payments When Payroll Records are Delinquent or Inadequate

1. "Withhold" means to cease payments by the awarding body, its agents or others who pay on its behalf to the contractor. Where the violation is by a subcontractor, the contractor shall be notified of the nature of the violation and reference made to its rights under Labor Code Section 1729.

   A release bond under Civil Code Section 3196 may not be posted for the release of the funds being withheld for the violation of the prevailing wage law.

2. "Contracts" except as otherwise provided by agreement, means only contracts under a single master contract, including a design-build contract, or contracts entered into as stages of a single project which may be the subject of withholding, pursuant to the Labor Code §1720, §1720.2, §1720.3, §1720.4, §1771, and §1771.5;

3. "Delinquent payroll records" means those not submitted on the basis set forth in the Contract;

4. "Inadequate payroll records" are any one of the following:

   a. A record lacking any of the information required by Labor Code §1776;

b. A record which contains all of the required information but is not certified, or is certified by someone who is not an agent of the contractor or subcontractor;

c. A record remaining uncorrected for one payroll period, after the Labor Compliance Program has given the contractor or subcontractor notice of inaccuracies detected by audit or record review. Provided, however, that prompt correction will stop any duty to withhold if such inaccuracies do not amount to one (1) percent of the entire Certified Weekly Payroll in dollar value and do not affect more than half the persons listed as workers employed on that Certified Weekly Payroll, as defined in Labor Code Section 1776 and Title 8 CCR section 16401 of Title 8 of the California Code of Regulations.

5. The withholding of contract payments when payroll records are delinquent or inadequate is required by Labor Code Section 1771.5(b)(5), and it does not require the prior approval of the Labor Commissioner. The Awarding Body shall only withhold those payments due or estimated to be due to the contractor or subcontractor whose payroll records are delinquent or inadequate, plus any additional amount that the LCP has reasonable cause to believe may be needed to cover a back wage and penalty assessment against the contractor or subcontractor whose payroll records are delinquent or inadequate; provided that a contractor shall be required in turn to cease all payments to a subcontractor whose payroll records are delinquent or inadequate until the LCP provides notice that the subcontractor has cured the delinquency or deficiency.

6. When contract payments are withheld, the LCP shall provide the contractor and subcontractor, if applicable, with immediate written notice that includes all of the following: (1) a statement that payments are being withheld due to delinquent or inadequate payroll records, and that identifies what records are missing or states why records that have been submitted are deemed inadequate; (2) specifies the amount being withheld; and (3) informs the contractor or subcontractor of the right to request an expedited hearing to review the withholding of contract payments under Labor Code Section 1742, limited to the issue of whether the records are delinquent or inadequate or the LCP has exceeded its authority under this section.

7. No contract payments shall be withheld solely on the basis of delinquent or inadequate payroll records after the required records have been produced.

8. In addition to withholding contract payments based on delinquent or inadequate payroll records, penalties shall be assessed under Labor Code Section 1776(g) for failure to timely comply with a written request for certified payroll records. The assessment of penalties under Labor Code Section 1776(g) does require the prior approval of the Labor Commissioner under section 16436 of these regulations.

C. Withholding for Violation for Not Paying the Per Diem Prevailing Wages

1. "Withhold" and “contracts” have the same meaning set forth in Sections 16435(a) and 16435(b) of these regulations.

2. Where the violation is by a subcontractor, the general contractor shall be notified of the nature of the violation and reference made to its rights under Labor Code Section 1729.

3. “Amount equal to the underpayment” is the total of the following determined by payroll review, audit, or admission of the contractor or subcontractor:

a. The difference between amounts paid workers and the correct General Prevailing Rate of Per Diem Wages, as defined in Labor Code Section 1773, and determined to be the prevailing rate

due workers in such craft, classification or trade in which they were employed and the amounts paid;

b. The difference between amounts paid on behalf of workers and the correct amounts of Employer Payments, as defined in Labor Code Section 1773.1 and determined to be part of the prevailing rate costs of contractors due for employment of workers in such craft, classification or trade in which they were employed and the amounts paid;

c. Estimated amounts of "illegal taking of wages";

d. Amounts of apprenticeship training contributions paid to neither the program sponsor's training trust nor the California Apprenticeship Council; and

e. Estimated penalties under Labor Code Sections 1775, 1776, and 1813.

4. The withholding of contract payments when, after investigation, it is established that underpayment or other violations have occurred requires the prior approval of the Labor Commissioner under CCR Sections 16436 and 16437.

5. Provisions relating to the penalties under Labor Code Sections 1775, and 1813:

a. Pursuant to Labor Code Section 1775, the contractor shall, as a penalty, forfeit up to fifty dollars ($50) for each calendar day, or portion thereof, for each worker paid less than the prevailing wages.

b. Pursuant to Labor Code Section 1813, the contractor shall, as a penalty, forfeit twenty-five dollars ($25) for each worker employed in the execution of the contract by the contractor or by any subcontractor for each calendar day during which such worker is required or permitted to work more than eight (8) hours in any one calendar day and 40 hours in any one calendar week.

D. Forfeitures Requiring Approval by the Labor Commissioner

1. For purposes of this section and CCR Section 16437, "forfeitures" means the amount of wages, penalties, and forfeitures assessed by the LCP and proposed to be withheld pursuant to Labor Code Section 1771.6(a), and includes the following: (1) the difference between the prevailing wage rates and the amount paid to each worker for each calendar day or portion thereof for which each worker was paid less than the prevailing wage rate by the contractor or subcontractor; and (2) penalties assessed under Labor Code Sections 1775, 1776 and 1813.

2. If the aggregate amount of forfeitures assessed as to a contractor or subcontractor is less than $1000.00, the forfeitures shall be deemed approved by the Labor Commissioner upon service and the Labor Commissioner's receipt of copies of the following: (1) the Notice of Withholding of Contract Payments authorized by Labor Code Section 1771.6(a); (2) an Audit as defined in CCR Section 16432(e), and (3) a brief narrative identifying the Bid Advertisement Date of the contract for public work and summarizing the nature of the violation, the basis of the underpayment, and the factors considered in determining the assessment of penalties, if any, under Labor Code Section 1775.

(c) For all other forfeitures, approval by the Labor Commissioner shall be requested and obtained in accordance with CCR Section 16437.

E. Determination of Amount of Forfeiture by the Labor Commissioner

1. When the LCO requests a determination of the amount of forfeiture, the request shall include a file or report to the Labor Commissioner which contains at least the information specified in subparts (a) through (i) below. Attachment G is a sample Request for Approval of Forfeiture under this section.

   (a) Whether the public work has been accepted by the awarding body and whether a valid notice of completion has been filed, and the dates if any when those events occurred, and the amount of funds being held in retention by the Awarding Body;

   (b) Any other deadline which if missed would impede collection;

   (c) Evidence of violation, in narrative form;

   (d) Evidence of violation obtained under CCR Section 16432 and a copy of the Audit prepared in accordance with CCR Section 16432 setting forth the amounts of unpaid wages and applicable penalties;

   (e) Evidence that before the forfeiture was sent to the Labor Commissioner (A) the contractor and subcontractor were given the opportunity to explain why there was no violation, or that any violation was caused by good faith mistake and promptly corrected when brought to the contractor or subcontractor's attention, and (B) the contractor and subcontractor either did not do so or failed to convince the LCP of its position;

   (f) Where the LCP seeks not only wages but also a penalty as part of the forfeiture, and the contractor or subcontractor has unsuccessfully contended that the cause of violation was a good faith mistake that was promptly corrected when brought to the contractor or subcontractor's attention, a short statement should accompany the proposal for a forfeiture, with a recommended penalty amount pursuant to Labor Code Section 1775(a);

   (g) Where the LCP seeks only wages or a penalty less than $50 per day as part of the forfeiture because the contractor or subcontractor has successfully contended that the cause of the violation was a good faith mistake that was promptly corrected when brought to the contractor or subcontractor's attention, the file should include the evidence as to the contractor or subcontractor's knowledge of his or her obligation, including the program's communication to the contractor or subcontractor of the obligation in the bid invitations, at the pre-job conference agenda and records, and any other notice given as part of the contracting process. With the file should be a statement, similar to that described in (6), and recommended penalty amounts, pursuant to Labor Code Section 1775(a);

   (h) The previous record of the contractor and subcontractor in meeting his or her their prevailing wage obligations; and

   (i) Whether the LCP has been granted initial, extended initial or final approval on only an interim or temporary basis under CCR Sections 16425 or 16426 or whether it has been granted extended approval under CCR Section 16427.

2. The file or report shall be served on the Labor Commissioner as soon as practicable after the violation has been discovered, and not less than 30 days before the final payment, but in no event not less than 30 days before the expiration of the limitations period set forth in Labor Code Section 1741.

3. A copy of the recommended forfeiture and the file or report shall be served on the contractor and subcontractor at the same time as it is sent to the Labor Commissioner. The Labor Compliance Program may exclude from the documents served on the contractor and subcontractor copies of documents secured from the contractor or subcontractor during an audit, investigation, or meeting if those are clearly referenced in the file or report.

4. The Labor Commissioner shall affirm, reject, or modify the forfeiture in whole or in part as the wages and penalties due.

5. The Labor Commissioner's determination of the forfeiture is effective on one of the two following dates:

   (1) For all programs other than those having extended authority under CCR Section 16427 LCPs with initial approval or an extension of initial approval pursuant to CCR Section 16425 or 16426, on the date the Labor Commissioner serves by first class mail, on the LCP, on the Awarding Body if different, on the contractor and on the subcontractor, if any, an endorsed copy of the proposed forfeiture, or a newly drafted forfeiture statement which sets out the amount of forfeiture approved. Service on the contractor or subcontractor is effective if made on the last address supplied by the contractor or subcontractor in the record. The Labor Commissioner's approval, modification or disapproval of the proposed forfeiture shall be served within 30 days of receipt of the proposed forfeiture.

   (2) For programs with final approval extended authority under CCR Section 16427 above, approval is effective 20 days after the requested forfeitures are served upon the Labor Commissioner, unless the Labor Commissioner serves a notice upon the parties, within that time period, that this forfeiture request is subject to further review. For such programs, a notice that approval will follow such a procedure will be included in the transmittal of the forfeiture request to the contractor. If the Labor Commissioner notifies the parties of a decision to undertake further review, the Labor Commissioner's final approval, modification or disapproval of the proposed forfeiture shall be served within 30 days of the date of notice of further review.

F. Deposits of Penalties and Forfeitures Withheld

1. Where the involvement of the Labor Commissioner had been limited to a determination of the actual amount of penalty, forfeiture or underpayment of wages, and the matter has been resolved without litigation by or against the Labor Commissioner, the Labor Compliance Program shall deposit penalties and forfeitures with the Awarding Body.

2. Where collection of fines, penalties or forfeitures results from administrative proceedings or court action to which the Labor Commissioner and Awarding Body or its Labor Compliance Program are both parties, the fines, penalties or forfeitures shall be divided between the general funds of the state and the Awarding Body, as the Hearing Officer or court may decide.

3. All penalties recovered in administrative proceedings or court action brought by or against the Labor Commissioner and to which the Awarding Body or its Labor Compliance Program is not a party, shall be deposited in the general fund of the state.

4. All wages and benefits which belong to an employee and are withheld or collected from a contractor or subcontractor, either by withholding or as a result of administrative proceedings or any court action, and which have not been paid to the employee or irrevocably committed on the employee's behalf to a benefit fund, shall be deposited with the Labor Commissioner, who shall handle such wages and benefits in accordance with Labor Code Section 96.7.

G. Debarment Policy

It is the policy of TSG that the public works prevailing wage requirements set forth in the California Labor Code, Section 1720-1861, be strictly enforced. In furtherance thereof, construction contractors and subcontractors found to be repeat violators of the California Labor Code shall be referred to the Labor Commissioner for debarment from bidding on or otherwise being awarded any public work contract, within the state of California, for the performance of construction and/or maintenance services for the period not to exceed three (3) years in duration. The duration of the debarment period shall depend upon the nature and severity of the labor code violations and any mitigating and/or aggravating factors, which may be presented at the hearing conducted by the Labor Commissioner for such purpose.

SECTION VIII - NOTICE OF WITHHOLDING AND REVIEW THEREOF

After determination of the amount of forfeiture by the Labor Commissioner, notice of withholding of contract payments shall be provided to the contractor and subcontractor, if applicable. The notice shall be in writing and shall describe the nature of the violation and the amount of wages, penalties, and forfeitures withheld. Service of the notice shall be completed pursuant to Section 1013 of the Code of Civil Procedure by first-class and certified mail to the contractor and subcontractor, if applicable. The notice shall advise the contractor and subcontractor, if applicable, of the procedure for obtaining review of the withholding of contract payments. TSG shall also serve a copy of the notice by certified mail to any bonding company issuing a bond that secures the payment of wages covered by the notice and to any surety on a bond, if their identities are known to the awarding body. **A copy of the Notice of Withholding of Contract Payments (NWCP) to be utilized is found as Attachment H to this document.**

Review of NWCP

1. An affected contractor or subcontractor may obtain review of a NWCP under this chapter by transmitting a written request to the office of the LCP that appears on the NWCP within 60 days after service of the NWCP. If no hearing is requested within 60 days after service of the NWCP, the NWCP shall become final.

2. Within ten days following receipt of the request for review, the LCP shall transmit to the Office of the Director-Legal Unit the request for review and copies of the Notice of Withholding of Contract Payments, any audit summary that accompanied the notice, and a proof of service or other documents showing the name and address of any bonding company or surety that secures payment of the wages covered by the notice. **A copy of the required Notice of Transmittal to be utilized is found as Attachment I to this document.**

3. Upon receipt of a timely request, a hearing shall be commenced within 90 days before the Director, who shall appoint an impartial hearing officer possessing the qualifications of an administrative law judge pursuant to subdivision (b) of Section 11502 of the Government Code. The appointed hearing officer shall be an employee of the department, but shall not be an employee of the Division of Labor Standards Enforcement. The contractor or subcontractor shall be provided with an opportunity to review evidence to be utilized by the LCP at the hearing within 20 days of the receipt of the written request for a hearing. Any evidence obtained by the LCP subsequent to the 20-day cutoff shall be promptly disclosed to the contractor or subcontractor. **A copy of a Notice of Opportunity to Review Evidence Pursuant to Labor Code Section 1742(b) form is found as Attachment J to this document.**

   The contractor or subcontractor shall have the burden of proving that the basis for the NWCP is incorrect. The NWCP shall be sufficiently detailed to provide fair notice to the contractor or subcontractor of the issues at the hearing.

Within 45 days of the conclusion of the hearing, the Director shall issue a written decision affirming, modifying, or dismissing the assessment. The decision of the Director shall consist of a notice of findings, findings, and an order. This decision shall be served on all parties pursuant to Section 1013 of the Code of Civil Procedure by first-class mail at the last known address of the party on file with the LCP. Within 15 days of the issuance of the decision, the Director may reconsider or modify the decision to correct an error, except that a clerical error may be corrected at any time. The Director has adopted regulations setting forth procedures for hearings**.**

4. An affected contractor or subcontractor may obtain review of the decision of the Director by filing a petition for a writ of mandate to the appropriate superior court pursuant to Section 1094.5 of the Code of Civil Procedure within 45 days after service of the decision. If no petition for writ of mandate is filed within 45 days after service of the decision, the order shall become final. If it is claimed in a petition for writ of mandate that the findings are not supported by the evidence, abuse of discretion is established if the court determines that the findings are not supported by substantial evidence in the light of the whole record.

5. A certified copy of a final order may be filed by the Labor Commissioner in the office of the clerk of the superior court in any county in which the affected contractor or subcontractor has property or has or had a place of business. The clerk, immediately upon the filing, shall enter judgment for the state against the person assessed in the amount shown on the certified order.

6. A judgment entered pursuant to this procedure shall bear the same rate of interest, shall have the same effect as other judgments, and shall be given the same preference allowed by law on other judgments rendered for claims for taxes. The clerk shall not charge for the service performed by him or her pursuant to this section.

7. This procedure shall provide the exclusive method for review of a NWCP to withhold contract payments pursuant to Labor Code Section 1771.7.

SECTION IX - DISTRIBUTION OF FORFEITED SUMS

1. Before making payments to the contractor of money due under a contract for public works, all amounts required to satisfy the NWCP shall be withheld and retained there from. The amounts required to satisfy the NWCP shall not be disbursed until receipt of a final order that is no longer subject to judicial review.

2. Pending a final order, or the expiration of the time period for seeking review of the notice of the withholding, any contract payments withheld shall not be disbursed.

3. From the amount recovered, the wage claim shall be satisfied prior to the amount being applied to penalties. If insufficient money is recovered to pay each worker in full, the money shall be prorated among all workers employed on the public works project who are paid less than the prevailing wage rate. The wage claim shall have **PRIORITY** over all Stop Notices filed against the prime contractor.

4. Wages for workers who cannot be located shall be placed in the Industrial Relations Unpaid Fund and held in trust for the workers pursuant to Labor Code Section 96.7. Penalties shall be paid into the General Fund of the School District that has enforced this chapter pursuant to Labor Code Section 1771.7.

SECTION X - OUTREACH ACTIVITIES

To ensure the successful implementation of the LCP, there shall be several outreach activities initiated and maintained.

A. Providing Information to the Public

The LCO shall be responsible for communication and outreach activities relative to public information on the LCP:

1. Regular presentations to contractors at all Job Walk Meetings (Pre-Bid conferences) and Job Start Meetings (Pre-Job conferences);

2. Ongoing communication via correspondence and with workers at job sites when review of the certified payroll records reveals the possibility of prevailing wage violations;

3. Periodic meetings with contractor organizations, prime contractors, and subcontractors interested in public works contracting.

B. In-Service Management Training on the Labor Compliance Program

TSG shall provide ongoing management in-servicing and workshops for project personnel relative to the terms, requirements and administration of the Labor Compliance Program. TSG shall require all personnel to document their review of TSG's Labor Compliance Policies and Procedures Manual, which outlines the responsibilities and procedures of the Labor Compliance Program. Whenever TSG's Labor Compliance Policies and Procedures Manual is updated to accurately reflect any amendments to the public works laws and regulations (including the laws and regulations governing Labor Compliance Programs), TSG personnel will be required to review and sign-off on the record sheet to indicate that they have undergone the latest training.

SECTION X - ANNUAL REPORTS

The LCP will submit to the Director an annual report on its operation within 60 days after the close of its annual reporting period, as defined in subpart (3) and (4) below. The annual report shall be made on the appropriate form [LCP-AR3, included as Attachment L], unless the Director has agreed to a different reporting format for a Program that has been granted extended authority under CCR Section 16427. A third party LCP that contracted with more than one Awarding Body or Joint Powers Authority during the annual reporting period shall separately report on Labor Code Section 1771.5(b) enforcement activities for each Awarding Body or Joint Powers Authority covered by the report.

The Annual Report for a person or entity operating a third party Labor Compliance Program shall also include: (1) a certification of compliance with conflict of interest disclosure requirements by employees and consultants who participate in making governmental decisions, as defined under Title 2, CCR Section 18701, and (2) a current statement disclosing the information required under CCR Section 16426(a)(2), (3) and (5).

Information in the Annual Report will be reported in sufficient detail to afford a basis for evaluating the scope and level of enforcement activity of the LCP. An annual report shall also include such additional information as the LCP may be required to report as a condition of its approval.

For purposes of this section, the annual reporting period shall be deemed to commence on the first of the month in which the LCP is first granted approval pursuant to CCR Section 16425 or 16426 and shall conclude on the last day of the month immediately preceding that date in the following year.

The Solis Group

Labor Compliance Program

FORMS

LABOR COMPLIANCE PROGRAM
JOB-START/PRE-CONSTRUCTION MEETING PACKET
CONTRACTOR HANDOUT

Included in this packet are materials to assist your company in complying with the Awarding Body's Labor Compliance Program (LCP). The Solis Group monitors and enforces the Awarding Body's LCP.

Have questions about how to comply with the LCP? Call the Lead Labor Compliance Officer assigned to monitor the project at (626) 685-6989.

PACKET INCLUDES:

1. Prevailing Wage / DIR Determination Website Link

2. Sample Prevailing Wage Determination

3. Pre-Construction Meeting Labor Compliance Responsibilities & Requirements

4. Prevailing Wage Contractor Handout

5. Division of Apprenticeship Standards – Apprenticeship Requirements

6. Required LCP Submittal Forms (Blank/Sample Forms)
   - Attachment 1: Forms Submittal Schedule
   - Attachment 2: LCP Checklist
   - Attachment 3: DAS-140
   - Attachment 4: DAS-142
   - Attachment 5: CAC-2
   - Attachment 6: Statement of Compliance
   - Attachment 7: Certified Payroll Record
   - Attachment 8: Statement of Non-Performance
   - Attachment 9: Fringe Benefit Statement
   - Attachment 10: Monthly Trustee Report
   - Attachment 11: Monthly Employment Utilization Reporting Form
   - Attachment 12: Public Works Affidavit

PREVAILING WAGE
DIR DETERMINATION WEBSITE LINK

http://www.dir.ca.gov/DLSR/statistics_research.html

The first bid advertisement date establishes which Department of Industrial Relations (DIR) determination to use for each work classification you assign to this project.

All contractors working on the _____ **Project** must refer to the following DIR determination:

200__ - __ / Prevailing Wage Determinations

(i.e. 2007-2 General prevailing wage determinations menu (journeyman) /
2007-2 General prevailing apprentice schedule menu).

NOTE: Contractors are held responsible for any/all increases to the prevailing wages. If you see a double asterisk after the determination expiration date (**), there will be an increase to the prevailing wages. If you have any questions, please call the Labor Compliance Officers assigned to monitor this project, at (626) 685-6989.

SAMPLE PREVAILING WAGE DETERMINATION

GENERAL PREVAILING WAGE DETERMINATION MADE BY THE DIRECTOR OF INDUSTRIAL RELATIONS
PURSUANT TO CALIFORNIA LABOR CODE PART 7, CHAPTER 1, ARTICLE 2, SECTIONS 1770, 1773 AND 1773.1
FOR COMMERCIAL BUILDING, HIGHWAY, HEAVY CONSTRUCTION AND DREDGING PROJECTS

CRAFT: # CARPENTER AND RELATED TRADES

DETERMINATION: SC-23-31-2-2002-1
ISSUE DATE: August 22, 2002
EXPIRATION DATE OF DETERMINATION: June 30, 2003** The rate to be paid for work performed after this date has been determined. If work will extend past this date, the new rate must be paid and should be incorporated in contracts entered into now. Contact the Division of Labor Statistics and Research for specific rates (415) 703-4774.

LOCALITY: All localities within Imperial, Inyo, Kern, Los Angeles, Mono, Orange, Riverside, San Bernardino, San Luis Obispo, Santa Barbara, and Ventura counties.

Classification (Journeyperson)	Basic Hourly Rate	Employer Payments: Health and Welfare	Pension	Vacation/ Holiday	Training	Straight-Time: Hours	Total Hourly Rate	Overtime Hourly Rate: Daily 1 1/2X	Saturday [b] 1 1/2X	Sunday and Holiday
[a] **AREA 1**										
Carpenter, Cabinet Installer, Insulation Installer, Hardwood Floor Worker, Acoustical Installer	[c] $29.00	2.45	1.01	2.88[f]	0.34	8	35.68	50.18	50.18	64.68
Pile Driverman, Derrick Bargeman, Rockslinger, Bridge or Dock Carpenter, Cable Splicer	29.13	2.45	1.01	2.88[f]	0.34	8	35.81	50.375	50.375	64.94
Bridge Carpenter	[c] 29.13	2.45	1.01	2.88[f]	0.34	8	35.81	50.375	50.375	64.94
Shingler	[c] 29.13	2.45	1.01	2.88[f]	0.34	8	35.81	50.375	50.375	64.94
Saw Filer	29.08	2.45	1.01	2.882[f]	0.34	8	35.76	50.30	50.30	64.84
Table Power Saw Operator	29.10	2.45	1.01	2.88[f]	0.34	8	35.78	50.33	50.33	64.88
Pneumatic Nailer or Power Stapler	29.25	2.45	1.01	2.88[f]	0.34	8	35.93	50.555	50.555	65.18
Roof Loader of Shingles	20.39	2.45	1.01	2.88[f]	0.34	8	27.07	37.265	37.265	47.46
Scaffold Carpenter	23.20	2.45	1.01	2.88[f]	0.34	8	29.88	41.48	41.48	53.08
Millwright	[c] 29.50	2.45	1.01	2.88[f]	0.34	8	36.18	50.93	50.93	65.68
Head Rockslinger	29.23	2.45	1.01	2.88[f]	0.34	8	35.91	50.525	50.525	65.14
Rock Bargeman or Scowman	29.03	2.45	1.01	2.88[f]	0.34	8	35.71	50.225	50.225	64.74
Diver, Wet (Up To 50 Ft. Depth) [d]	[c] 64.26	2.45	1.01	2.88[f]	0.34	8	70.94	103.07	103.07	135.20
Diver, (Stand-By)	[c] 32.13	2.45	1.01	2.88[f]	0.34	8	38.81	54.875	54.875	70.94
Diver's Tender	[c] 31.13	2.45	1.01	2.88[f]	0.34	8	37.81	53.375	53.375	68.94
[a] **AREA 2**										
Carpenter, Cabinet Installer, Insulation Installer, Hardwood Floor Worker, Acoustical Installer	[c] 28.43	2.45	1.01	2.88[f]	0.34	8	35.11	49.325	49.325	63.54
Shingler	[c] 28.56	2.45	1.01	2.88[f]	0.34	8	35.24	49.52	49.52	63.80
Saw Filer	28.51	2.45	1.01	2.88[f]	0.34	8	35.19	49.445	49.445	63.70
Table Power Saw Operator	28.53	2.45	1.01	2.88[f]	0.34	8	35.21	49.475	49.475	63.74
Pneumatic Nailer or Power Stapler	28.68	2.45	1.01	2.88[f]	0.34	8	35.36	49.70	49.70	64.04
Roof Loader of Shingles	20.37	2.45	1.01	2.88[f]	0.34	8	27.05	37.235	37.235	47.42

DETERMINATION: SC-31-741-1-2003-1
ISSUE DATE: February 22, 2003
EXPIRATION DATE OF DETERMINATION: May 31, 2003* Effective until superseded by a new determination issued by the Director of Industrial Relations. Contact the Division of Labor Statistics and Research (415) 703-4774 for the new rates after 10 days from the expiration date, if no subsequent determination is issued.

LOCALITY: All localities within Imperial, Inyo, Kern, Los Angeles, Mono, Orange, Riverside, San Bernardino, San Luis Obispo, Santa Barbara, and Ventura counties.

Classification (Journeyperson)	Basic Hourly Rate	Employer Payments: Health and Welfare	Pension	Vacation/ Holiday	Training	Straight-Time: Hours	Total Hourly Rate	Overtime Hourly Rate: Daily 1 1/2x	Saturday/ Sunday [b] 1 1/2x	Holiday 2X
Terrazzo Installer	$30.84	2.45	1.01	2.88[f]	-	8	37.18	52.60	52.60	68.02
Terrazzo Finisher	24.34	2.45	1.01	2.88[f]	-	8	30.68	42.85	42.85	55.02

# Indicates an apprenticeable craft. Rates for apprentices are available in the General Prevailing Wage Apprentice Schedules.

a **AREA 1** - Imperial, Los Angeles, Orange, Riverside, San Bernardino, San Luis Obispo, Santa Barbara and Ventura counties.
AREA 2 - Inyo, Kern, and Mono counties. For Bridge Carpenter, Scaffold Carpenter, Pile Driverman, Derrick Bargeman, Rockslinger, Bridge or Dock Carpenter, Cable Splicer, Millwright, Head Rockslinger, Rock Bargeman or Scowman, Diver, Wet (Up to 50 Ft. Depth), Diver (Stand-By), and Diver's Tender rates, please see **Area 1** as this rate applies to **Area 2** as well. Basic Hourly Rates for **Area 2** include an additional amount deducted for vacation/holiday.

b Saturdays in the same work week may be worked at straight-time rates if a job is shut down during the normal work week due to inclement weather, major mechanical breakdown or lack of materials beyond the control of the Employer.

c When performing welding work requiring certification, carpenters and pile drivers will receive an additional $1.00 per hour.

d Shall receive a minimum of 8 hours pay for any day or part thereof.

e For specific rates over 50 ft depth, contact the Division of Labor Statistics and Research. Rates for Technicians, Manifold Operators, Pressurized Submersible Operators, Remote Control Vehicle Operators, and Remote Operated Vehicle Operators, as well as rates for Pressurized Bell Diving and Saturation Diving are available upon request.

f Includes an amount for supplemental dues.

RECOGNIZED HOLIDAYS: Holidays upon which the general prevailing hourly wage rate for Holiday work shall be paid, shall be all holidays in the collective bargaining agreement, applicable to the particular craft, classification, or type of worker employed on the project, which is on file with the Director of Industrial Relations. If the prevailing rate is not based on a collectively bargained rate, the holidays upon which the prevailing rate shall be paid shall be as provided in Section 6700 of the Government Code. You may obtain the holiday provisions for the current determinations on the Internet at http://www.dir.ca.gov/DLSR/PWD. Holiday provisions for current or superseded determinations may be obtained by contacting the Prevailing Wage Unit at (415) 703-4774.

TRAVEL AND/OR SUBSISTENCE PAYMENT: In accordance with Labor Code Sections 1773.1 and 1773.9, contractors shall make travel and/or subsistence payments to each worker to execute the work. You may obtain the travel and/or subsistence provisions for the current determinations on the Internet at http://www.dir.ca.gov/DLSR/PWD. Travel and/or subsistence requirements for current or superseded determinations may be obtained by contacting the Prevailing Wage Unit at (415) 703-4774.

PRE-CONSTRUCTION MEETING
LABOR COMPLIANCE RESPONSIBILITIES & REQUIREMENTS

I. REQUIRED SUBMITTALS (Submittal Schedule & Forms)

A. LCP Checklist – Submit at Pre-Construction Meeting (Review and sign)

1. Payment of Prevailing Wage Rates
   - Public Works Project requires the payment of prevailing wages to all employees
   - The prevailing wage determination applicable to this project can be found on the California Department of Industrial Relations website at: http://www.dir.ca.gov/DLSR/statistics_research.html
   - **You must refer to the _____ Determination**
2. Apprentices
   - All contractors must make a "good faith effort" to employ apprentices (if craft is apprenticeable).
   - Submit a DAS-140 (Public Works Contract Award Information) to your appropriate Apprenticeship Committee. Submit copy of your certified mail certificate as proof of your submittal.
3. Penalties
4. CPRs
5. Non-Discrimination in Employment
6. Kickbacks Prohibited
7. Acceptance of Fees Prohibited
8. Listing of Subcontractors
9. Proper Licensing
10. Unfair Competition Prohibited
11. Workers' Compensation Insurance
12. OSHA
13. Hiring Undocumented Workers
14. Itemized Wage Statements to Employees
15. Public Works Affidavit

B. DAS-140 & DAS-142 – Submit before you start work on the project.

- Requirement to "announce" that you will be working on a public works project (compliance is proven via submittal of DAS-140)
- Requirement to "request the dispatch of apprentices" for this public works project (compliance is proven via submittal of DAS-142 or via verbal request)
- These forms should be sent to the local apprenticeship committee for your craft.
- If you need the contact information for your applicable apprenticeship committee, call TSG at (626) 685-6989

C. CAC-2 – Submit each month, or 1-time submittal (at beginning or end of project)

- For every craft and every hour worked in that craft, you must make the appropriate training fund contributions. Record the total number of hours workers worked on the job, multiply by the appropriate training amount for that craft, and send a check to appropriate fund (see below).
- *Union Contractors:* On the CAC-2, indicate which Union Trust Fund you make training fund contributions to. TSG will audit Trust Fund Reports.
- *Non-Union contractors:* Must make training fund contributions to the California Apprenticeship Council. Send a copy of the CAC-2 form & copy of the contribution check to TSG.

D. CPRs & Statements of Compliance – Submit for each week you work on the project.

- *CPRs must be submitted for every week that work was performed on the project.*

- *Make sure that you include an accurate work classification for all your workers. (e.g. 'Laborer Group 2', 'Laborer Apprentice level 1' or 'apprentice 65%'*
- *CPRs are not valid unless accompanied by a signed statement of compliance*
- *CPRs must be submitted even on the weeks in which you did not perform any work – This is called a non-performance payroll.*
- *The last payroll should be marked as "final" – when we see the final payroll it triggers our closeout procedures.*

E. Fringe Benefit Statement - Submit with your 1st CPR.

- *Indicate each benefit amount for every individual craft you employ on this project.*
- *If fringe benefits are paid to workers in cash, please write that next to the benefit dollar amounts on the form.*
- *If fringe benefits are paid to a Union or Employer's Trust Fund on behalf of your workers, write the contact information in the designated places.*
- *Note: Training Fund Contributions are never paid to the employees.*

F. Trustee Reports – Submit monthly (typically 2 months in the rear)

- *If fringe benefits are paid to a Union or Employer's Trust Fund on behalf of your workers, Trust Fund Reports & copies of the contribution checks must be submitted monthly – this proves the benefits were paid for every hour worked on this project.*

G. Public Works Affidavit – Submit with your "final" CPR

- *Once TSG has received all the required submittals from your firm, and has resolved any outstanding issues, TSG will sign-off on the PWA & forward it to the District along with a Request for Release of Retention.*

II. PENALTIES

Goal: Resolve all issues before they reach the penalty stage. If workers are underpaid, we will give you the opportunity to correct the situation. However, if we do not receive the documents we need in a timely manner, or do not receive responses to our letters, we will not hesitate to impose penalties. Penalties can add up, ranging from $25 a day per worker to $50 dollars a day per worker.

LABOR COMPLIANCE PROGRAM
The Solis Group
234 North El Molino Avenue, Suite 202
Pasadena, CA 91101

Phone: (626) 685-6989
Fax: (626) 685-6985

The Solis Group

PREVAILING WAGE CONTRACTOR HANDOUT

PUBLIC WORKS REQUIREMENTS:

(1) The appropriate number of apprentices are on the job site, as set forth in Labor Code Section 1777.5;
(2) Worker's compensation coverage, as set forth in Labor Code Sections 1860 and 1861;
(3) Keep accurate records of the work performed on the public works project, as set forth in Labor Code Section 1812;
(4) Inspection of payroll records pursuant to Labor Code Section 1776, and as set forth in 8 CCR Section 16400(e);
(5) Withhold monies. See Labor Code Section 1727;
(6) Ensure that public works projects are not split or separated into smaller work orders or projects for the purpose of evading the applicable provisions of Labor Code Section 1771;
(7) Deny the right to bid on public work contracts to contractors or subcontractors who have violated public work laws, as set forth in Labor Code Section 1777.7;
(8) Not permit workers on public works to work more than eight hours a day or 40 hours in any one calendar week, unless compensated at not less than time and a half as set forth in Labor Code Section 1815.
Exception: If the prevailing wage determination requires a higher rate of pay for overtime work than is required under Labor Code Section 1815, then that higher overtime rate must be paid [as specified in 16200(a)(3)(F)];
(9) Not take or receive any portion of the workers' wages or accept a fee in connection with a public works project, as set forth in Labor Code Sections 1778 and 1779;
(10) Comply with those requirements as specified in Labor Code Sections 1776(g), 1777.5, 1810, 1813, and 1860; and
(11) Other requirements imposed by law.

THE CONTRACTOR AND SUBCONTRACTOR SHALL:

(1) Pay not less than the prevailing wage to all workers, as defined in the California Code of Regulations Section 16000(a), and as set forth in Labor Code Sections 1771 and 1774;
(2) Comply with the provisions of Labor Code Sections 1773.5, 1775, and 1777.5 regarding public works job sites;
(3) Provide workers' compensation coverage as set forth in Labor Code Section 1861;
(4) Comply with Labor Code Sections 1778 and 1779 regarding receiving a portion of wages or acceptance of a fee;
(5) Maintain and make available for inspection payroll records, as set forth in Labor Code Section 1776;
(6) Pay workers overtime pay, as set forth in Labor Code Section 1815 or as provided in the collective bargaining agreement adopted by the Director as set forth in 8 CCR Section 16200(a)(3);
(7) Comply with Section 16101 of these regulations regarding discrimination.
(8) Be subject to provisions of Labor Code Section 1777.7 which specifies the penalties imposed on a contractor who willfully fails to comply with provisions of Section 1777.5;
(9) Comply with those requirements as specified in Labor Code Sections 1810 and 1813; and
(10) Comply with other requirements imposed by law.

APPRENTICE TRAINING:

SEE LABOR CODE SECTION 1777.5 (e)

Prior to commencing work on a contract for public works, every contractor shall submit contract award information (via submittal of a DAS-140) to an applicable apprenticeship program that can supply apprentices to the site of the public work. The information submitted shall include an estimate of journeyman hours to be performed under the contract, the number of apprentices proposed to be employed, and the approximate dates the apprentices would be employed. A copy of this information shall also be submitted to the awarding body if requested by the awarding body. Within 60 days after concluding work on the contract, each contractor and subcontractor shall submit to the awarding body, if requested, and to the apprenticeship program a verified statement of the journeyman and apprentice hours performed on the contract. The information under this subdivision shall be public. The apprenticeship programs shall retain this information for 12 months.

APPRENTICE TRAINING CONTRIBUTION REQUIREMENTS:

SEE CALIFORNIA CODE OF REGULATIONS: TITLE 8, ARTICLE 4,
16200(G) **Wage rates, training contributions and apprenticeship contributions.**

Apprenticeship rates shall be determined by the Director of Industrial Relations using apprentice wage standards set forth in the collective bargaining agreement and/or approved by the California Apprenticeship Council. A contractor or subcontractor on a public works contract must pay training fund contributions or apprenticeship contributions in one of the following manners:

1. Into the appropriate craft apprenticeship program in the area of the site of the public work; or
2. An equivalent amount shall be paid to the California Apprenticeship Council (CAC) administered by DAS (if the trust fund is unable to accept such contributions).
3. If neither of the above will accept the funds, cash pay shall be as provided for in the California Code of Regulations Section 16200(a)(3)(I).

SEE CALIFORNIA CODE OF REGULATIONS: TITLE 8, ARTICLE 10, SECTION 230.2
§230.2. **Payment of Apprenticeship Training Contributions to the Council.**

(a) Contractors who are neither required nor wish to make apprenticeship training contributions to the applicable local training trust fund shall make their training contributions to the Council. Contractors may refer to the Director of the Department of Industrial Relations applicable prevailing wage determination for the amount owed for each hour of work performed by journeymen and apprentices in each apprenticeable occupation.

(b) Training contributions to the Council are due and payable on the 15th day of each month for work performed during the preceding month.

(c) Training contributions to the Council shall be paid by check and shall be accompanied by a completed CAC-2 Form, Training Fund Contributions or contain the following information:
   (1) The name, address, and telephone number of the contractor making the contribution.
   (2) The contractor's license number.
   (3) The name and address of the public agency that awarded the contract.
   (4) The jobsite location, including the county where the work was performed.
   (5) The contract or project number.
   (6) The time period covered by the enclosed contributions.
   (7) The contribution rate and total hours worked by apprenticeable occupation.

CERTIFYING PERSON

SEE CALIFORNIA CODE OF REGULATIONS: TITLE 8, GROUP 3, ARTICLE 1, SECTION 16000 DEFINITIONS.

A person with the authority to affirm under penalty of perjury that the records provided, depict truly, fully and correctly the type of work performed, the hours worked, days worked and amounts paid.

CHANGES TO PREVAILING RATE AFTER AWARD

SEE LABOR CODE SECTION: 1773.6

1773.6. If during any quarterly period the Director of Industrial Relations shall determine that there has been a change in any prevailing rate of per diem wages in any locality, he shall make such change available to the awarding body and his determination shall be final. Such determination by the Director of Industrial Relations shall not be effective as to any contract for which the notice to bidders has been published. *Exceptions: classifications marked as double asterisks.*

CREDITS, FOR FRINGE BENEFIT PAYMENTS

SEE CALIFORNIA CODE OF REGULATIONS: TITLE 8, GROUP3, ARTICLE 4

16200(i) Credit Available For Actual Payment of Fringe Benefit Costs up to the Prevailing Amount. The contractor obligated to pay the full prevailing rate of per diem wages may take credit for amounts up to the total of all fringe benefit amounts listed as prevailing in the appropriate wage determination. This credit may be taken only as to amounts which are actual payments under Employer Payments Section 16000(1)-(3). In the event that the total of Employer Payments by a contractor for the fringe benefits listed as prevailing is less than the aggregate amount set out as prevailing in the wage determination, the contractor must pay the difference directly to the employee. No amount of credit for payments over the aggregate amount of employer payments shall be taken nor shall any credit decrease the amount of direct payment of hourly wages of those amounts found to be prevailing for straight time or overtime wages.

THE RULE:

The contractor can pay amounts for individual benefits different than the state shows in the wage reports so long as it is not more than the total amount permitted for all benefits. Any contractor paid amount less than the total benefit requirements listed in the state wage reports must be paid to the employee.

EMPLOYEE'S SUBJECT TO PREVAILING WAGES

SEE LABOR CODE SECTION 1771, 1772 & 1776

All workers on the project shall be paid the wage of the trade they are most closely related to. This includes: any one on site, and off site even at remote manufacturing facilities.

1771. Except for public works projects of one thousand dollars ($1,000) or less, not less than the general prevailing rate of per diem wages for work of a similar character in the locality in which the public work is performed, and not less than the general prevailing rate of per diem wages for holiday and overtime work fixed as provided in this chapter, shall be paid to all workers employed on public works.

1772. Workers employed by contractors or subcontractors in the execution of any contract for public work are deemed to be employed upon public work.

1776. (a) Each contractor and subcontractor shall keep an accurate payroll record, showing the name, address, social security number, work classification, and straight time and overtime hours worked each day and week, and the actual per diem wages paid to each journeyman, apprentice, worker, or other employee employed by him or her in connection with the public work.

EMPLOYER PAYMENTS

SEE CALIFORNIA CODE OF REGULATIONS: TITLE 8, ARTICLE 1, SECTION 16000 DEFINITIONS

(1) The rate of contribution irrevocably made by a contractor or subcontractor to a trustee or to a third person pursuant to a fund, plan, or program for the benefit of employees, their families and dependents, or retirees;

(2) The rate of costs to the contractor or subcontractor which may be reasonably anticipated in providing benefits to employees, their families and dependents or to retirees pursuant to an enforceable commitment or agreement to carry out a financially responsible plan or program which was communicated in writing to the workers affected; and

(3) The rate of contribution irrevocably made by the contractor or subcontractor for apprenticeship or other training programs authorized by Section 3071 and/or 3093 of the Labor Code.

FRINGE BENEFIT PAYMENT REQUIREMENTS

SEE CALIFORNIA CODE OF REGULATIONS: TITLE 8, ARTICLE 1, SECTION 16000 DEFINITIONS

All fringe benefits must be irrevocably paid to an authorized fund or to the employee.
No unpaid amounts are allowed.

FRINGE BENEFITS INCLUDE

CALIFORNIA CODE OF REGULATIONS: TITLE 8, ARTICLE 1, SECTION 16000. DEFINITIONS

The prevailing rate of employer payments for any or all programs or benefits for employees, their families and dependents, and retirees which are of the types enumerated below:

(1) Medical and hospital care, prescription drugs, dental care, vision care, diagnostic services, and other health and welfare benefits;
(2) Retirement plan benefits;
(3) Vacations and holidays with pay, or cash payments in lieu thereof;
(4) Compensation for injuries or illnesses resulting from occupational activity;
(5) Life, accidental death and dismemberment, and disability or sickness and accident insurance;
(6) Supplemental unemployment benefits;
(7) Thrift, security savings, supplemental trust, and beneficial trust funds otherwise designated, provided all of the money except that used for reasonable administrative expenses is returned to the employees;
(8) Occupational health and safety research, safety training, monitoring job hazards, and the like, as specified in the applicable collective bargaining agreement;
(9) See definition of "Employer Payments," (3).
(10) Other bonafide benefits for employees, their families and dependents, or retirees as the Director may determine; and
(11) Travel time and subsistence pay as provided for in Labor Code Section 1773.8.

FRINGE BENEFITS DO NOT INCLUDE

CALIFORNIA CODE OF REGULATIONS: TITLE 8, ARTICLE 1, SECTION 16000. DEFINITIONS

The term "general prevailing rate of per diem wages" does not include any employer payments for:

(1) Job related expenses other than travel time and subsistence pay;
(2) Contract administration, operation of hiring halls, grievance processing, or similar purposes except for those amounts specifically earmarked and actually used for administration of those types of employee or retiree benefit plans enumerated above;
(3) Union, organizational, professional or other dues except as they may be included in and withheld from the basic taxable hourly wage rate;
(4) Industry or trade promotion;
(5) Political contributions or activities;

(6) Any benefit for employees, their families and dependents, or retirees including any benefit enumerated above where the contractor or subcontractor is required by Federal, State, or local law to provide such benefit; or
(7) Such other payments as the Director may determine to exclude.

PAYROLL RECORDS INCLUDE

CALIFORNIA CODE OF REGULATIONS: TITLE 8, ARTICLE 1, SECTION 16000. DEFINITIONS

All time cards, bank certified cancelled checks, cash receipts, trust fund forms, books, documents, schedules, forms, reports, receipts or other evidences which reflect job assignments, work schedules by days and hours, and the disbursement by way of cash, check, or in whatever form or manner, of funds to a person(s) by job classification and/or skill pursuant to a public works project.

PERSONS REQUIRED TO RECEIVE PREVAILING WAGES

SEE LABOR CODE SECTIONS:

1771. Prevailing wages shall be paid to all workers employed on public works.
1774. The contractor to whom the contract is awarded, and any subcontractor under him, shall pay not less than the specified prevailing rates of wages to all workmen employed in the execution of the contract.

WITHHOLDING PAYMENTS, JUSTIFICATION

SEE LABOR CODE SECTION: 1727 & 1771.5(b),(5)
SEE CALIFORNIA CODE OF REGULATIONS: TITLE 8, ARTICLE 5, SECTION 16435

16435(a) "Withhold" means to cease payments by the awarding body, or others who pay on its behalf, or agents, to the general contractor. Where the violation is by a subcontractor, the general contractor shall be notified of the nature of the violation and reference made to its rights under Labor Code Section 1729.

(1) "Contracts." Except as otherwise provided by agreement, only contracts under a single master contract, or contracts entered into as stages of a single project, may be the subject of withholding.
(2) "Delinquent payroll records" means those not submitted on the date set in the contract.
(3) "Inadequate payroll records" are any one of the following:
(4) A record lacking the information required by Labor Code Section 1776;
(5) A record which contains the required information but is not certified, or certified by someone not an agent of the contractor or subcontractor;
(6) A record remaining uncorrected for one payroll period, after the awarding body has given the contractor notice of inaccuracies detected by audit or record review, provided, however, that prompt correction will stop any duty to withhold if such inaccuracies do not amount to one percent of the entire Certified Weekly Payroll in dollar value and do not affect more than half the persons listed as workers employed on that Certified Weekly Payroll, as defined in Labor Code Section 1776 and Title 8 CCR Section 16401.

DIRECTOR OF INDUSTRIAL RELATIONS PRECEDENTIAL DECISIONS WHICH REQUIRE PREVAILING WAGES:

Decision 92-036: stands for the payment of out-of-state workers if they are working on California "Public Works"

Decision 93-019: stands for the payment of truck drivers removing, delivering or relocating material on "Public Works"

Decision 94-017: stands for the payment of waste processors off site if the waste is exclusively from "Public Works"

COURT DECISIONS:

Standard Traffic Services v. Department of Transportation (case 132667) Shasta: partners are due prevailing wages if working on "Public Works"

STATE OF CALIFORNIA
DEPARTMENT OF INDUSTRIAL RELATIONS
DIVISION OF APPRENTICESHIP STANDARDS
28 CIVIC CENTER PLAZA, ROOM 525
SANTA ANA, CA 92701

TO ALL PUBLIC WORKS CONTRACTORS

Congratulations on having been awarded a public works project.

The Division of Apprenticeship Standards wishes to bring to your attention your responsibilities under California Labor Code Section 1777.5 Apprentices on Public Works. (Excerpts from California Labor Code relating to apprentices on public works. DAS-10 is attached).

Compliance with California Labor Code Section 1777.5 requires all public works contractors and subcontractors to:

- Submit contract award information within 10 days of contract award, to the applicable Joint Apprenticeship Committee, which shall include an estimate of Journeyman hours to be performed under the contract, the number of apprentices to be employed, and the approximate dates the apprentices will be employed. This information may be submitted on the attached form. DAS 140.

- Employ apprentices on the public work in a ratio to journeymen of no less than one hour of apprentices work for every five hours of labor performed by a journeyman.

- Pay the apprentice rate on public works projects only to those apprentices who are registered as defined in Labor Code Section 3077.

- Contribute to the training fund in the amount identified in the Prevailing Wage Rate publication for journeymen and apprentices. Contractors who choose not to contribute to the local training trust fund must make their contribution to the California Apprenticeship Council (CAC) at P.O. Box 420603, San Francisco, CA 94142.

Training fund contributions to the CAC are due and payable on the 15th day of each month for work performed during the preceding month.

Training fund contributions to the CAC shall be paid by check and shall be accompanied by a completed form CAC-2 (attached).

Failure to comply with the provisions of the Labor Code Section 1777.5 may result in the loss of the right to bid on all public works projects for a period of one to three years and the imposition of a civil penalty of $100.00 for each calendar day of noncompliance. Contractors should provide a copy of this material to each subcontractor.

If the Division of Apprenticeship Standards can be of assistance to you, please contact our office at (714) 558-4126.

STATE OF CALIFORNIA – DEPARTMENT OF INDUSTRIAL RELATIONS – DIVISION OF APPRENTICESHIP STANDARDS

EXERPTS FROM THE CALFORNIA LABOR CODE RELATING TO APPRENTICES ON PUBLIC WORKS

Chapter 1 of Division 2

APPRENTICES ON PUBLIC WORKS

1773.3. An awarding agency whose public works contract falls within the jurisdiction of Section 1777.5 shall, within five days of the award, send a copy of the award to the Division of Apprenticeship Standards. When specifically requested by a local joint apprenticeship committee, the division shall notify the local joint apprenticeship committee regarding all such awards applicable to the joint apprenticeship committee making the request. Within five days of a finding of any discrepancy regarding the ratio of apprentices to journeymen, pursuant to the certificated fixed number of apprentices to journeymen, the awarding agency shall notify the Division of Apprenticeship Standards.

1776. (a) Each contractor and subcontractor shall keep accurate payroll records, showing the name, address, social security number, work classification, straight time and overtime hours worked each day and week, and the actual per diem wages paid to each journeyman, apprentice, worker, or other employee employed by him or her in connection with the public work. Each payroll record shall contain or be verified by a written declaration that it is made under penalty of perjury, stating both of the following:

(1) The information contained in the payroll record is true and correct.

(2) The employer has complied with the requirements of Sections 1771, 1811, and 1815 for any work performed by his or her employees on the public works project.

(b) The payroll records enumerated under subdivision (a) shall be certified and shall be available for inspection at all reasonable hours at the principal office of the contractor on the following basis:

(1) A certified copy of an employee's payroll record shall be made available for inspection or furnished to the employee or his or her authorized representative on request.

(2) A certified copy of all payroll records enumerated in subdivision (a) shall be made available for inspection or furnished upon request to a representative of the body awarding the contract, the Division of Labor Standards Enforcement, and the Division of Apprenticeship Standards of the Department of Industrial Relations.

(3) A certified copy of all payroll records enumerated in subdivision (a) shall be made available upon request by the public for inspection or for copies thereof. However, a request by the public shall be made through either the body awarding the contract, the Division of Apprenticeship Standards, or the Division of Labor Standards Enforcement. If the requested payroll records have not been provided pursuant to paragraph (2), the requesting party shall, prior to being provided the records, reimburse the costs of preparation by the contractor, subcontractors, and the entity through which the request was made. The public shall not be given access to the records at the principal office of the contractor.

(c) The certified payroll records shall be on forms provided by the Division of Labor Standards Enforcement or shall contain the same information as the forms provided by the division.

(d) A contractor or subcontractor shall file a certified copy of the records enumerated in subdivision (a) with the entity that requested the records within 10 days after receipt of a written request.

(e) Any copy of records made available for inspection as copies and furnished upon request to the public or any public agency by the awarding body, the Division of Apprenticeship Standards, or the Division of Labor Standards Enforcement shall be marked or obliterated in a manner so as to prevent disclosure of an individual's name, address, and social security number. The name and address of the contractor awarded the contract or the subcontractor performing the contract shall not be marked or obliterated.

(f) The contractor shall inform the body awarding the contract of the location of the records enumerated under subdivision (a), including the street address, city and county, and shall, within five working days, provide a notice of a change of location and address.

(g) The contractor or subcontractor shall have 10 days in which to comply subsequent to receipt of a written notice requesting the records enumerated in subdivision (a). In the event that the contractor or subcontractor fails to comply within the 10-day period, he or she shall, as a penalty to the state or political subdivision on whose behalf the contract is made or awarded, forfeit twenty-five dollars ($25) for each calendar day, or portion thereof, for each worker, until strict compliance is effectuated. Upon the request of the Division of Apprenticeship Standards or the Division of Labor Standards Enforcement, these penalties shall be withheld from progress payments then due. A contractor is not subject to a penalty assessment pursuant to this section due to the failure of a subcontractor to comply with this section.

(h) The body awarding the contract shall cause to be inserted in the contract stipulations to effectuate this section.

(i) The director shall adopt rules consistent with the California Public Records Act, (Chapter 3.5 (commencing with Section 6250), Division 7, Title 1, Government Code) and the Information Practices Act of 1977, (Title 1.8 (commencing with Section 1798), Part 4, Division 3, Civil Code) governing the release of these records, including the establishment of reasonable fees to be charged for reproducing copies of records required by this section.

(j) This section shall remain in effect only until January 1, 2003, and as of that date is repealed, unless a later enacted statute, that is enacted before January 1, 2003, deletes or extends that date.

1777.5. (a) Nothing in this chapter shall prevent the employment of properly registered apprentices upon public works.

(b) Every apprentice employed upon public works shall be paid the prevailing rate of per diem wages for apprentices in the trade to which he or she is registered and shall be employed only at the work of the craft or trade to which he or she is registered.

(c) Only apprentices, as defined in Section 3077, who are in training under apprenticeship standards that have been approved by the Chief of the Division of Apprenticeship Standards and who are parties to written apprentice agreements under Chapter 4 (commencing with Section 3070) of Division 3 are eligible to be employed at the apprentice wage rate on public works. The employment and training of each apprentice shall be in accordance with either (1) the apprenticeship standards and apprentice agreements under which he or she is training or (2) the rules and regulations of the California Apprenticeship Council.

(d) When the contractor to whom the contract is awarded by the state or any political subdivision, in performing any of the work under the contract, employs workers in any apprenticeable craft or trade, the contractor shall employ apprentices in at least the ratio set forth in this section and may apply to any apprenticeship program in the craft or trade that can provide apprentices to the site of the public work for a certificate approving the contractor under the apprenticeship standards for the employment and training of apprentices in the area or industry affected. However, approval or denial of the apprenticeship program shall be subject to review by the Administrator of Apprenticeship. The apprenticeship program or programs, upon approving the contractor, shall arrange for the dispatch of apprentices to the contractor. A contractor covered by an apprenticeship program's standards shall not be required to submit any additional application in order to include additional public works contracts under that the program. "Apprenticeable craft or trade," as used in this section, means a craft or trade determined as an apprenticeable occupation in accordance with rules and regulations prescribed by the California Apprenticeship Council. As used in this section, "contractor" includes any subcontractor under a contractor who performs any public works not excluded by subdivision (o).

(e) Prior to commencing work on a contract for public works, every contractor shall submit contract award information to an applicable apprenticeship program that can supply apprentices to the site of the public work. The information submitted shall include an estimate of journeyman hours to be performed under the contract, the number of apprentices proposed to be employed, and the approximate dates the apprentices would be employed. A copy of this information shall also be submitted to the awarding body if requested by the awarding body. Within 60 days after concluding work on the contract, each contractor and subcontractor shall submit to the awarding body, if requested, and to the apprenticeship program a verified statement of the journeyman and apprentice hours performed on the contract. The information under this subdivision shall be public. The apprenticeship programs shall retain this information for 12 months.

(f) The apprenticeship program that can supply apprentices to the area of the site of the public work shall ensure equal employment and affirmative action in apprenticeship for women and minorities.

(g) The ratio of work performed by apprentices to journeymen employed in a particular craft or trade on the public work may be no higher than the ratio stipulated in the apprenticeship standards under which the apprenticeship program operates where the contractor agrees to be bound by those standards, but, except as otherwise provided in this section, in no case shall the ratio be less than one hour of apprentice work for every five hours of journeyman work.

(h) This ratio of apprentice work to journeyman work shall apply during any day or portion of a day when any journeyman is employed at the jobsite and shall be computed on the basis of the hours worked during the day by journeymen so employed. Any work performed by a journeyman in excess of eight hours per day or 40 hours per week shall not be used to calculate the ratio. The contractor shall employ apprentices for the number of hours computed as above before the end of the contract or, in the case of a subcontractor, before the end of the subcontract. However, the contractor shall endeavor, to the greatest extent possible, to employ apprentices during the same time period that the journeymen in the same craft or trade are employed at the jobsite. Where an hourly apprenticeship ratio is not feasible for a particular craft or trade, the Division of Apprenticeship Standards, upon application of an apprenticeship program, may order a minimum ratio of not less than one apprentice for each five journeymen in a craft or trade classification.

(i) A contractor covered by this section that has agreed to be covered by an apprenticeship program's standards upon the issuance of the approval certificate, or that has been previously approved for an apprenticeship program in the craft or trade, shall employ the number of apprentices or the ratio of apprentices to journeymen stipulated in the applicable apprenticeship standards, but in no event less than the 1-to-5 ratio required by subdivision (g).

(j) Upon proper showing by a contractor that he or she employs apprentices in a particular craft or trade in the state on all of his or her contracts on an annual average of not less than one hour of apprentice work for every five hours of labor performed by journeymen, the Division of Apprenticeship Standards may grant a certificate exempting the contractor from the 1-to-5 hourly ratio, as set forth in this section for that craft or trade.

(k) An apprenticeship program has the discretion to grant to a participating contractor or contractor association a certificate, which shall be subject to the approval of the Administrator of Apprenticeship, exempting the contractor from the 1-to-5 ratio set forth in this section when it finds that any one of the following conditions is met:

(1) Unemployment for the previous three-month period in the area exceeds an average of 15 percent.

(2) The number of apprentices in training in the area exceeds a ratio of 1 to 5.

(3) There is a showing that the apprenticeable craft or trade is replacing at least one-thirtieth of its journeymen annually through apprenticeship training, either on a statewide basis or on a local basis.

(4) Assignment of an apprentice to any work performed under a public works contract would create a condition that would jeopardize his or her life or the life, safety, or property of fellow employees or the public at large, or the specific task to which the apprentice is to be assigned is of a nature that training cannot be provided by a journeyman.

(l) When an exemption is granted pursuant to subdivision (k) to an organization that represents contractors in a specific trade from the 1-to-5 ratio on a local or statewide basis, the member contractors will not be required to submit individual applications for approval to local joint apprenticeship committees, if they are already covered by the local apprenticeship standards.

(m) A contractor to whom a contract is awarded, who, in performing any of the work under the contract, employs journeymen or apprentices in any apprenticeable craft or trade shall contribute to the California Apprenticeship Council the same amount that the director determines is the prevailing amount of apprenticeship training contributions in the area of the public works site. A contractor may take as a credit for payments to the council any amounts paid by the contractor to an approved apprenticeship program that can supply apprentices to the site of the public works project. The contractor may add the amount of the contributions in computing his or her bid for the contract. At the end of each fiscal year the California Apprenticeship Council shall make grants to each apprenticeship program in proportion to the number of hours of training provided by the program for which the program did not receive contributions, weighted by the regular rate of contribution for the program. These grants shall be made from funds collected by the California Apprenticeship Council during the fiscal year pursuant to this subdivision from contractors that employed registered apprentices but did not contribute to an approved apprenticeship program. All these funds received during the fiscal year shall be distributed as grants.

(n) The body awarding the contract shall cause to be inserted in the contract stipulations to effectuate this section. The stipulations shall fix the responsibility of compliance with this section for all apprenticeable occupations with the prime contractor.

(o) This section does not apply to contracts of general contractors or to contracts of specialty contractors not bidding for work through a general or prime contractor when the contracts of general contractors or those specialty contractors involve less than thirty thousand dollars ($30,000) or 20 working days.

(p) All decisions of an apprenticeship program under this section are subject to Section 3081.

1777.6. It shall be unlawful for an employer or a labor union to refuse to accept otherwise qualified employees as registered apprentices on any public works, on the ground of the race, religious creed, color, national origin, ancestry, sex, or age, except as provided in Section 3077, of such employee.

1777.7. (a) A contractor or subcontractor that knowingly violates Section 1777.5 shall forfeit as a civil penalty an amount not exceeding one hundred dollars ($100) for each full calendar day of noncompliance. The amount of this penalty shall be based on consideration whether the violation was a good faith mistake due to inadvertence. A contractor or subcontractor that knowingly commits a second or subsequent violation of Section 1777.5 within a three-year period, where the noncompliance results in apprenticeship training not being provided as required by this chapter, shall forfeit as a civil penalty the sum of not more than three hundred dollars ($300) for each full calendar day of noncompliance. Notwithstanding Section 1727, upon receipt of a determination that a civil penalty has been imposed, the awarding body shall withhold the amount of the civil penalty from contract progress payments then due or to become due.

(b) (1) In the event a contractor or subcontractor is determined by the Administrator of Apprenticeship to have knowingly violated any provision of Section 1777.5, the Administrator shall deny to the contractor or subcontractor, both individually and in the name of the business entity under which the contractor or subcontractor is doing business, the right to bid on or receive any public works contract for a period of up to one year for the first violation and for a period of up to three years for a second or subsequent violation. Each period of debarment shall run from the date the determination of noncompliance by the Administrator of Apprenticeship.

(2) An affected contractor or subcontractor may obtain a review of the debarment or civil penalty by transmitting a written request to the office of the Administrator within 30 days after service of the order of debarment or civil penalty. If the Administrator receives no request for review within 30 days after service, the order of debarment or civil penalty shall become final for the period authorized.

(3) Within 20 days of the timely receipt of a request for hearing, the Administrator shall provide the contractor or subcontractor the opportunity to review any evidence the Administrator may offer at the hearing. The Administrator shall also promptly disclose to the contractor or subcontractor any nonprivileged documents obtained after the 20-day time limit.

(4) Within 90 days of the timely receipt of the a request for hearing, a hearing shall be commenced before an impartial hearing officer designated by the Administrator and possessing the qualifications of an administrative law judge pursuant to Section 11502 of the Government Code. The contractor or subcontractor shall have the burden of showing compliance with Section 1777.5. The decision to debar shall be reviewed by a hearing officer or court only for abuse of discretion.

(5) Within 45 days of the conclusion of the hearing, the hearing officer shall issue a written decision affirming, modifying, or dismissing the debarment or civil penalty. The decision shall contain a notice of findings, findings, and an order. This decision shall be deemed the final decision of the Administrator and shall be served on all parties and the awarding body pursuant to Section 1013 of the Code of Civil Procedure by first-class mail at the last known address of the party on file with the Administrator. Within 15 days of issuance of the decision, the hearing officer may reconsider or modify the decision to correct an error, except that a clerical error may be corrected at any time.

(6) An affected contractor or subcontractor may obtain review of the final decision of the Administrator by filing a petition for a writ of mandate to the appropriate superior court pursuant to Section 1094.5 of the Code of Civil Procedure within 45 days after service of the final decision to debar or to assess a civil penalty. If no petition for a writ of mandate is filed within 45 days after service of the final decision, the order shall become final. If the petitioner claims that the findings are not supported by the evidence, abuse of discretion is established if the court determines that the findings are not supported by substantial evidence in light of the entire record.

(7) The Administrator may file a certified copy of a final order with the clerk of the superior court in any county in which the affected contractor or subcontractor has property or has or had a place of business.

(c) If a subcontractor is found to have violated Section 1777.5, the prime contractor of the project is not liable for any penalties under subdivision (a), unless the prime contractor had knowledge of the subcontractor's failure to comply with the provisions of Section 1777.5 or unless the prime contractor fails to comply with any of the following requirements:

(1) The contract executed between the contractor and the subcontractor or the performance of work on the public works project shall include a copy of the provisions of Sections 1771, 1775, 1776, 1777.5, 1813, and 1815.

(2) The contractor shall continually monitor a subcontractor's use of apprentices required to be employed on the public works project pursuant to subdivision (d) of Section 1777.5, including, but not limited to, periodic review of the certified payroll of the subcontractor.

(3) Upon becoming aware of a failure of the subcontractor to employ the required number of apprentices, the contractor shall take corrective action, including, but not limited to, retaining funds due the subcontractor for work performed on the public works project until the failure is corrected.

(4) Prior to making the final payment to the subcontractor for work performed on the public works project, the contractor shall obtain an affidavit signed under penalty of perjury from the subcontractor that the subcontractor has employed the required number of apprentices on the public works project.

(d) In lieu of the penalty provided for in subdivision (a) or (b), the director may for a first-time violation and with the concurrence of the apprenticeship program, order the contractor or subcontractor to provide apprentice employment equivalent to the work hours that would have been provided for apprentices during the period of noncompliance.

(e) Any funds withheld by the awarding body pursuant to this section shall be deposited in the General Fund if the awarding body is a state entity, or in the equivalent fund of an awarding body if the awarding body is an entity other than the state.

(f) The interpretation and enforcement of Section 1777.5 and this section shall be in accordance with the rules and procedures of the California Apprenticeship Council.

Division of Apprenticeship Standards

APPRENTICES ON PULIC WORKS

SUMMARY OF REQUIREMENTS

Compliance with California Labor Code Section 1777.5 requires all public works contractors and subcontractors to:

- Submit contract award information to the applicable joint apprenticeship committee, including an estimate of the journeyman hours to be performed under the contract, the number of apprentices to be employed, and the approximate dates the apprentices will be employed.

  The contract award information shall be in writing, and shall be provided to the applicable apprenticeship committee within 10 days of the date of the agreement or contract award, but in no event later than the first day in which the contractor has workers employed upon the public work. (California Code of Regulations, Title 8, Section 230.)

- Employ apprentices on the public work in a ratio to journeymen of no less than one hour of apprentice work for every five hours of labor performed by a journeyman.

- Contribute to the training fund in the amount identified in the Prevailing Wage Rate publication for journeymen and apprentices. Contractors who choose not to contribute to the local training trust fund must make their contributions to the California Apprenticeship Council, P.O. Box 420603, San Francisco, CA 94142. Training contributions to the Council are due and payable on the 15th of the month for work performed during the preceding month.

  Training contributions to the Council shall be paid by check and shall be accompanied by a completed CAC2 form, Training Fund Contributions, or the following information (California Code of Regulations, Title 8, Section 230.2 c):

  1. Name, address and telephone number of the contractor making the contribution.
  2. Contractor's license number.
  3. Name and address of the public agency that awarded the contract.
  4. Jobsite location, including the county where the work was performed.
  5. Contract or project number
  6. Time period covered by the enclosed contributions.
  7. Contribution rate and total hours worked by the apprenticable occupation(s).

- Pay the apprentice rate on public works projects only to those apprentices who are registered, as defined in Labor Code Section 3077:

Sec. 3077. The term "apprentice" as used in this chapter means a person at least 16 years of age who has entered into a written agreement, in this chapter called an "apprentice agreement," with an employer or program sponsor. The term of apprenticeship for each apprenticeable occupation shall be approved by the chief, and in no case shall provide for no less than 2,000 hours or reasonably continuous employment for such person for his or her participation in an approved program of training through employment and through education in related and supplemental subjects.

FORMS SUBMITTAL SCHEDULE

Document Name	Frequency	Submittal Due Date	Notes:
LCP Checklist	1-time submittal	At pre-construction/LCP meeting	Must be submitted by all listed contractors and subcontractors
Form DAS-140 Public Works Contract Award Info **Form DAS-142** (recommended)	1-time submittal	Prior to start of work	A copy MUST be forwarded to the Labor Compliance Officer *A DAS-142, the form to "request for dispatch of an apprentice" is not a required submittal; however, making a request for the dispatch of an apprentice IS REQUIRED (verbal or written). Therefore, the submittal of a DAS-142 is recommended, since it serves as proof the request was made.*
Fringe Benefit Statement	1-time submittal, then as benefit amounts change	Submitted with 1st CPR, then as contribution amount changes	In the event fringe benefits are paid "in cash" to workers, contractor should indicate "fringe benefits paid in cash" across this form. Note: Training contributions should be paid to the CAC or an approved apprenticeship training program; not to the employee.
Statement of Compliance **Certified Payroll Report (CPR)** **Statement of Non-Performance**	Weekly	Within 10 days of end of payroll period	CPRs to be submitted to the Labor Compliance Officer within 10 days from the end of the payroll period. All documents modifying the payrolls must accompany the CPR; these include but are not limited to: approved "make-up days," "special-shifts," "alternative work schedules." All inactive workweeks (after the submission of the 1st CPR) should be reported on a "Statement of Non-Performance"
Owner-Operators Statement of Compliance **Owner-Operator CPR**	Weekly	(If applicable) Within 10 days of end of payroll period	If applicable, all Owner-Operators are to be reported on these forms. Forward forms to the Labor Compliance Officer weekly.
Employer's Monthly Report to Trustees **Copy of the contribution check**	Monthly	Within 15 days of end of reporting month	Submit monthly reports sent to the trust/plan along with a copy of the contribution check, to the Labor Compliance Officer.
Form CAC-2 Training Funds Contributions **Copy of the contribution check**	Monthly	By the 15th of each month, for work performed in the preceding month	A copy MUST be forwarded to the Labor Compliance Officer Monthly submittal should reflect the accurate monthly man-hours performed during the previous month. The CAC-2 form must be accompanied with a copy of the contribution check.
List of Sub-Tier Subcontractors	1-time submittal, then as subs are brought on	Prior to start of work	Must be submitted by all contractors. Indicate which subcontractors the contractor intends to utilize in execution of its project work
Public Works Affidavit	1-time submittal	At conclusion of work	To be submitted to the Labor Compliance Officer by the prime and each sub at the conclusion of the work and prior to release of retention.

CALIFORNIA CODE OF REGULATIONS CHECKLIST

LABOR COMPLIANCE PROGRAM
The Solis Group
234 North El Molino Avenue, Suite 202
Pasadena, CA 91101
Phone: (626) 685-6989
Fax: (626) 685-6985

The Solis Group

CHECKLIST OF LABOR LAW REQUIREMENTS FOR REVIEW AT JOB START MEETINGS

(In accordance with CCR Section 16430)

The federal and state labor law requirements applicable to the contract are composed of, but not limited to, the following:

LCP Checklist	Contractor's Initials
1. Payment of Prevailing Wage Rates The award of a public works contract requires that all workers employed on the project be paid not less than the specified general prevailing wage rates by the contractor and its subcontractors per Labor Code Section 1770. The contractor is responsible for obtaining and complying with all applicable general prevailing wage rates for trades workers and any rate changes which may occur during the term of the contract. Prevailing wage rates and rate changes are to be posted at the job site for workers to view or be provided to workers upon request. By signing the LCP Checklist, all contractors and subcontractors are assenting to obtaining the correct prevailing wage rates.	
2. Apprentices It is the duty of the contractor and subcontractors to employ registered apprentices on public works projects per Labor Code Section 1777.5;	
3. Penalties Penalties, including forfeitures and debarment, shall be imposed for contractor/subcontractor failure to pay prevailing wages, failure to maintain and submit accurate certified payroll records upon request, failure to employ apprentices, and failure to pay employees for all hours worked at the correct prevailing wage rate, in accordance with Labor Code Sections 1775, 1776, 1777.7, and 1813.	
4. Certified Payroll Records Per Labor Code Section 1776, contractors and subcontractors are required to keep accurate payroll records which reflect the name, address, social security number, and work classification of each employee; the straight time and overtime hours worked each day and each week; the fringe benefits; and the actual per diem wages paid to each journeyperson, apprentice, worker, or other employee hired in connection with a public works project. In an effort to prove that per diem wages were paid to the workers on behalf of the contractor, by signing the LCP Checklist the contractor is assenting to the fact that it will provide TSG with copies of bank certified cancelled checks make to workers, the California Apprenticeship Council, and/or Union or other trust funds, as required by the California Labor Code §1776(a)(2)(b), and the California Department of Industrial Relations (Chapter 8, Subchapter 3, Article 1, §16000). Employee payroll records shall be certified and shall be made available for inspection at all reasonable hours at the principal office of the contractor/subcontractor, or shall be furnished to any employee, or to his or her authorized representative, on request. Contractors and subcontractors shall maintain their certified payrolls on a weekly basis and shall submit said payrolls to TSG in a timely manner, and at least on a monthly basis. In the event that there has been no work performed during a given week, the Certified Payroll Record shall be annotated "No Work" for that week.	

CALIFORNIA CODE OF REGULATIONS CHECKLIST

5. Nondiscrimination in Employment Prohibitions against employment discrimination are contained in Labor Code Sections 1735 and 1777.6; the Government Code; the Public Contracts Code; and Title VII of the Civil Rights Act of 1964, as amended. All contractors and subcontractors are required to implement equal employment opportunities as delineated below: a. Equal Employment Poster The equal employment poster shall be posted at the job site in a conspicuous place visible to employees and employment applicants for the duration of the project.	
6. Kickbacks Prohibited Per Labor Code Section 1778, contractors and subcontractors are prohibited from accepting, taking wages illegally, or extracting "kickbacks" from employee wages.	
7. Acceptance of Fees Prohibited Contractors and subcontractors are prohibited from exacting any type of fee for registering individuals for public work (Labor Code Section 1779), or for filling work orders on public works contracts (Labor Code Section 1780);	
8. Listing of Subcontractors Contractors are required to list all subcontractors hired to perform work on a public works project when that work is equivalent to more than one-half of one percent of the total effort (Public Contracts Code Section 4104) TSG will request sub-contractor listings for all project contractors reported on the project.	
9. Proper Licensing Contractors and subcontractors are required to be properly licensed. Penalties will be imposed for employing workers while unlicensed (Labor Code Section 1021 and Business and Professions Code Section 7000 et seq. under California Contractors License Law).	
10. Unfair Competition Prohibited Contractors and subcontractors are prohibited from engaging in unfair competition (Business and Professions Code Sections 17200-17208).	
11. Workers' Compensation Insurance All contractors and subcontractors are required to be insured against liability for workers' compensation, or to undertake self-insurance in accordance with the provisions of Labor Code §3700 (LC§1861).	
12. OSHA Contractors and subcontractors are required to comply with the Occupational, Safety and Health laws and regulations applicable to the particular public works project. In accordance with federal and state laws and contract documents, the undersigned contractor herein certifies that it will comply with the foregoing labor law requirements; and fully understands that failure to comply with these requirements will subject it to the penalties cited herein.	
13. Hiring Undocumented Workers Contractors and subcontractors are required to secure proof of eligibility/citizenship from all project workers due to the federal prohibition against hiring undocumented workers.	
14. Itemized Wage Statements to Employees Contractors and subcontractors are required to provide itemized wage statements to their employees under Labor Code Section 226.	
15. Public Works Affidavit Prior to the release of the final payment to the subcontractors, the prime Contractor shall obtain a Public Works Affidavit (signed under penalty of perjury) from the subcontractor stating that the subcontractor paid the specified prevailing wage rates to its workers (CLC §1775.5 (b) (4)).	

Certification:

I acknowledge that I have been informed and are aware of the foregoing requirements and that I am authorized to make and sign this certification.

Contractor Signature:	***Date:***
Labor Compliance Officer's Signature:	***Date:***
Awarding Body:	
Project Name or Number:	

DAS–140

PUBLIC WORKS CONTRACT AWARD INFORMATION

Contract award information must be sent to your Apprenticeship Committee if you are approved to train. If you are not approved to train, you must send the information (which may be this form) to ALL applicable Apprenticeship Committees in your craft or trade in the area of the site of the public work. Go to: http://www.dir.ca.gov/das/PublicWorksForms.htm for information about programs in your area and trade. You may also consult your local Division of Apprenticeship Standards (DAS) office whose telephone number may be found in your local directory under California, State of, Industrial Relations, Division of Apprenticeship Standards.

Do not send this form to the Division of Apprenticeship Standards.

NAME OF YOUR COMPANY	CONTRACTOR'S STATE LICENSE NO
MAILING ADDRESS- NUMBER & STREET, CITY, ZIP CODE	AREA CODE & TELEPHONE NO.
NAME & ADDRESS OF PUBLIC WORKS PROJECT	DATE YOUR CONTRACT EXECUTED
	DATE OF EXPECTED OR ACTUAL START OF PROJECT
NAME & ADDRESS OF PUBLIC AGENCY AWARDING CONTRACT	ESTIMATED NUMBER OF JOURNEYMEN HOURS
	OCCUPATION OF APPRENTICE
THIS FORM IS BEING SENT TO: (NAME & ADDRESS OF APPRENTICESHIP PROGRAM(S))	ESTIMATED NUMBER OF APPRENTICE HOURS
	APPROXIMATE DATES TO BE EMPLOYED

This is not a request for dispatch of apprentices.

Contractors must make a separate request for actual dispatch, in accordance with Section 230.1(a) California Code of Regulations

Check One Of The Boxes Below

1. ☐ We are already approved to train apprentices by the _____ Apprenticeship Committee. We will employ and train under their Standards. Enter name of the Committee

2. ☐ We will comply with the standards of _____ Apprenticeship Committee for the duration of this job only. Enter name of the Committee

3. ☐ We will employ and train apprentices in accordance with the California Apprenticeship Council regulations, including § 230.1 (c) which requires that apprentices employed on public projects can only be assigned to perform work of the craft or trade to which the apprentice is registered and that the apprentices must at all times work with or under the direct supervision of journeyman/men.

Signature _____ *Date* _____

Typed Name _____

Title _____

State of California - Department of Industrial Relations DIVISION OF APPRENTICESHIP STANDARDS

DAS 140 (REV. 1/04)

DAS-142

REQUEST FOR DISPATCH OF AN APPRENTICE

Do not send this form to DAS

You may use this form to request dispatch of an apprentice from the Apprenticeship Committee in the craft or trade in the area of the public work. Go to: http://www.dir.ca.gov/das/PublicWorksForms.htm for information about programs in your area and trade. You may also consult your local Division of Apprenticeship Standards (DAS) office whose telephone number may be found in your local directory under California, State of, Industrial Relations, Division of Apprenticeship Standards.

Date: _____

To Applicable Apprenticeship Committee: _____

Address: _____

Telephone: _____ Fax: _____

Contractor Requesting Dispatch: _____

Address: _____

Telephone: _____ Fax: _____

Person making request: _____

Number of Apprentice(s) Needed _____ Craft or Trade_____

Date Apprentice(s) to Report: _____ (48 hours notice required)

Name of Person to Report to: _____

Address to Report to: _____

Time to Report: _____

You may use this form, or make a verbal or written request, to ask for the dispatch of an apprentice. Please take note of California Code of Regulations, *Title 8, § 230.1 (a)* which says in part: *if in response to a written request an Apprenticeship Committee does not dispatch any apprentice to a contractor who has agreed to employ and train apprentices in accordance with either the Apprenticeship Committee's Standards or these regulations within 72 hours of such request (excluding Saturdays, Sundays and holidays) the contractor shall not be considered in violation of this section as a result of failure to employ apprentices ...*

DAS142 (Rev. 9-03)

CAC–2

State of California
Department of Industrial Relations
California Apprenticehip Council
P. O. Box 420603
San Francisco, CA 94142

TRAINING FUND CONTRIBUTIONS

Please use a separate ***form*** for each jobsite, listing the occupations for the jobsite. One ***check*** payable to the California Apprenticeship Council, may be submitted for all jobsites and/or occupations. Training fund contributions are ***not accepted*** by the California Apprenticeship Council for federal public works projects, or for non-apprenticeable occupations such as utility technicians, teamsters, etc.

California Apprenticeship Council

NAME AND ADDRESS OF CONTRACTOR/SUBCONTRACTOR MAKING CONTRIBUTION	CONTRACTOR'S LICENSE NMBER
	CONTRACT OR PROJECT NUMBER
	JOBSITE LOCATION (INCLUDE COUNTY) IF APPLICABLE. GIVE NAME OF SCHOOL, HOSPITAL, BUILDING. ETC.
NAME AND ADDRESS OF PUBLIC AGENCY AWARDING CONTRACT	PERIOD COVERED BY CONTRIBUTION (FROM-TO)

CLASSIFICATIONS) OF WORKERS (CARPENTER, PLUMBER, ELECTRICIAN, ETC.)	COUNTY WORK PERFORMED IN	HOURS	CONTRIBUTION RATE PER HOUR	AMOUNT
				0.00
				0.00
				0.00
				0.00
				0.00
				0.00
				0.00
			Total	$0.00

SIGNATURE PLEASE TYPE OR PRINT YOUR NAME	DATE
TITLE	AREA CODE & TELEPHONE NUMBER

STATEMENT OF COMPLIANCE

Date:_____

I, _____do hereby certify under penalty of perjury:
(Name of signatory party) (Title)

(1) That all the information in this report is true and correct;

(2) That I pay or supervise the payment of the persons reported as employed by _____
(Contractor or Subcontractor)

on the _____, that during the payroll period commencing on the ___ day of _____
(Project)

20 __and ending the ___ day of _____, 20 __, all persons employed on said project have been paid their full weekly wages earned, that no rebates have or will be made either directly or indirectly to or on behalf of said _____ from the full weekly wages earned by any person and that no deductions
(Contractor or Subcontractor)

have been made either directly or indirectly from the full wages earned by any person, other than permissible deductions, as described below.

(3) That any payrolls otherwise under this contract required to be submitted for the above period are correct and complete; that the wage rates for laborers or mechanics contained therein are not less than the applicable wages contained in any wage determination incorporated into the contract; that the classifications set forth therein for each laborer or mechanic conform with the work he or she performed.

(4) That any apprentices employed in the above period are duly registered in a bona fide apprenticeship program registered with a State apprenticeship agency.

(5) That :

(a) WHERE FRINGE BENEFITS ARE PAID TO APPROVED PLANS, FUNDS OR PROGRAMS

_____ In addition to the basic hourly wage rates paid to each laborer or mechanic listed in the above referenced payroll, payment of fringe benefits as listed in the contract have been or will be made to appropriate programs for the benefit of such employees, except as noted in Section 5 (c) below.

(b) WHERE FRINGE BENEFITS ARE PAID IN CASH

_____ Each laborer or mechanic listed in the above referenced payroll has been paid as indicated on the payroll, an amount not less than the sum of the applicable basic hourly wage rate plus the amount of the required fringe benefits as listed in the contract except as noted in Section 5 (c) below:

(c) EXCEPTIONS

EXCEPTION (CRAFT)	EXPLANATION
Remarks:	
Name and Title	Signature

On federally-funded projects, permissible deductions are defined in Regulations, Part 3 (29 CFR Subtitle A), issued by the Secretary of Labor under the Copeland Act, as amended (48 Stat. 948; 63 Stat. 108; 72 Stat. 967; 76 Stat. 357; 40 U.S. C. 276c). The willful falsification of any of the above statements may subject the contractor or subcontractor to civil or criminal prosecution (Section 1001 of Title 118 and Section 231 of Title 31 of the United States Code.)

CERTIFIED PAYROLL

NAME OF CONTRACTOR/SUBCONTRACTOR	ADDRESS	CONTRACTOR'S LICENSE #/SPECIALTY LICENSE # WORKERS COMPENSATION POLICY #
PAYROLL NO.	FOR WEEK ENDING	PROJECT NAME AND LOCATION / CONTRACT NO.

(1) NAME, ADDRESS, AND SOCIAL SECURITY NUMBER OF EMPLOYEE	(2) No. of W/H Exmptn	(3) Work Classification	OT or ST	(4) Day and Date / Hours Worked Daily							(5) Hour Total	(6) Pay Rate	(7) Gross Amount Earned									(9) Net Wages Paid for Week	Check No.
													This Project	All Projects	Federal Tax	FICA	State Tax	SDI	VAC	Other	Total Deduction		
			OT																				
			ST																				
			OT																				
			ST																				
			OT																				
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STATEMENT OF NON-PERFORMANCE

Payroll Number: _____

I, _____, _____ do hereby state that
(Name of Signatory party) (Title)

NO PERSONS employed by _____ performed work on
(Name of submitting company)

the construction project known as _____

for the payroll period commencing on the _____ day of _____, 200__
(1st date of week) (month)

and ending on the _____ day of _____, 200__.
(last date of week) (month)

_____
Signature of Authorized Person

_____
Date

*THIS STATEMENT IS NOT REQUIRED TO BE SUBMITTED UNTIL AFTER SUBMISSION OF THE INITIAL PAYROLL REPORT

FRINGE BENEFIT STATEMENT

Contract No.:	Project Name:	Date:

INSTRUCTIONS: This form is to be submitted with the first certified payroll. In order that the Fringe Benefit rates can be used for checking payrolls or applied to Force Account work which may be done on the above contract the hourly rates for fringe bene

Classification:	Effective Date:	Subsistence or Travel Pay:
Health and Welfare $	Trust Fund Paid To: (Name)	
	Address:	
Pension $	Trust Fund Paid To: (Name)	
	Address:	
Vacation/Holiday $	Trust Fund Paid To: (Name)	
	Address:	
Training and/or Other $	Trust Fund Paid To: (Name)	
	Address:	
Classification:	Effective Date:	Subsistence or Travel Pay:
Health and Welfare $	Trust Fund Paid To: (Name)	
	Address:	
Pension $	Trust Fund Paid To: (Name)	
	Address:	
Vacation/Holiday $	Trust Fund Paid To: (Name)	
	Address:	
Training and/or Other $	Trust Fund Paid To: (Name)	
	Address:	
Classification:	Effective Date:	Subsistence or Travel Pay:
Health and Welfare $	Trust Fund Paid To: (Name)	
	Address:	
Pension $	Trust Fund Paid To: (Name)	
	Address:	
Vacation/Holiday $	Trust Fund Paid To: (Name)	
	Address:	
Training and/or Other $	Trust Fund Paid To: (Name)	
	Address:	

Supplemental statement must be submitted during the progress of work should a change in rate of any of the classifications be made. I CERTIFY THAT THE FRINGE BENEFIT PAYMENTS ARE MADE TO THE APPROVED PLANS, FUNDS OR PROGRAMS AS LISTED ABOVE.

Submitted (Contractor/Subcontractor)	By (Name and Title)	Signature

EMPLOYER'S MONTHLY REPORT TO TRUSTEES

1 THIS REPORT IS TO COVER HOURS FOR THE MONTH OF:	ACCOUNT NO.	I do hereby certify under penalty of perjury that the employees listed below constitute all the employees that I am required to make payments to the Trust. Furthermore, I certify that the hours shown for each employee are the total hours which he/she worked or was paid.
LICENSE NO.		
EMPLOYER NAME AND ADDRESS If the above information is incorrect, please indicate changes which should be made.		Signed by: ____________ Title: ____________

2 COMPUTATION OF CONTRIBUTIONS

6 TOTAL HOURS ON ALL PAGES	A. VACATION/DUES	B. PENSION	C. TRAINING AND RETRAINING	D. HEALTH AND WELFARE	E. INDUSTRY	
	________PER HOUR $	________PER HOUR $	________PER HOUR $	________PER HOUR $	________PER HOUR $	MAKE ONE CHECK FOR TOTAL AMOUNTS OF COLUMNS A, B, C, D & E
LIQUIDATED DAMAGES						
TOTAL DUE EACH TRUST	$	$	$	$	$	

3 EMPLOYEE'S NAME	INITIALS 1ST 2ND	4 SOCIAL SECURITY NO.	5 HOURS

IMPORTANT: SOCIAL SECURITY NUMBER MUST BE FILLED TO ASSURE PROPER CREDIT.

TOTAL HOURS THIS PAGE

7 IMPORTANT THIS REPORT MUST BE FILED EVEN THOUGH NO EMPLOYEES WORKED THIS MONTH.

☐ NO EMPLOYEES WORKED THIS MONTH. PLEASE CONTINUE MAILING REPORT FORMS.

☐ TRANSFER TO INACTIVE STATUS. WE HAD NO EMPLOYEE TO REPORT THIS MONTH AND DO NOT ANTICIPATE HIRING ANY IN THE NEAR FUTURE.

IMPORTANT

REPORTS ARE DUE THE 15TH OF THE FOLLOWING MONTH AND MUST BE RECEIVED AT THE BANK BY THE 20TH TO AVOID LIGUIDATED DAMAGES (EVEN IF THERE WERE NO EMPLOYEES) AND INTEREST. LIQUIDATED DAMAGES ARE CALCULATED AT 20% OF THE TRUST OR $25 PER TRUST, WHICHEVER IS GREATER. INTEREST IS DUE AT THE MAXIMUM RATES PERMITTED BY LAW AND SPECIFIED IN THE TRUST AGREEMENT OF EACH TRUST. THESE RATES VARY AND MAY APPROACH OR EXCEED 20% PER ANNUM.

MONTHLY EMPLOYMENT UTILIZATION REPORT

U.S. DEPARTMENT OF LABOR EMPLOYMENT STANDARDS ADMINISTRATION TSG/ACTA008

This report is required by Executive Order 11246, Sec. 203. Faliure to report can result in contracts being canceled, terminated or suspended in whole or in part and the contractor may be declared ineligible for further government contracts for federally	1. Covered Area (SMSA OR EA) **LOS ANGELES COUNTY** 2. Employer I. D. No.	3. Current Goals Minority: Female:	4. Reporting Period From:_____ To:_____

Project Name and Address	Contract Number(s)	Name and Address of Reporting Contractor	Federal Funding Agency

5. Construction Trade	Classification	6. TOTAL ALL HOURS OF EMPLOYMENT (Federal and Non-Federal)										7. Minority Percentage	8. Female Percentage	9. Total Number of Employees		10. Total Number of Minority Employees	
		6a. Total Hours of All Employees by Trade		6b. Black (not of Hispanic Origin)		6c. Hispanic		6d. Asian or Pacific Islander		6e. American Indian or Alaskan Native							
		M	F	M	F	M	F	M	F	M	F			M	F	M	F
	Journeyworkers																
	Apprentices																
	Trainees																
	Subtotal																
	Journeyworkers																
	Apprentices																
	Trainees																
	Subtotal																
	Journeyworkers																
	Apprentices																
	Trainees																
	Subtotal																
	Journeyworkers																
	Apprentices																
	Trainees																
	Subtotal																
Total Journeyworkers (All Trades) Total Apprentices (All Trades) Total Trainees (All Trades) Grand Total (All Trades/All Classifications)																	

11. COMPANY OFFICIAL'S SIGNATURE AND TITLE	12. TELEPHONE ( )	13. DATE SIGNED	PAGE _____ OF _____

PUBLIC WORKS AFFIDAVIT

California Labor Code §1775(b)(4)

The undersigned Contractor/Subcontractor has complied with the provisions of the California Labor Code §1771, §1773.1, §1775, §1776, §1777.5, §1813 and §1815, and has paid all employees the specified general prevailing rate of per diem wages to its employees, and any amounts due pursuant to §1813, on the public project:

Awarding Body/District: _____

Project: _____

I declare under penalty of perjury under the laws of the State of California that the foregoing is true and correct.

Executed this _____ day of _____, 20_____ at _____, California.

_____
(Company Name)

_____
(Name)

_____
(Title)

_____
(Signature)

_____
(Approved by LCO/Program Manager)

[No final payment or retention shall be paid to the contractor until it certifies that all benefit payments owed by the contractor are made or otherwise guaranteed.]

THE SOLIS GROUP	CONFIDENTIAL EMPLOYEE INTERVIEW
NAME OF PERSON BEING INTERVIEWED:	DISTRICT:
ADDRESS:	PROJECT NAME:
CITY, STATE, ZIP:	PROJECT LOCATION:
PHONE #:	PRIME CONTRACTOR:
SOCIAL SECURITY #:	SUBCONTRACTOR:

1. Who is your present Employer?	2. How long have your worked for this Employer?
3. How long have you worked on this project?	a) What is your hourly wage rate? $
b) How are you paid? ☐ CASH ☐ CHECK ☐ COMBINATION CASH AND CHECK	
c) How often are you paid? ☐ WEEKLY ☐ BI-WEEKLY ☐ OTHER (please list)	
d) Are you a Union Member? ☐ YES ☐ NO LOCAL NUMBER:	
e) If you are a Union Member: ☐ JOURNEYMAN ☐ APPRENTICE LEVEL OF APPRENTICESHIP:	
f) What is your classification? CARPENTER CEMENT MASON DRY WALL ELECTRICIAN IRON WORKER LABORER OP.ENG. PLUMBER ROOFER SHEET METAL WORKER OTHER (please list)	
g) Do you change crafts during the day? If yes, what?	
h) What kind of Fringe Benefits ($) do you receive? ☐ FRINGE BENEFITS PAID INTO A TRUST FUND?	
i) Work being performed at the time of interview:	
j) Type of work you have been doing this past week: ☐ SAME AS WORK BEING PERFORMED TODAY	
k) List tools and equipment that you use on the job. ☐ BASIC HAND TOOLS FOR IDENTIFIED TRADE	
l) Did you come here directly from home or company yard/office? ☐ HOME ☐ YARD/OFFICE If YARD/ OFFICE, time left:	
m) Are you operating a company vehicle? ☐ YES ☐ NO If yes, do you take this vehicle to another job site? ☐ YES ☐ NO	
4. How many hours do you work on this job per day (excluding lunch and breaks)?	
A) During this project, have you also worked on an other project? ☐ YES ☐ NO If YES, which:	
a) What time did you enter and leave the job site yesterday? ☐ ENTER ☐ LEAVE	
b) Have you worked overtime on this project? ☐ MON-FRI ☐ WEEKENDS ☐ HOLIDAYS ☐ NEVER	
c) Are you paid time and 1/2, or double-time, for overtime? ☐ YES ☐ NO ☐ UNKNOWN	
d) How are you paid for your overtime? ☐ CHECK ☐ CASH ☐ OTHER	
5. How do you keep record of hours worked? ☐ SIGN-IN SHEET ☐ TIMECARD ☐ TIMECLOCK ☐ OTHER ☐ FOREMAN RECORDS & SUBMITS	
6. Additional questions for Owner Operators	a) Equipment description:
b) Do you own the equipment? YES ☐ NO ☐	c) Legal Owner:
d) Hourly Rate $ _____ (Fully operated and maintained)	e) Base Equipment Rate: $
f) Truck (Cal-T) Number:	

Employee Comments or Questions:

Interview Notes:

☐ Photo taken of Company Truck

☐ Only Recorded Worker Head Count - Work could not be impeded due to: _____

Employee Signature: X _____

FOR THE SOLIS GROUP USE ONLY:

I, _____ (INTERVIEWER) interviewed the above person on _____ / _____ DATE TIME

Revised 2/16/07

SITE VISITATION EMPLOYEE INTERVIEW TRACKING LOG

SAMPLE SITE VISITATION LOG

SITE	VISIT DATE	PRIME CONTRACTOR	SUB CONTRACTOR	EMPLOYEE NAME	SOCIAL SECURITY #	POSITION TITLE	TASK PERFORMED AT INTERVIEW	PAY RATE	COMPLIANT/ NON-COMPLIANT	CROSS-CHECK WITH PAYROLL DOCS / COMMENTS
Hoover	9/1/99	Baker	Mills Plumbing	John Doe	111-11-1111	Plumber	Repairing Plumbing	$34.19	Compliant	Certified Payroll Records check out
Hoover	9/1/00	Baker	Mills Plumbing	Mark Baker	222-22-2222	Laborer	Painting	$10.40	Non-Compliant	Certified Payroll does not check out with interview

SITE VISITATION EMPLOYEE INTERVIEW TRACKING LOG

APPENDIX 3-C

Design Plans

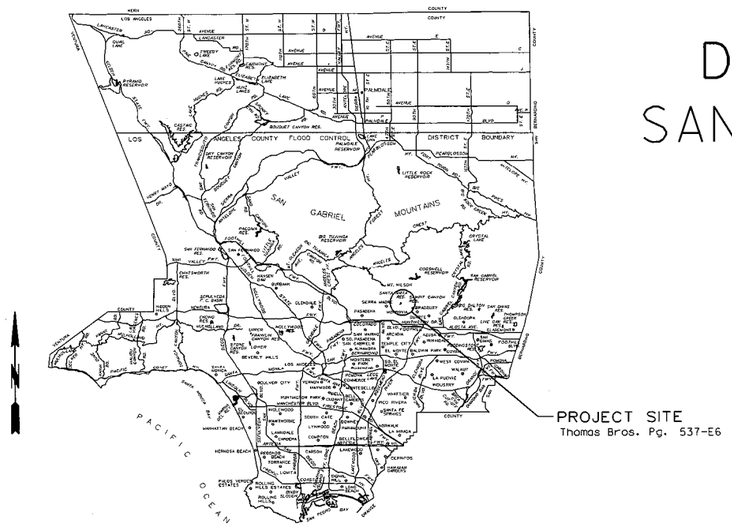
100% Santa Anita Dam Seismic Mitigation / Outlet Tower

100% Spreading Grounds Phase I

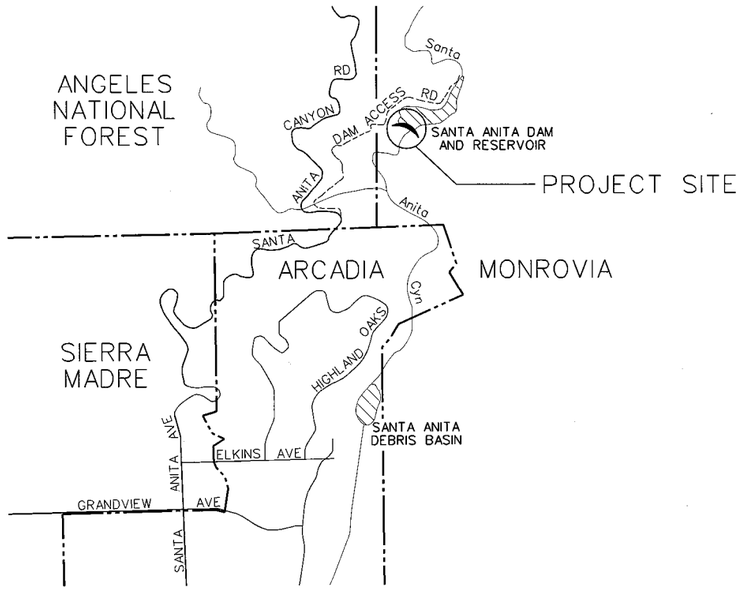
60% Spreading Grounds Phase II

60% Headworks Rehabilitation

COUNTY OF LOS ANGELES
DEPARTMENT OF PUBLIC WORKS
SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION



LOCATION MAP



VICINITY MAP
NOT TO SCALE

INDEX TO PROJECT DRAWINGS

SH. NO.	DESCRIPTION
1	TITLE SHEET
2	GENERAL PLAN AND ELEVATION
3	EXISTING SLUICEWAY GATE REMOVAL DETAILS
4	SLUICEWAY RISER ELEVATIONS
5	UPPER RISER CONCRETE DETAILS
6	LOWER RISER CONCRETE DETAILS
7	RISER BASE CONCRETE DETAILS
8	SLUICEWAY RISER TRASHRACK DETAILS
9	SLUICEWAY RISER TRACKRACK COVER DETAILS
10	EXISTING SLUICEWAY TRASHRACK RELOCATION DETAILS #1
11	EXISTING SLUICEWAY TRASHRACK RELOCATION DETAILS #2
12	SLUICEWAY RISER LADDER DETAILS #1
13	SLUICEWAY RISER LADDER DETAILS #2
14	RISER #2 ELEVATIONS AND SECTIONS
15	RISER #2 CONCRETE DETAILS
16	2'x 2' SLIDE GATE TRASHRACK DETAILS
17	72"x72" SLUICEWAY SLIDE GATE FRONT ELEVATION AND SIDE SECTION VIEW
18	24"x24" SLIDE GATE FRONT ELEVATION AND SIDE SECTION VIEW
19	UPSTREAM ELEVATION AND SECTION HYDRAULIC LINE LAYOUT
20	EXISTING CONTROL HOUSE AND NEW HYDRAULIC POWER UNIT
21	HYDRAULIC LINE - PIPE HANGER DETAIL

DRAFTER S. GREGOWSKE | DESIGNER S. GREGOWSKE | CHECKER KENNETH HO | CADD PROJECT FILE NAME WRDD000016-STRUCTURES.DGN

TWO DAYS BEFORE YOU DIG
CALL USA TOLL FREE
1-800-227-2600

APPROVED GAIL FARBER DIRECTOR OF PUBLIC WORKS
BY [signature] DEPUTY DIRECTOR 12-14-2009 DATE

RECOMMENDED
BY [signature] ASSISTANT DEPUTY DIRECTOR 12/10/09 DATE

SUBMITTED
BY [signature] 12-10-2009 DATE

DATE	MK	DESCRIPTION

REVISIONS

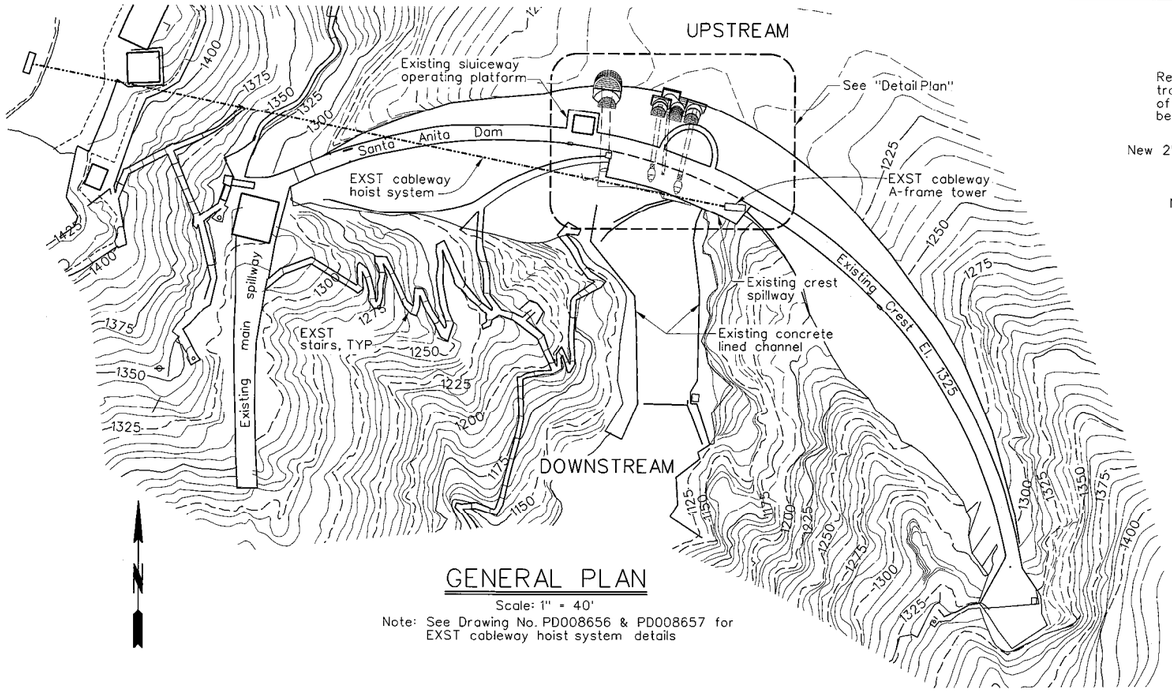
REGISTERED PROFESSIONAL ENGINEER SCOTT D. GREGOWSKE NO. 68087 EXP. 06/30/11 CIVIL STATE OF CALIFORNIA
[signature] PROJECT ENGINEER 12/10/09 DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS
SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

TITLE SHEET

PIN WRDD000016 | JOB X5009661 | DWG 55-D92.1 | SHEET 1 OF 21

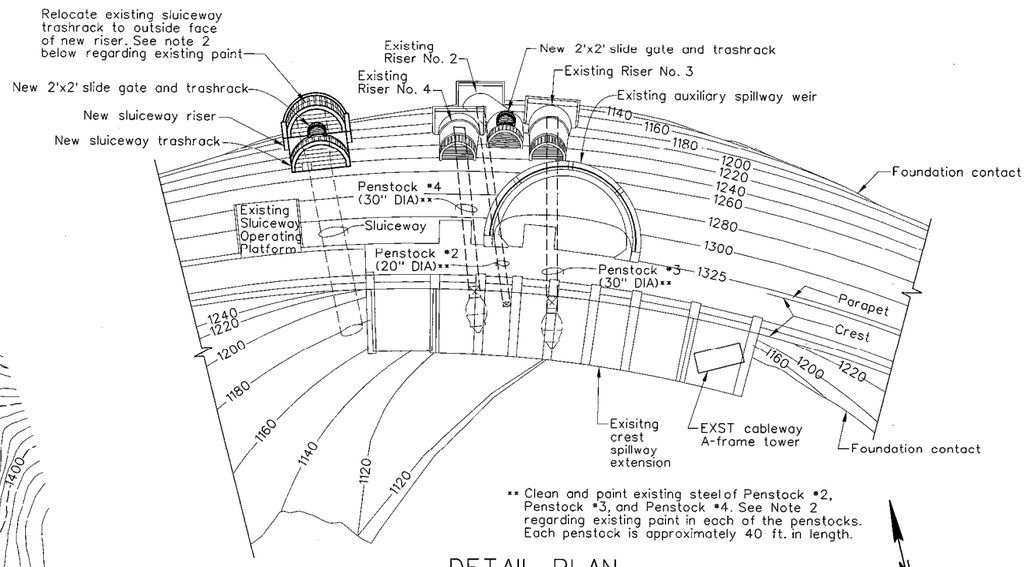
DATE: $DATE$ TIME: $TIME$ FILE: $FILEL$



GENERAL PLAN

Scale: 1" = 40'

Note: See Drawing No. PD008656 & PD008657 for EXST cableway hoist system details

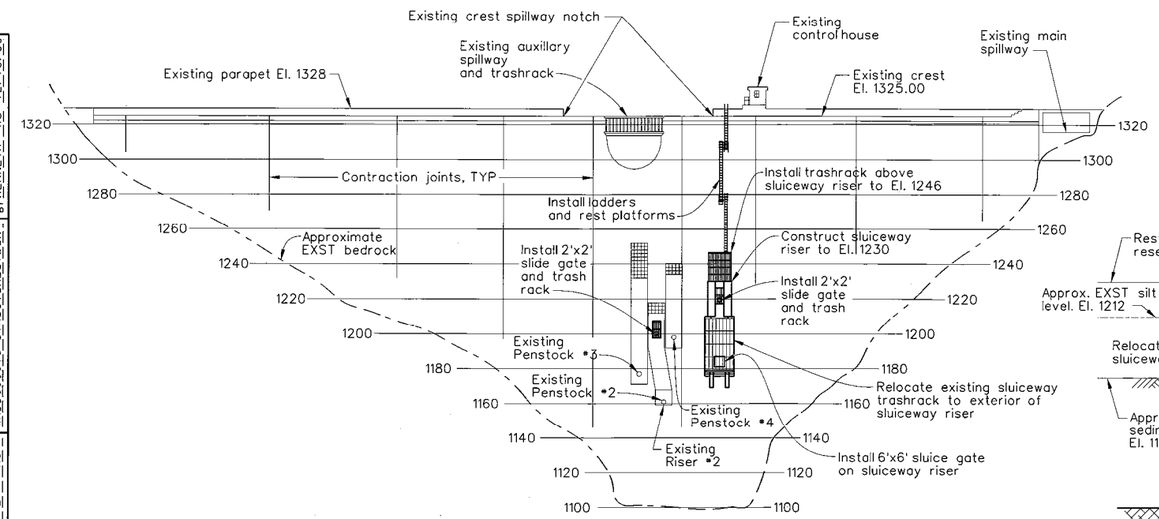


** Clean and paint existing steel of Penstock #2, Penstock #3, and Penstock #4. See Note 2 regarding existing paint in each of the penstocks. Each penstock is approximately 40 ft. in length.

DETAIL PLAN

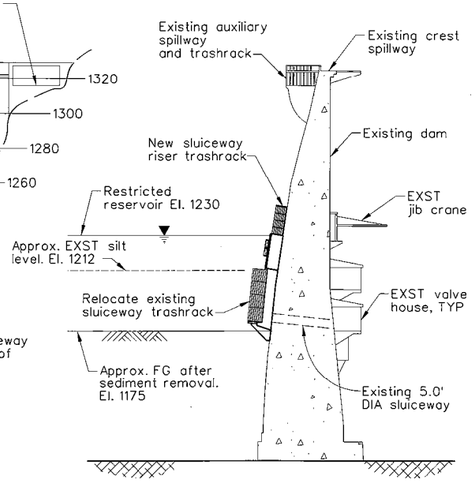
Not to Scale

Notes:
1. For clarity, ladder and rest platforms to sluiceway trashrack are not shown.
2. Existing paint system may contain lead based paint. Contractor shall refer to the Special Provision for the requirements regarding lead based paint.



UPSTREAM ELEVATION

Scale: 1" = 40'



SLUICEWAY SECTION

Scale: 1" = 40'

GENERAL NOTES

DESIGN: American Concrete Institute: Building Code Requirements For Structural Concrete (ACI 318-02) and Commentaries (ACI 318R-02)

American Institute of Steel Construction, Inc.: Manual of Steel Construction Load and Resistance Factor Design (LRFD) Third Edition

US Army Corps of Engineers: EM 1110-2-2104 Strength Design for Reinforced -Concrete Hydraulic Structures

US Dept. of Interior Bureau of Reclamation: Design of Small Dams, Third Edition

DESIGN STRESSES:

Concrete:		f'c = 4,500 psi
Reinforcing Steel (A706):		fy = 60,000 psi
Structural Steel (A36):		fy = 36,000 psi
Welding Structural Steel (AWS D1.1):		Electrodes - Fu = 70 ksi MIN

DESIGN LOAD:

Hydraulic Fluid Pressure:	62.4 pcf
Silt Fluid Pressure:	85.0 pcf

CONSTRUCTION: Standard Specifications for Public Works Construction 2006 Edition (Greenbook). Los Angeles County Department of Public Works Additions and Amendments to the Standard Specifications for Public Works Construction, 2006 Edition (Graybook).

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

DATE	MK	DESCRIPTION
		REVISIONS

REGISTERED PROFESSIONAL ENGINEER — SCOTT D. GREGOWSKE — NO. 68087 — EXP. 06/30/11 — CIVIL — STATE OF CALIFORNIA

PROJECT ENGINEER — DATE 12/10/09

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

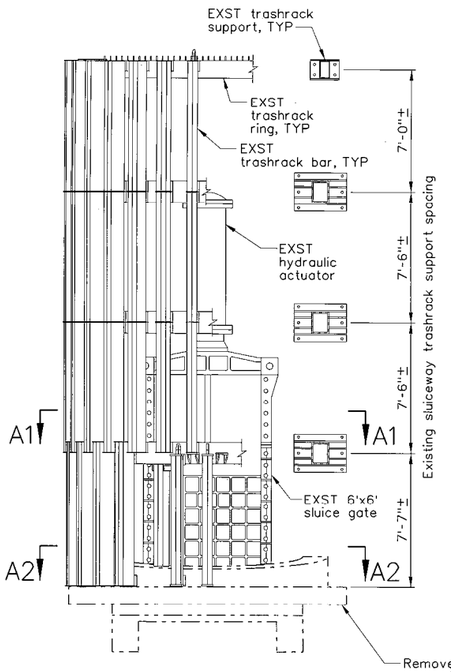
SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

GENERAL PLAN AND ELEVATION

PIN WRDD000016 | JOB X5009661 | DWG 55-D92.2 | SHEET 2 OF 21

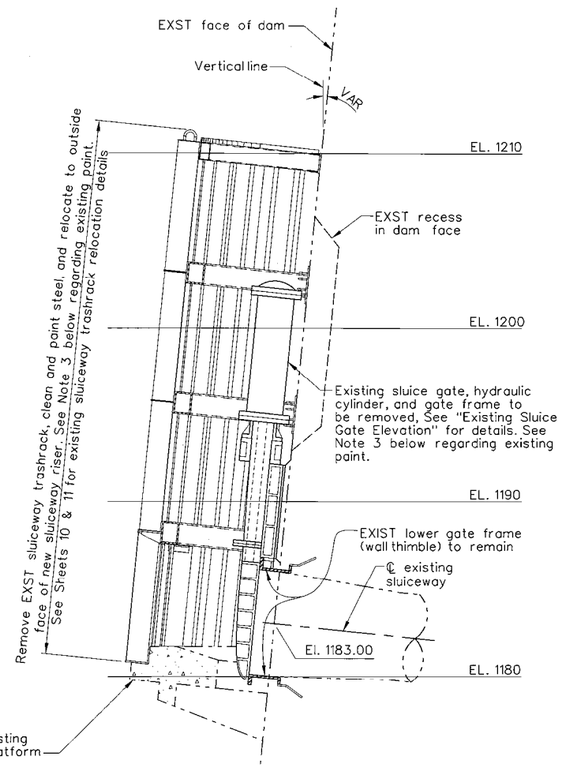
DRAFTER S. GREGOWSKE | DESIGNER S. GREGOWSKE | CHECKER KENNETH HO | CADD PROJECT FILE NAME WRDD000016-STRUCTURES.DGN | REVIEWED By KENNETH HO DATE 12/10/09

DATE: $DATE$ TIME: $TIME$ FILE: $FILE$



EXISTING FRONT ELEVATION

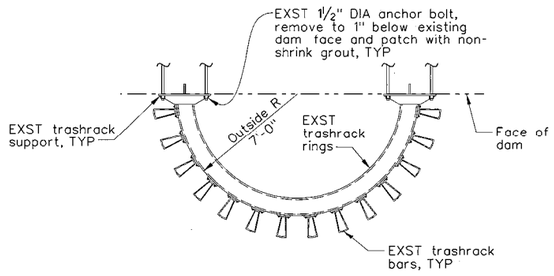
Scale: 1/4" = 1'-0"



EXISTING SIDE ELEVATION

Scale: 1/4" = 1'-0"

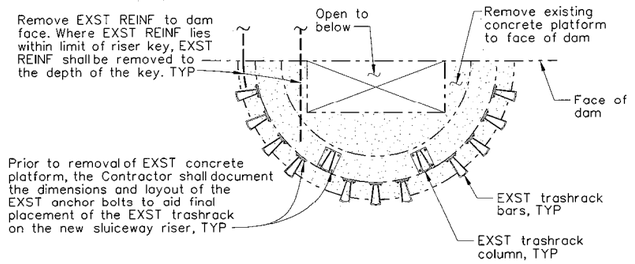
Notes:
1. For clarity, the complete trashrack is not shown.
2. See Drawing Nos. PD008550 to PD008553 for EXST sluiceway trashrack details.
3. Existing paint system may contain lead based paint. Contractor shall refer to the Special Provisions for requirements regarding lead based paint.



SECTION A1-A1

Scale: 1/4" = 1'-0"

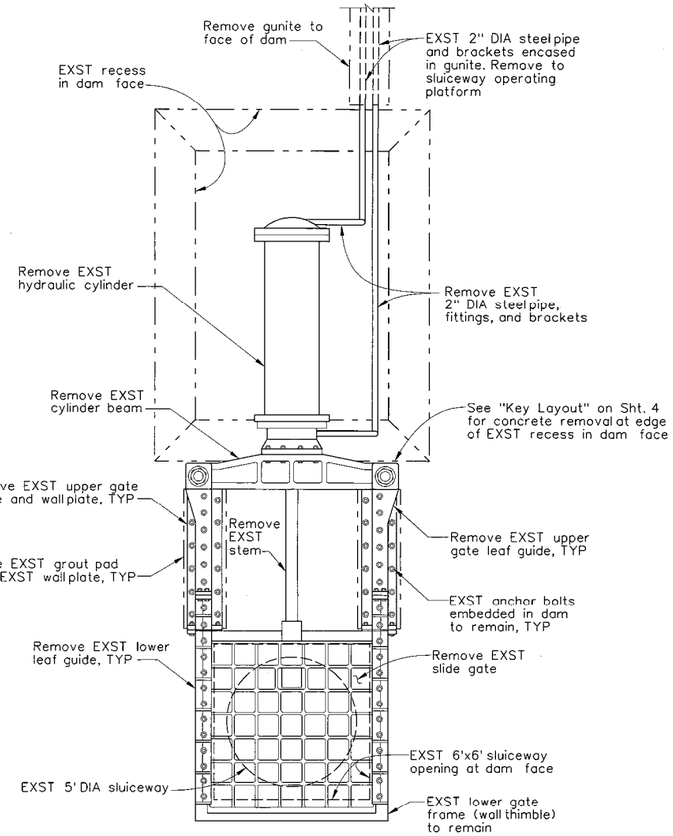
Note:
1. For clarity, the sluice gate not shown.
2. The Engineer shall inspect the condition of the existing sluiceway trashrack and determine if repair/replacement of any steel trashrack elements is required.



SECTION A2-A2

Scale: 1/4" = 1'-0"

Note:
1. For clarity, the sluice gate and platform reinforcement is not shown.
2. See Drawing No. PD008553 for EXST concrete platform details.
3. Concrete removal for riser key not shown. See "Key Layout" on Sheet 4 for additional concrete removal details.
4. [ ] Concrete Removal Limit



EXISTING SLUICE GATE ELEVATION

Scale: 3/8" = 1'-0"

Notes:
1. See Drawing Nos. PD008599 to PD008605 for existing sluice gate details
2. See Drawing Nos. PD008614 and PD008615 for existing steel pipe details
3. The paint system for the existing sluiceway gate, gate frame, and hydraulic cylinder may contain lead based paint. Contractor shall refer to the Special Provisions for requirements regarding lead based paint.

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

DATE	MK	DESCRIPTION

REVISIONS

REGISTERED PROFESSIONAL ENGINEER — SCOTT D. GREGOWSKE — NO. 68087 — EXP. 06/30/11 — CIVIL — STATE OF CALIFORNIA

PROJECT ENGINEER 12/10/09 DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

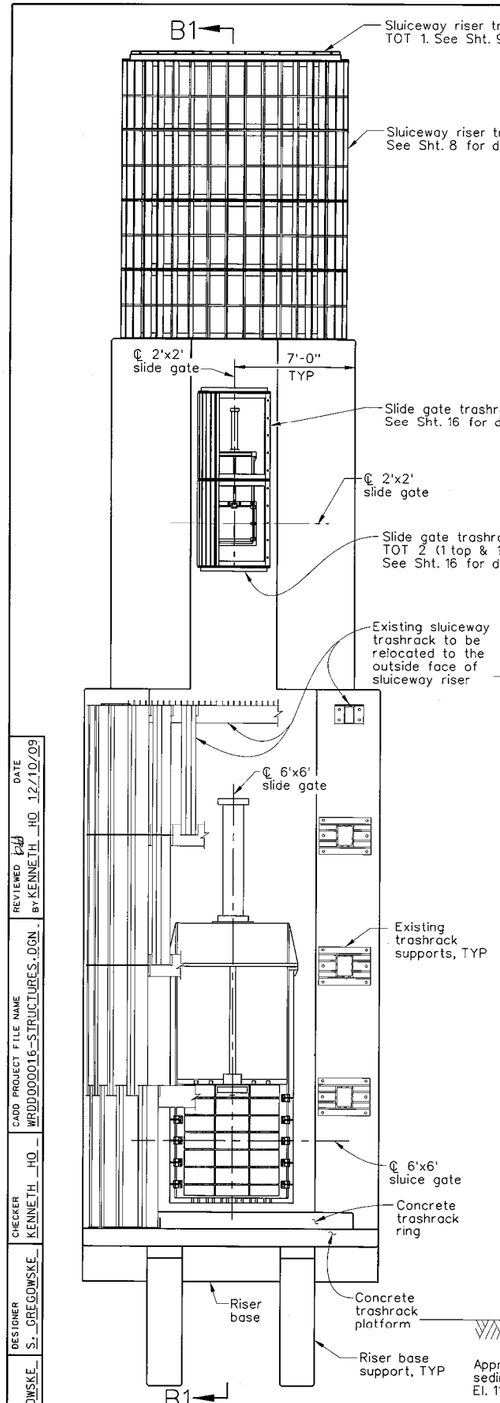
SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

EXISTING SLUICEWAY GATE REMOVAL DETAILS

PIN WRDD000016	JOB X5009661	DWG 55-D92.3	SHEET 3 OF 21

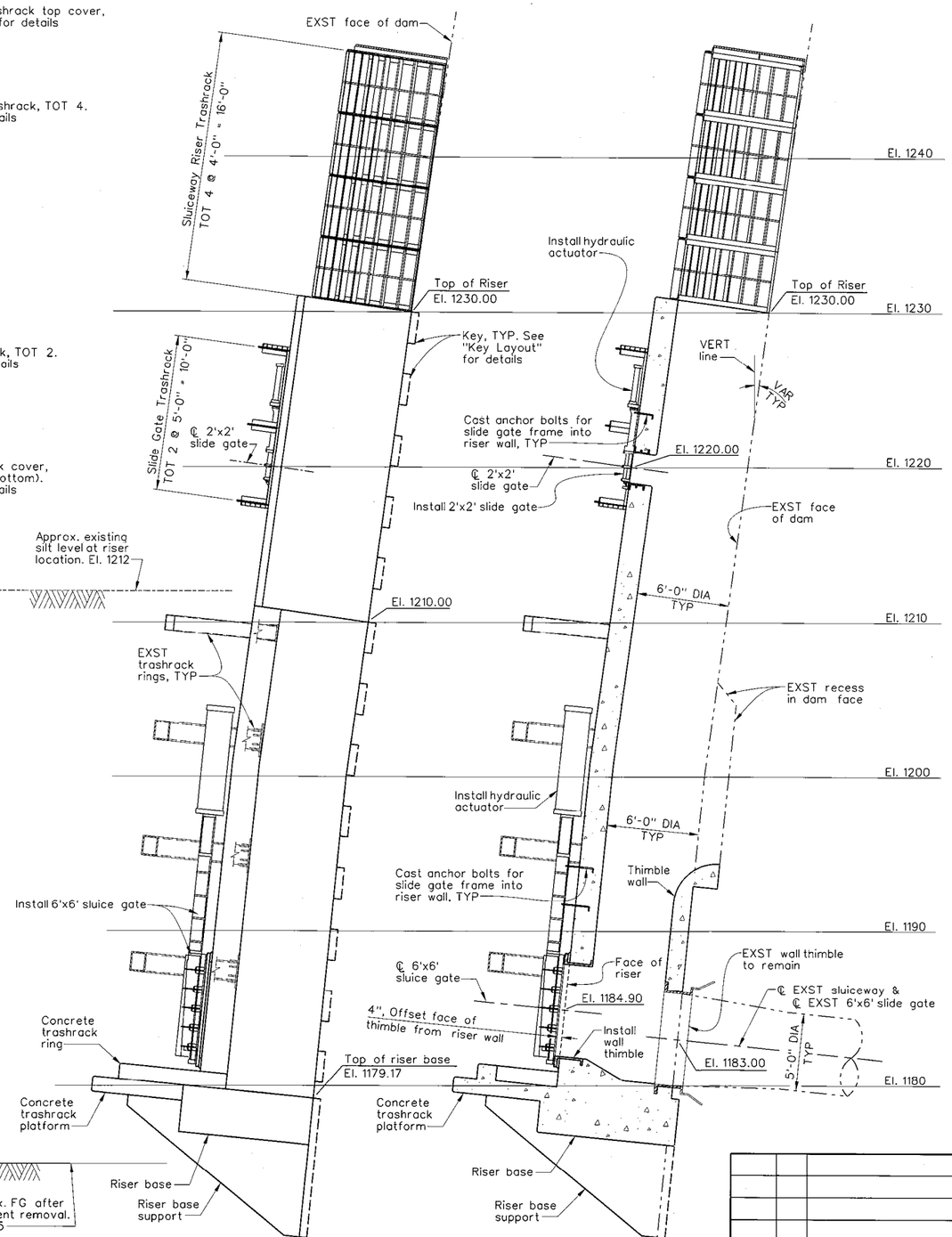
DRAFTER S. GREGOWSKE
DESIGNER S. GREGOWSKE
CHECKER KENNETH HO
CADD PROJECT FILE NAME WRDD000016-STRUCTURES.DGN
REVIEWED by KENNETH HO DATE 12/10/09

DATE: $DATE$ TIME: $TIME$ FILE: $FILE$



FRONT ELEVATION

Scale: 1/4" = 1'-0"

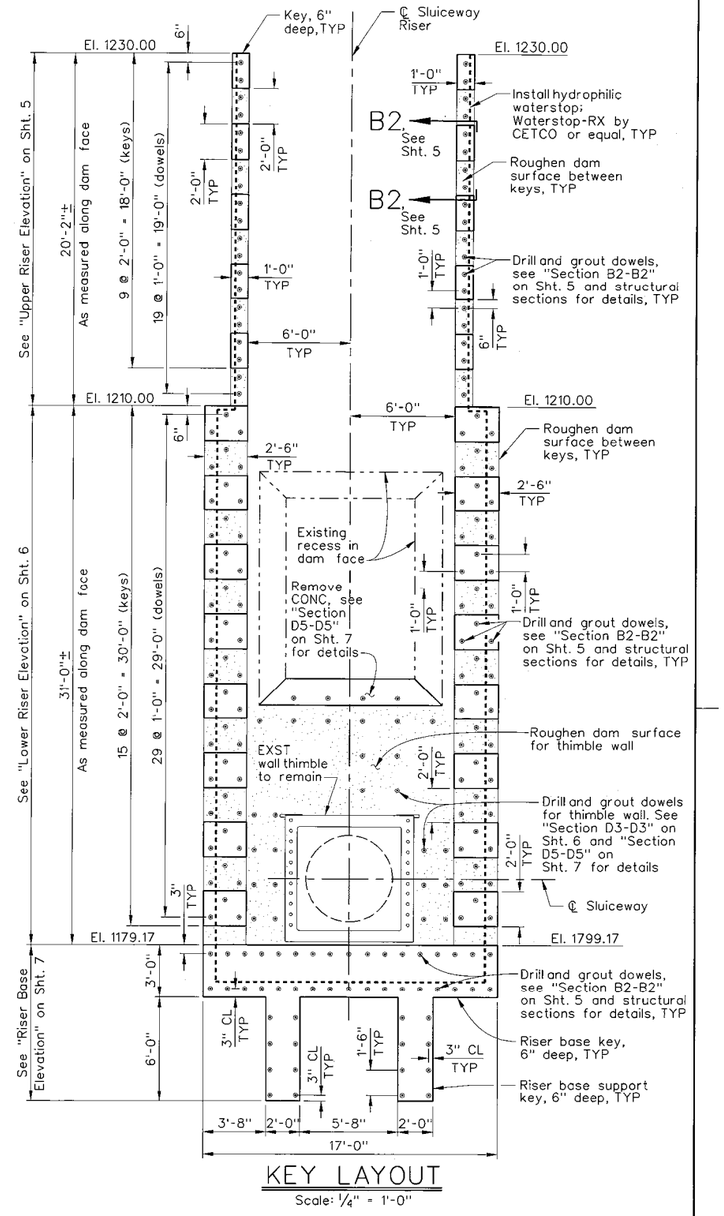


SIDE ELEVATION

Scale: 1/4" = 1'-0"

SECTION B1-B1

Scale: 1/4" = 1'-0"



KEY LAYOUT

Scale: 1/4" = 1'-0"

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REGISTERED PROFESSIONAL ENGINEER
SCOTT D. GREGOWSKE
NO. 68087
EXP. 06/30/11
CIVIL
STATE OF CALIFORNIA

PROJECT ENGINEER 12/10/09 DATE

DRAFTER S. GREGOWSKE
DESIGNER S. GREGOWSKE
CHECKER KENNETH HO
CADD PROJECT FILE NAME WRD0D0016-STRUCTURES.DGN
REVIEWED by KENNETH HO 12/10/09
DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

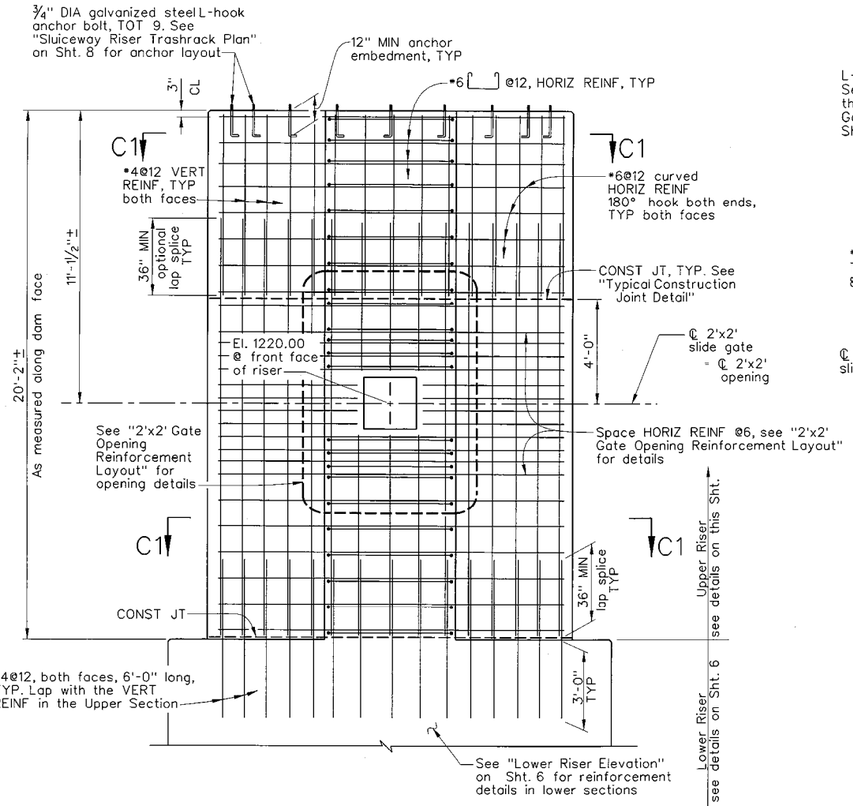
SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

SLUICEWAY RISER ELEVATIONS

PIN WRD0D0016 | JOB X5009661 | DWG 55-D92.4 | SHEET 4 OF 21

PS-ZB 07/05

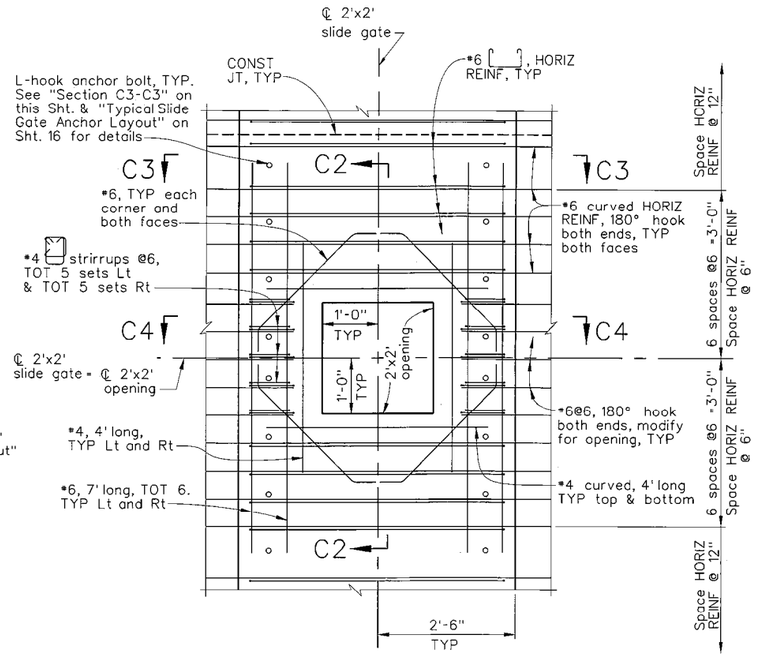
DATE: $DATE$ TIME: $TIME$ FILE: $FILE$



UPPER RISER ELEVATION

Scale: ⅜" = 1'-0"

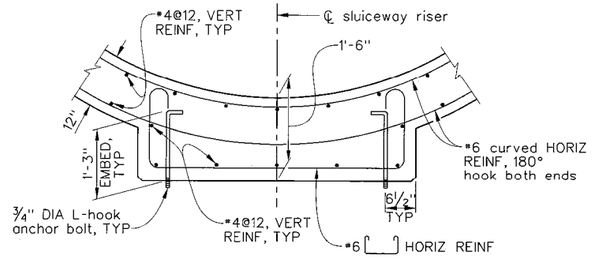
Note: For clarity, addtional reinforcement for 2'x2' opening not shown.



2'x2' GATE OPENING REINFORCEMENT LAYOUT

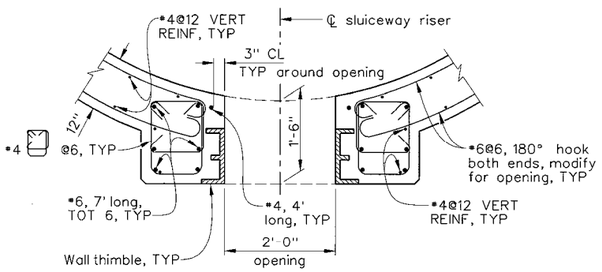
Scale: ¾" = 1'-0"

Note: 1. Wall thimble not shown for clarity
2. Riser reinforcement not shown for clarity



SECTION C3-C3

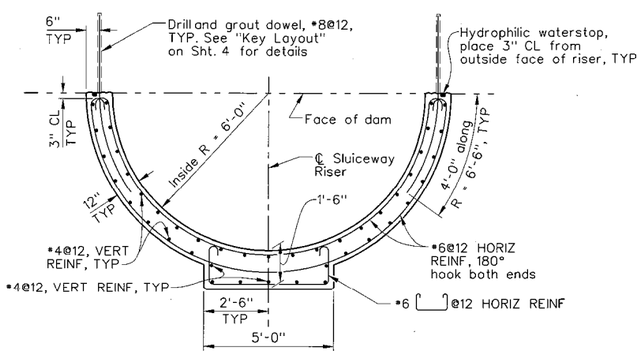
Scale: ¾" = 1'-0"



SECTION C4-C4

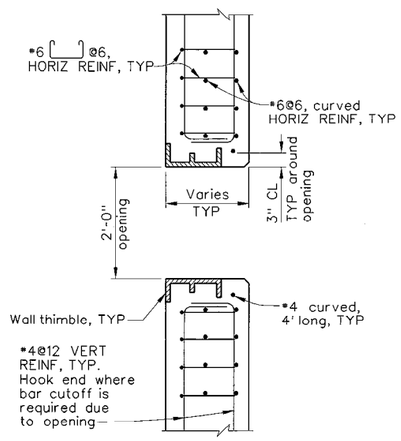
Scale: ¾" = 1'-0"

Note: For clarity, cast-in-place anchor bolts and vertical reinforcement not shown.



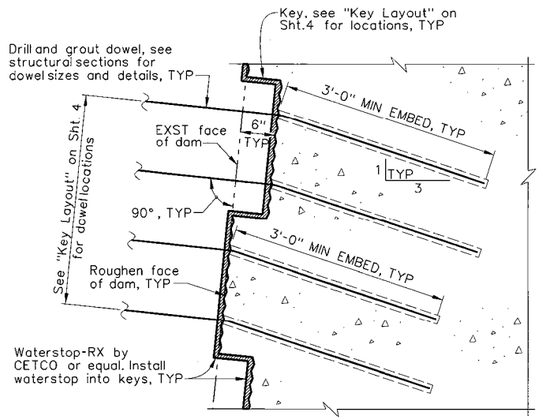
SECTION C1-C1

Scale: ⅜" = 1'-0"



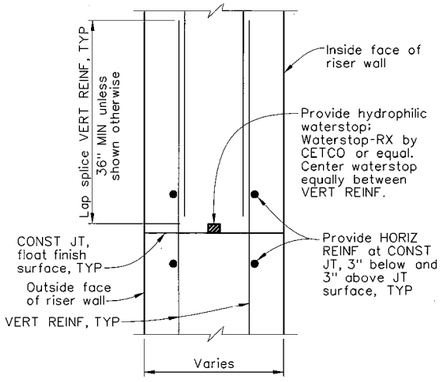
SECTION C2-C2

Scale: ¾" = 1'-0"



SECTION B2-B2

Scale: 1" = 1'-0"



TYPICAL CONSTRUCTION JOINT DETAIL

Not to Scale

NOTE:
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REGISTERED PROFESSIONAL ENGINEER — SCOTT D. GREGOWSKE — NO. 68087 — EXP. 06/30/11 — CIVIL — STATE OF CALIFORNIA

PROJECT ENGINEER 12/10/09 DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA DAM AND RESERVOIR RISER MODIFICATION

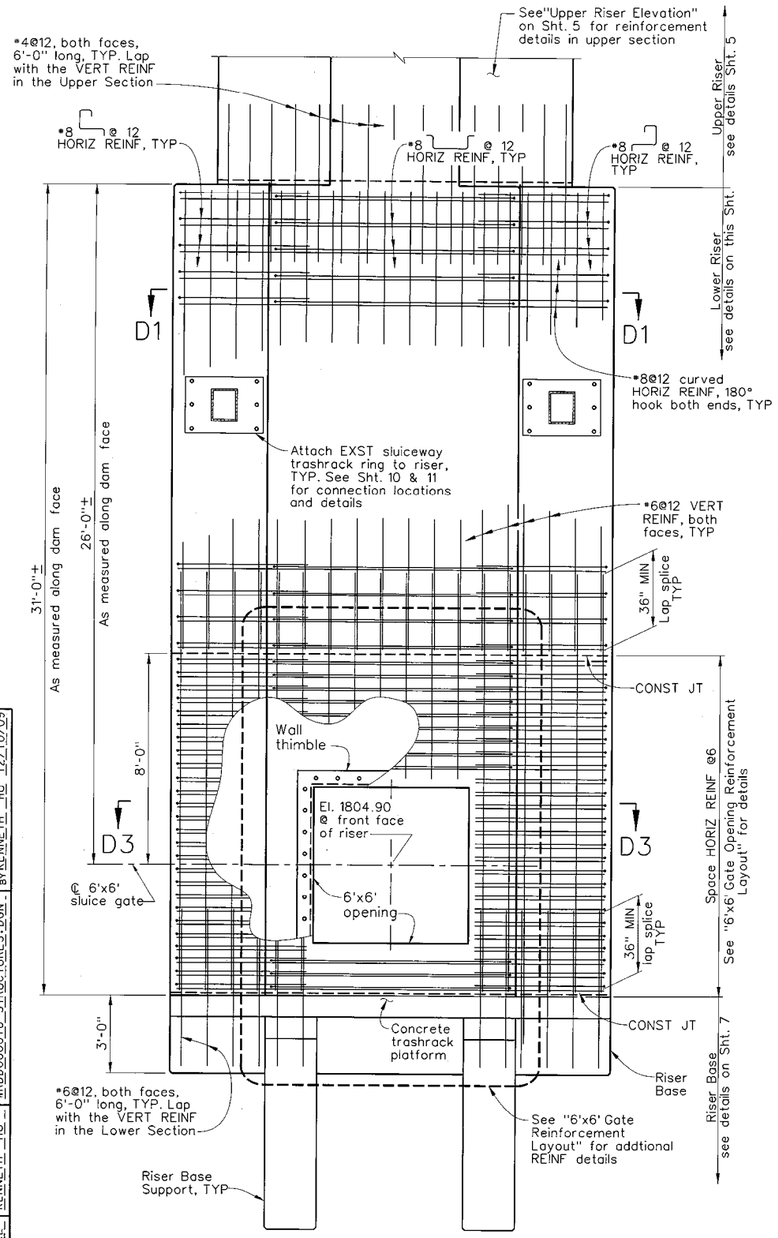
UPPER RISER CONCRETE DETAILS

PIN WRDD000016 | JOB X5009661 | DWG 55-D92.5 | SHEET 5 OF 21

PS-CD 07/03

DRAFTER S. GREGOWSKE | DESIGNER S. GREGOWSKE | CHECKER KENNETH HO | CADD PROJECT FILE NAME WRDD000016-STRUCTURES.DGN | REVIEWED BY KENNETH HO DATE 12/10/09

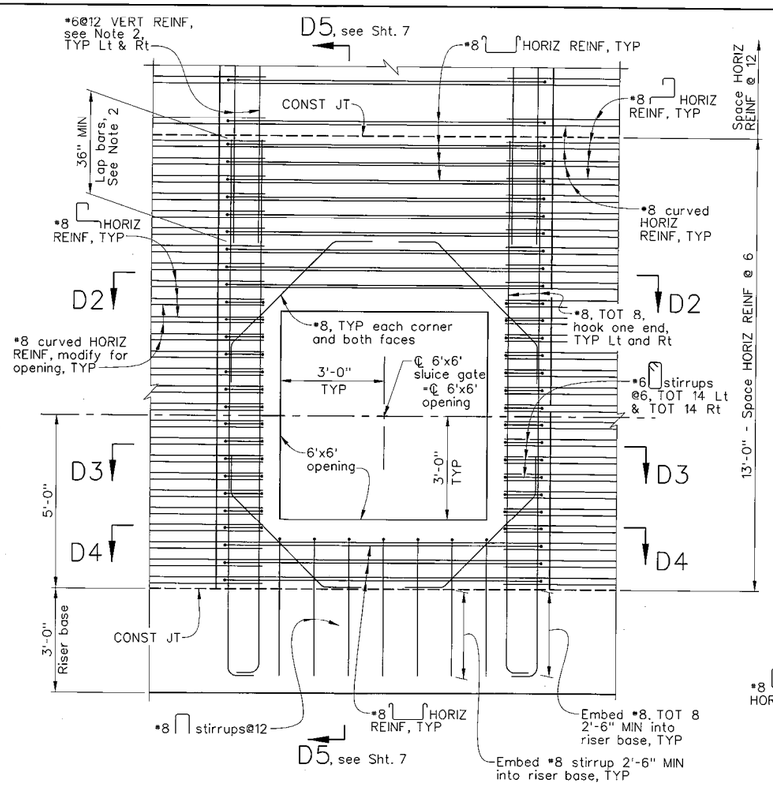
DATE: $DATE$ TIME: $TIME$ FILE: $FILE$



LOWER RISER ELEVATION

Scale: 3/8" = 1'-0"

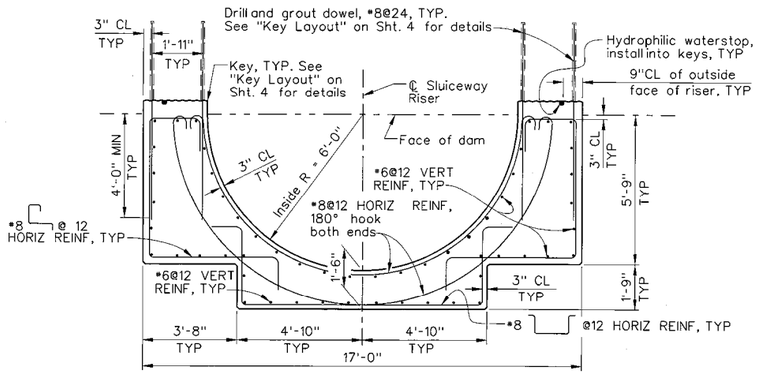
Notes:
1. For clarity, concrete trashrack ring is not shown.
2. For clarity, addtional reinforcement for 6'x6' opening not shown.
3. For clarity, riser base reinforcement for not shown. See Sheet 7 for riser base reinforcement details.



6'x6' GATE OPENING REINFORCEMENT LAYOUT

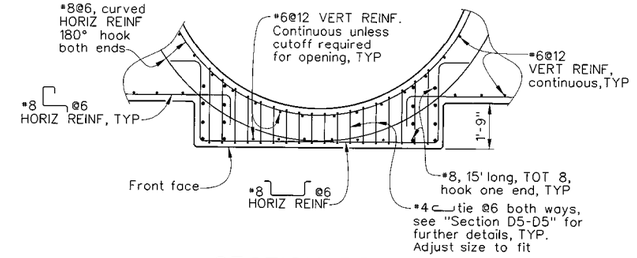
Scale: 1/2" = 1'-0"

Notes:
1. For clarity, wall thimble, concrete trashrack platform, riser base supports, and vertical riser reinforcement are not shown
2. Lap *6@12 VERT REINF with *8 bars. Lap only the *6 bars placed on the front face on the riser.



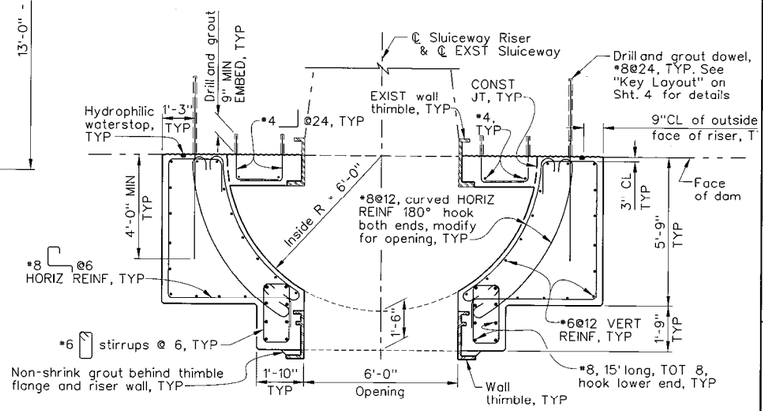
SECTION D1-D1

Scale: 3/8" = 1'-0"



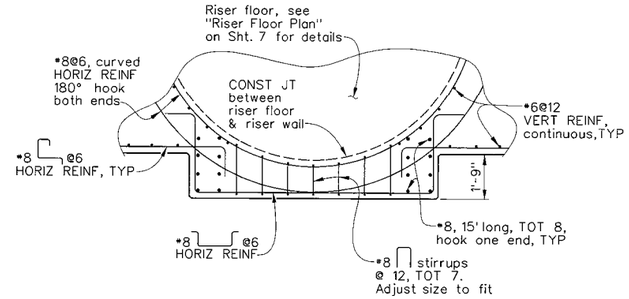
SECTION D2-D2

Scale: 3/8" = 1'-0"



SECTION D3-D3

Scale: 3/8" = 1'-0"



SECTION D4-D4

Scale: 3/8" = 1'-0"

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COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

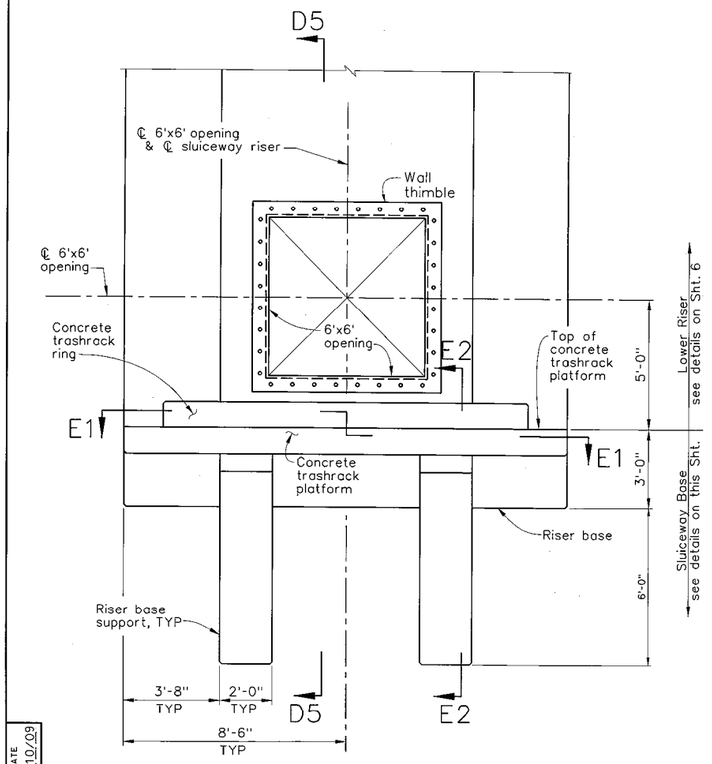
SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

LOWER RISER CONCRETE DETAILS

PIN WRDD000016 | JOB X5009661 | DWG 55-D92.6 | SHEET 6 OF 21

DRAFTER S. GREGOWSKE | DESIGNER S. GREGOWSKE | CHECKER KENNETH HO | CADD PROJECT FILE NAME WRDD000016-STRUCTURES.DGN | REVIEWED by KENNETH HO 12/10/09 DATE

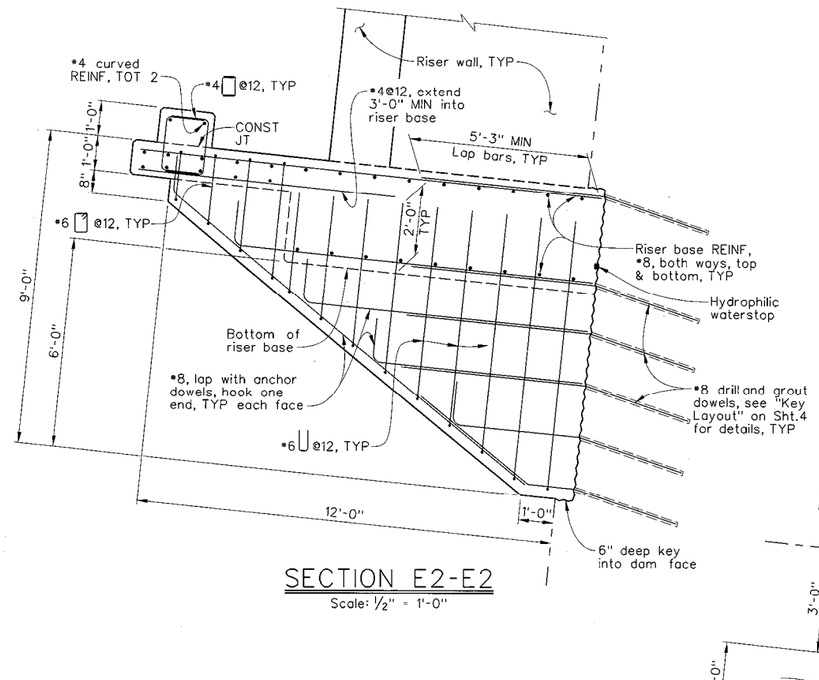
DATE: $DATE$ TIME: $TIME$ FILE: $FILE$



RISER BASE ELEVATION

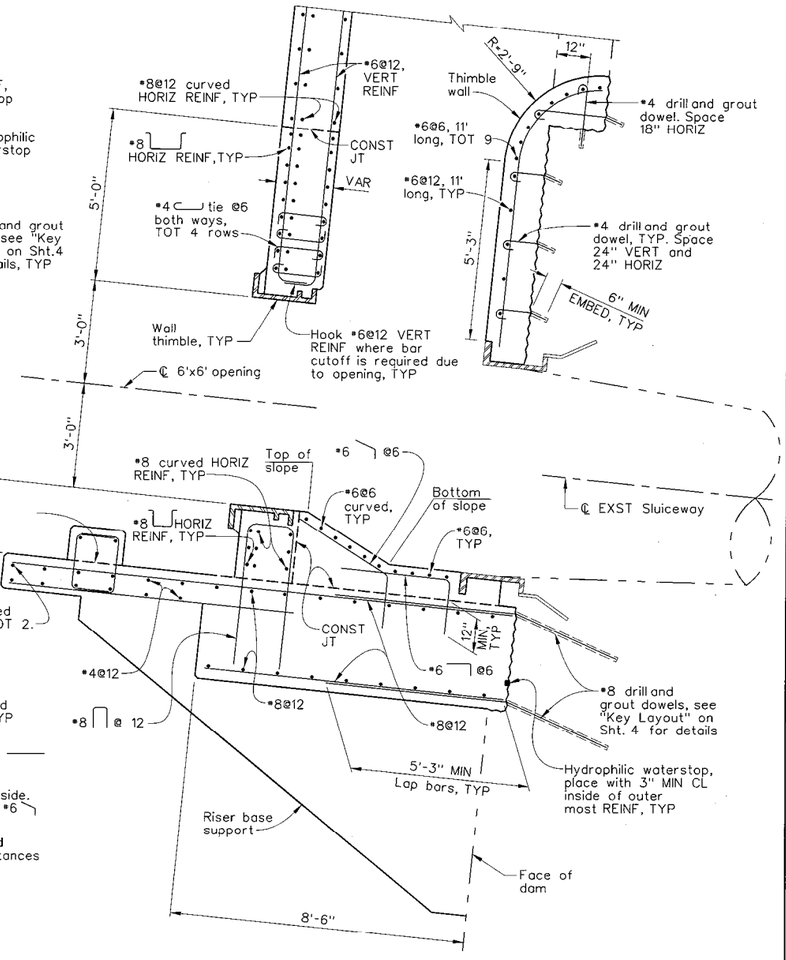
Scale: 3/8" = 1'-0"

Notes: 1. For clarity, concrete trashrack ring is not shown.



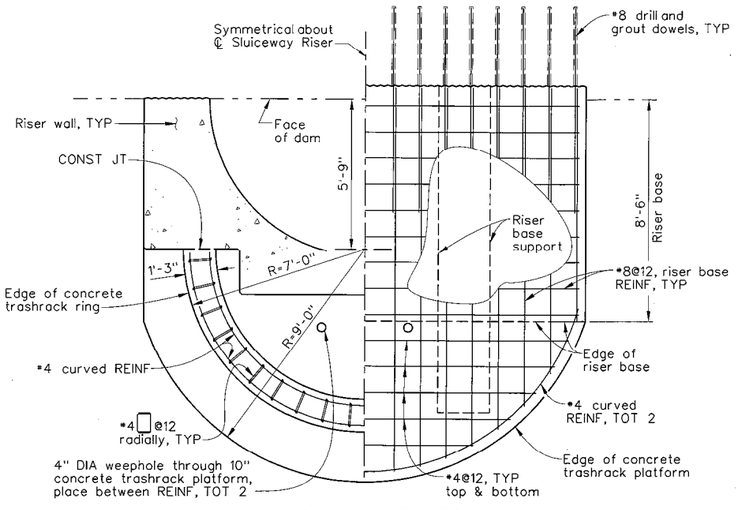
SECTION E2-E2

Scale: 1/2" = 1'-0"



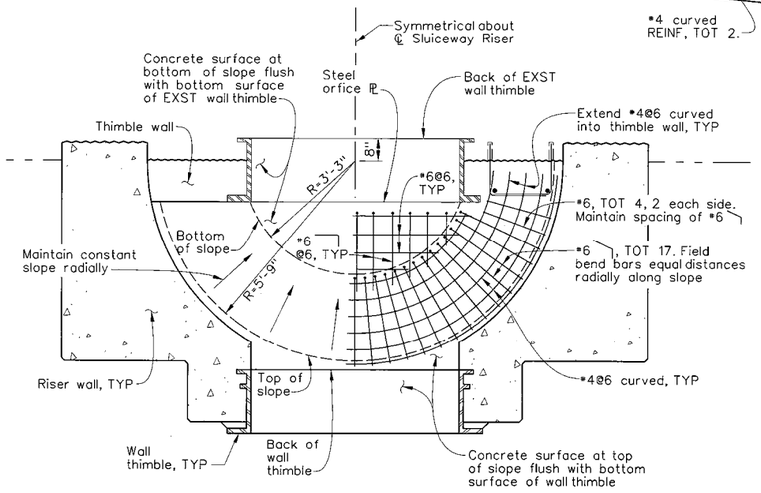
SECTION D5-D5

Scale: 1/2" = 1'-0"



SECTION E1-E1

Scale: 3/8" = 1'-0"



RISER FLOOR PLAN

Scale: 1/2" = 1'-0"

NOTE:
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DRAFTER S. GREGOWSKE
DESIGNER S. GREGOWSKE
CHECKER KENNETH HO
CADD PROJECT FILE NAME WRDD000016-STRUCTURES.DGN
REVIEWED by KENNETH HO DATE 12/10/09

REGISTERED PROFESSIONAL ENGINEER
SCOTT D. GREGOWSKE
NO. 68087
EXP. 06/30/11
CIVIL
STATE OF CALIFORNIA

PROJECT ENGINEER 12/10/09 DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

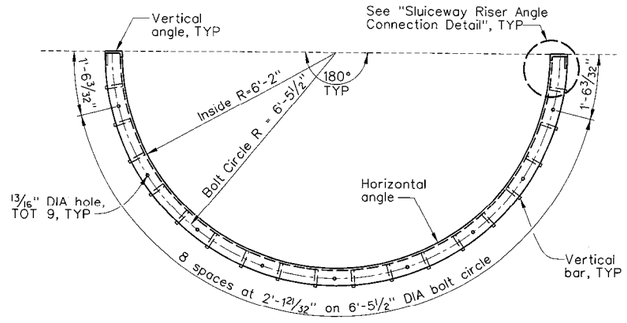
SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

RISER BASE CONCRETE DETAILS

PIN WRDD000016 | JOB X5009661 | DWG 55-D92.7 | SHEET 7 OF 21

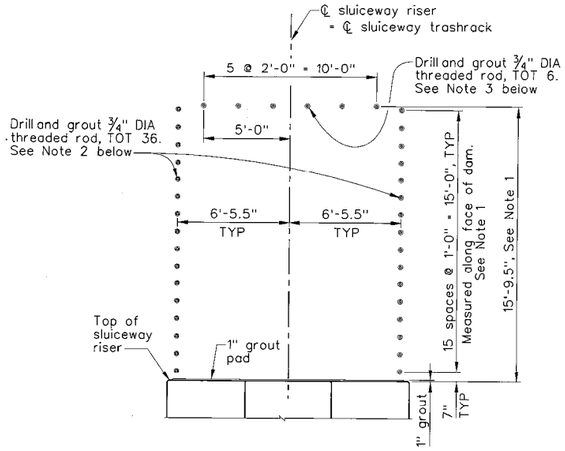
PS-DB 07/05

DATE: $DATE$ TIME: $TIME$ FILE: $FILE$



SLUICEWAY RISER TRASHRACK PLAN

Scale: 1/2" = 1'-0"

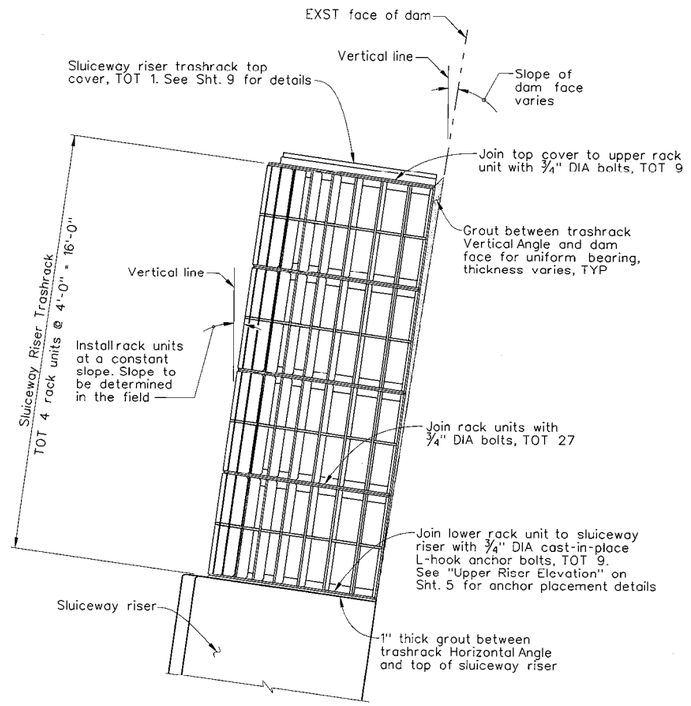


SLUICEWAY RISER TRASHRACK ANCHOR LAYOUT

Scale: 1/2" = 1'-0"

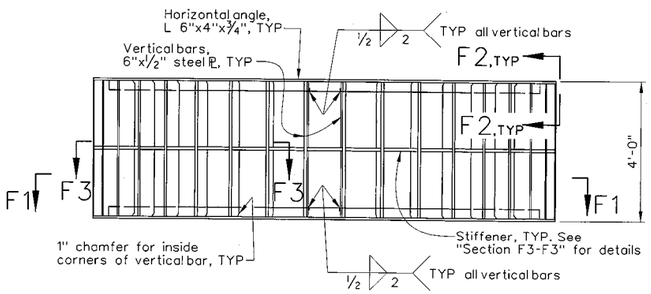
Notes:

1. Conctractor shall verify anchor layout dimension prior to placing drill and grout anchors.
2. See "Section F2-F2" on this Sht. and TYP anchor embedment.
3. See "Section G2-G2" on Sht. 9 for anchor embedment and layout details.



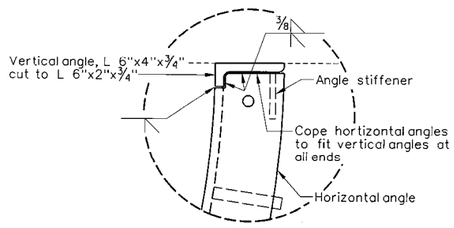
SLUICEWAY RISER TRASHRACK INSTALLATION

Scale: 3/8" = 1'-0"



SLUICEWAY RISER TRASHRACK ELEVATION

Scale: 1/2" = 1'-0"

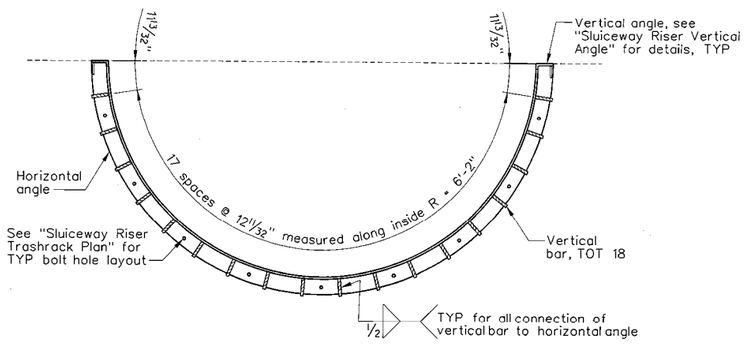


SLUICEWAY RISER ANGLE CONNECTION DETAIL

Scale: 2" = 1'-0"

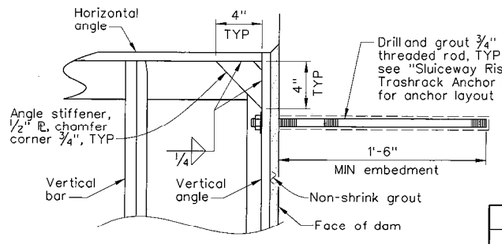
Notes:

1. All bolts, threaded anchor rods, L-hook anchor rods shall be galvanized, and included nut and washer.



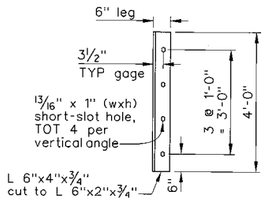
SECTION F1-F1

Scale: 1/2" = 1'-0"



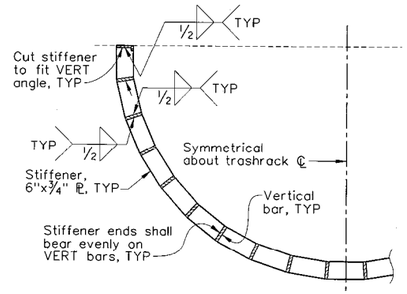
SECTION F2-F2

Scale: 2" = 1'-0"



SLUICEWAY RISER VERTICAL ANGLE

Scale: 1/2" = 1'-0"



SECTION F3-F3

Scale: 1/2" = 1'-0"

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SCOTT D. GREGOWSKE — No. 68087 — EXP. 06/30/11 — CIVIL — REGISTERED PROFESSIONAL ENGINEER — STATE OF CALIFORNIA

PROJECT ENGINEER — 12/10/09 DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

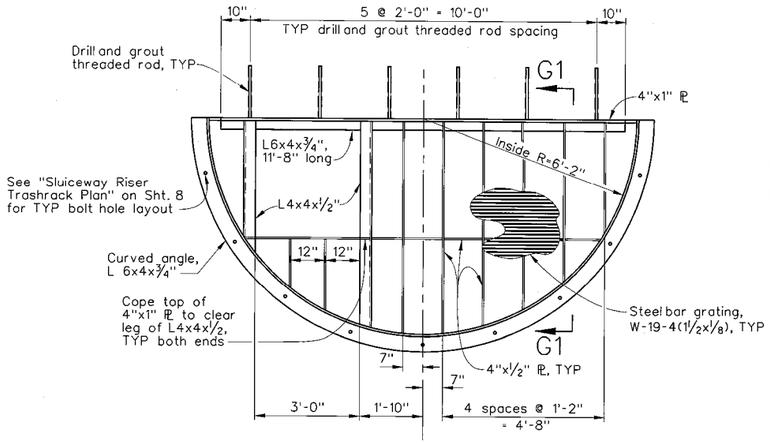
SLUICEWAY RISER TRASHRACK DETAILS

PIN WRDD000016 | JOB X5009661 | DWG 55-D92.8 | SHEET 8 OF 21

PS-CG 07/05

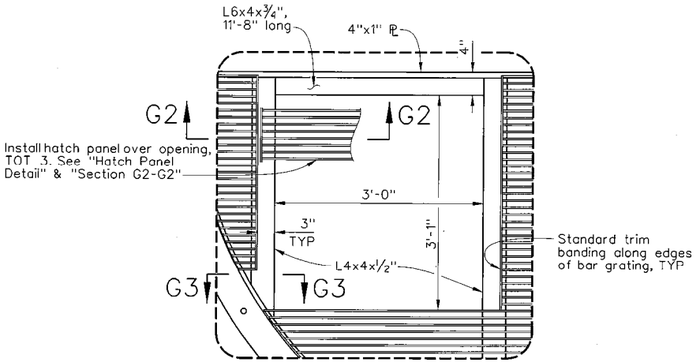
DRAFTER S. GREGOWSKE | DESIGNER S. GREGOWSKE | CHECKER KENNETH HO | CADD PROJECT FILE NAME WRDD000016-STRUCTURES.DGN | REVIEWED By KENNETH HO DATE 12/10/09

DATE: $DATE$ TIME: $TIME$ FILE: $FILE$



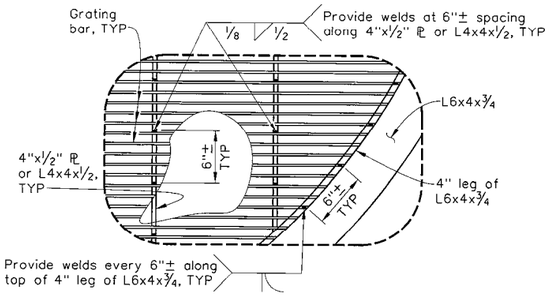
SLUICEWAY RISER TRASHRACK TOP COVER

Scale: 1/2" = 1'-0"



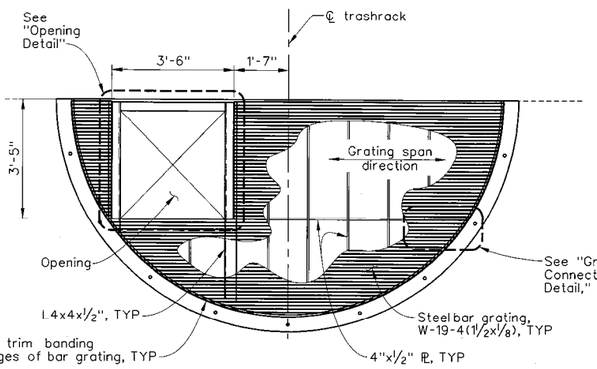
OPENING DETAIL

Scale: 1" = 1'-0"



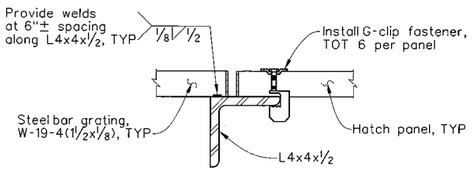
GRATING CONNECTION DETAIL

Not to scale



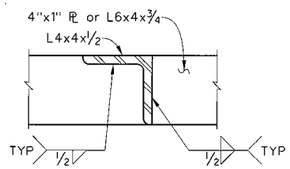
TOP COVER GRATING DETAIL

Scale: 1/2" = 1'-0"



SECTION G2-G2

Not to Scale



SECTION G3-G3

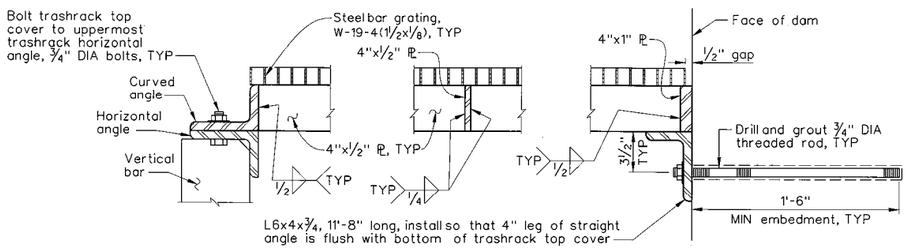
Not to Scale

HATCH PANEL DETAIL

Not to scale

Notes:

1. All bolts, threaded anchor rods, and G-clip fasteners shall be galvanized, and include nut and washer.



SECTION G1-G1

Scale: 2" = 1'-0"

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

DRAFTER	DESIGNER	CHECKER	CADD PROJECT FILE NAME	REVIEWED BY	DATE
S. GREGOWSKE	S. GREGOWSKE	KENNETH HO	WRDD000016-STRUCTURES.DGN	KENNETH HO	12/12/09

DATE	MK	DESCRIPTION
		REVISIONS

REGISTERED PROFESSIONAL ENGINEER — SCOTT D. GREGOWSKE — NO. 68087 — EXP. 06/30/11 — CIVIL — STATE OF CALIFORNIA

PROJECT ENGINEER — 12/10/09 DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

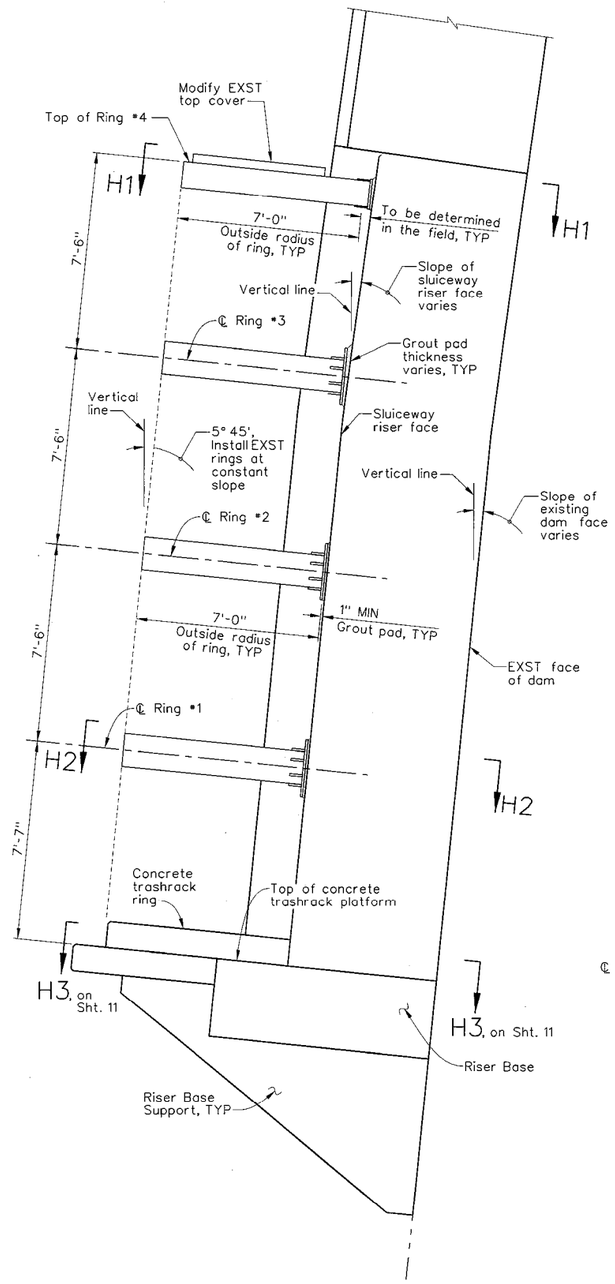
SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

SLUICEWAY RISER TRASHRACK COVER DETAILS

PIN WRDD000016	JOB X5009661	DWG 55-D92.9	SHEET 9 OF 21

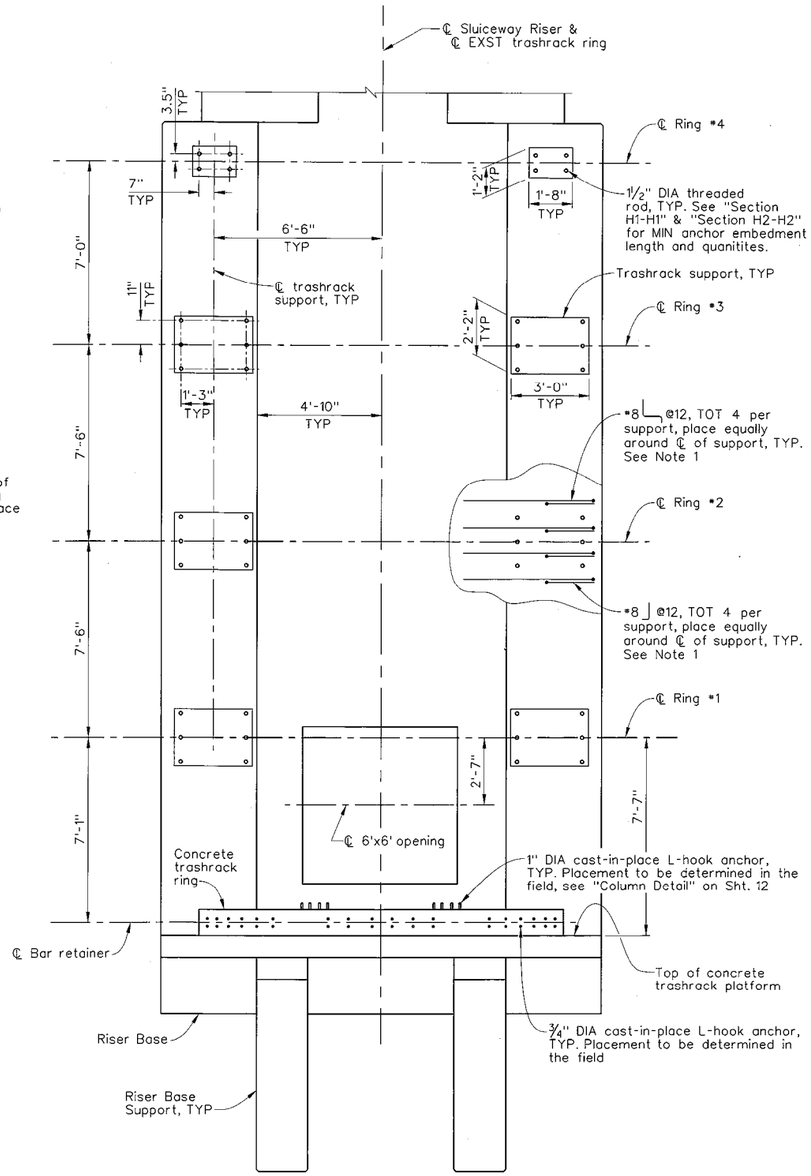
PS:DB:GMS

DATE: $DATE$ TIME: $TIME$ FILE: $FILE$



EXISTING SLUICEWAY TRASHRACK INSTALLATION

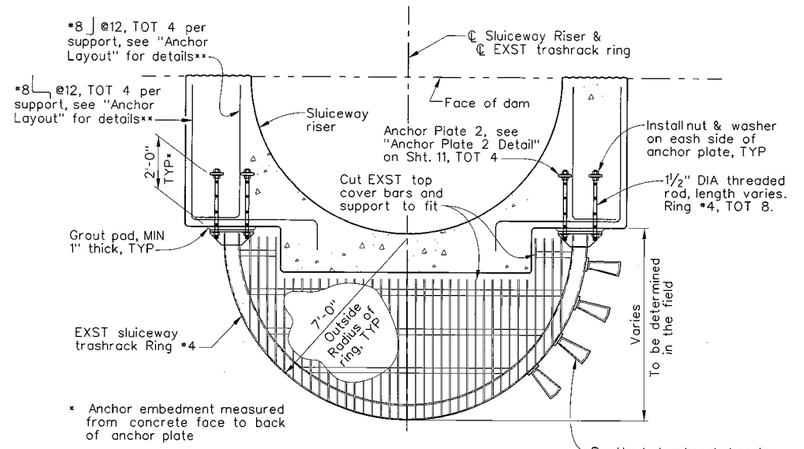
Scale: 3/8" = 1'-0"



EXISTING SLUICEWAY TRASHRACK ANCHOR LAYOUT

Scale: 3/8" = 1'-0"

Notes: 1. #8@12 reinforcement in addition to the riser reinforcement shown on Sheet 6. The addtional #8@12 reinforcement is not required at Ring #1 supports.



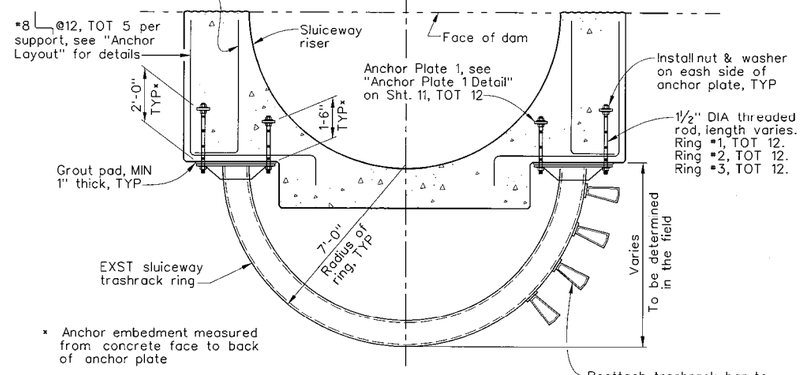
* Anchor embedment measured from concrete face to back of anchor plate

** #8@12 REINF in addition to the riser reinforcement shown on Sht. 6.

SECTION H1-H1

Scale: 3/8" = 1'-0"

Note: For clarify, not all trashrack bars shown.



* Anchor embedment measured from concrete face to back of anchor plate

** #8@12 REINF in addition to the riser reinforcement shown on Sht. 6. The addtional #8@12 REINF not required at Ring #1 supports

SECTION H2-H2

Scale: 3/8" = 1'-0"

Note: For clarify, not all trashrack bars shown.

NOTE:
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DRAFTER	DESIGNER	CHECKER	CADD PROJECT FILE NAME	REVIEWED	DATE
S. GREGOWSKE	S. GREGOWSKE	KENNETH HO	WRDD000016-STRUCTURES.DGN	by KENNETH HO	12/10/09

DATE	MK	DESCRIPTION
		REVISIONS

REGISTERED PROFESSIONAL ENGINEER — SCOTT D. GREGOWSKE — NO. 68087 — EXP. 06/30/11 — CIVIL — STATE OF CALIFORNIA

PROJECT ENGINEER — 12/10/09 — DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

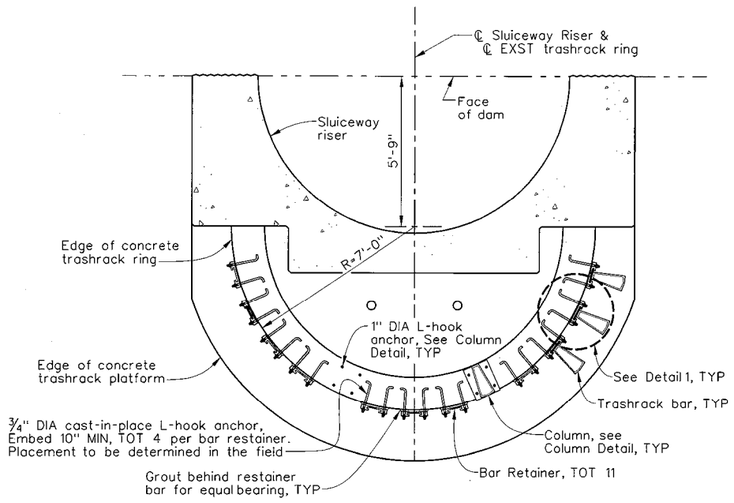
SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

EXISTING SLUICEWAY TRASHRACK RELOCATION DETAILS #1

PW PIN WRDD000016 | JOB X5009661 | DWG 55-D92.10 | SHEET 10 OF 21

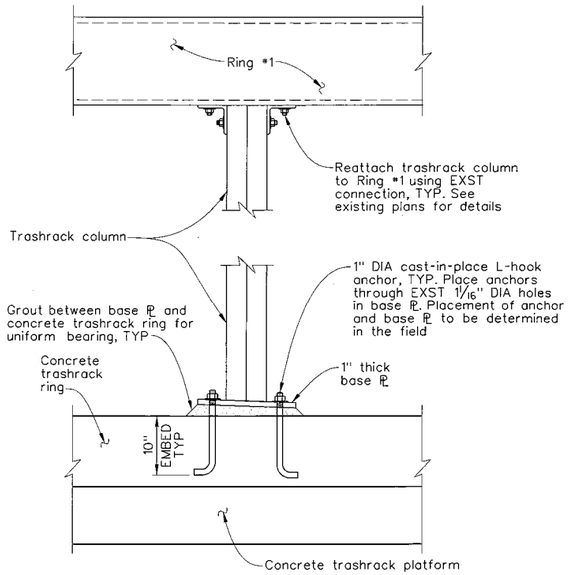
FS-DS 07/05

DATE: $DATE$ TIME: $TIME$ FILE: $FILE$



SECTION H3-H3

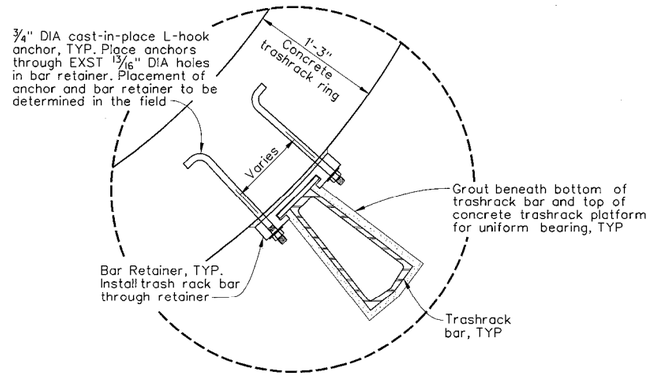
Scale: ⅜" = 1'-0"



FRONT ELEVATION

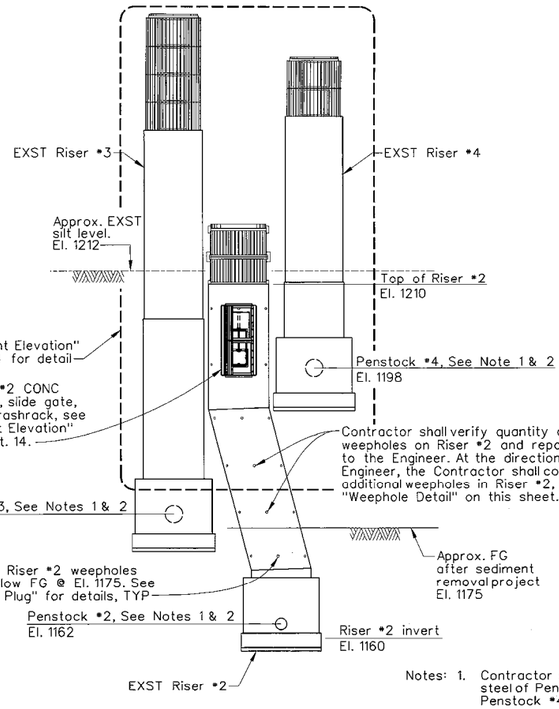
COLUMN DETAIL

Scale: 1" = 1'-0"



DETAIL 1

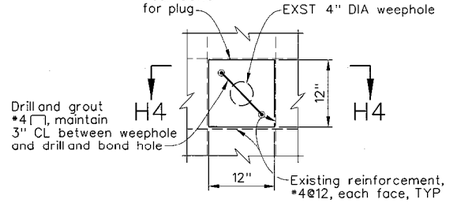
Not to Scale



RISER #2 WEEPHOLES

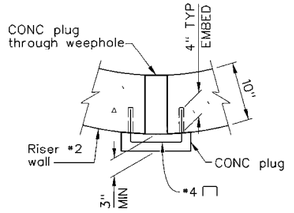
Scale: 1" = 10'

Notes:
1. Contractor shall clean and paint existing steel of Penstock #2, Penstock #3, and Penstock #4.
1. Existing paint system in all penstocks may contain lead based paint. Contractor shall refer to the Special Provisions for requirements regarding lead based paint.



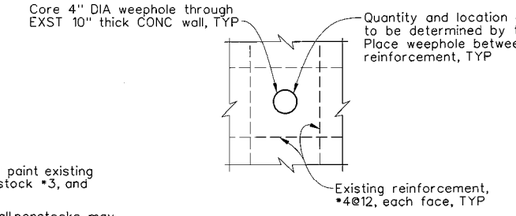
WEEPHOLE PLUG

Scale: 1" = 1'-0"



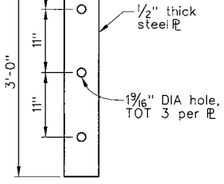
SECTION H4-H4

Scale: 1" = 1'-0"



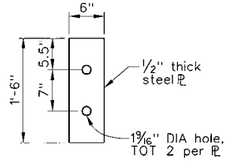
WEEPHOLE DETAIL

Scale: 1" = 1'-0"



ANCHOR PLATE 1

Scale: 1" = 1'-0"



ANCHOR PLATE 2

Scale: 1" = 1'-0"

NOTE:
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DATE	MK	DESCRIPTION

REVISIONS

SCOTT D. GREGOWSKE, No. 68087, Exp. 06/30/11, CIVIL, State of California

PROJECT ENGINEER — DATE 12/10/09

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

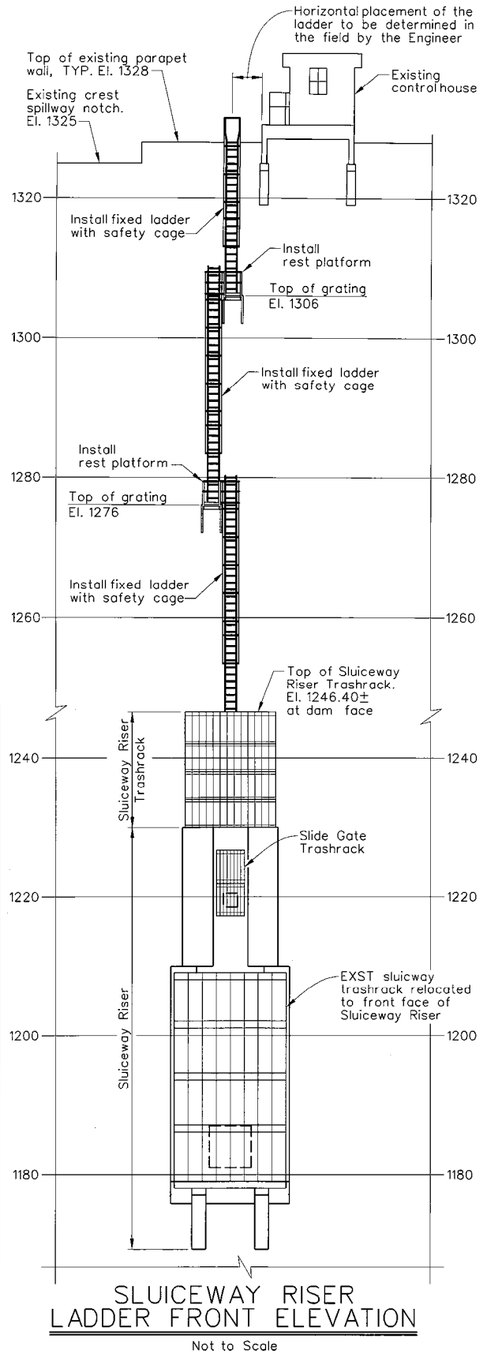
EXISTING SLUICEWAY TRASHRACK RELOCATION DETAILS #2

PIN WRDD000016 | JOB X5009661 | DWG 55-D92.11 | SHEET 11 OF 21

PS-DSI 07/05

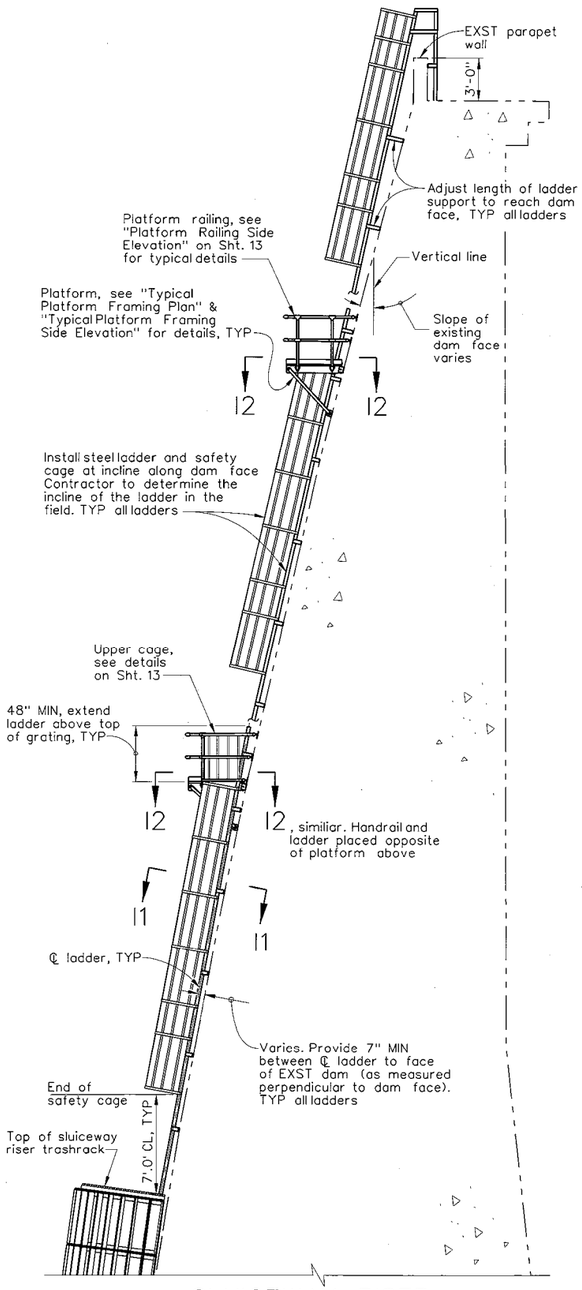
DRAFTER S. GREGOWSKE | DESIGNER S. GREGOWSKE | CHECKER KENNETH HO | CADD PROJECT FILE NAME WRDD000016-STRUCTURES.DGN | REVIEWED by KENNETH HO 12/10/09

DATE: $DATE$ TIME: $TIME$ FILE: $FILE$



SLUICEWAY RISER LADDER FRONT ELEVATION

Not to Scale



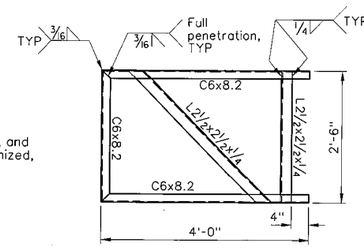
SLUICEWAY RISER LADDER SIDE ELEVATION

Not to Scale

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

Notes:

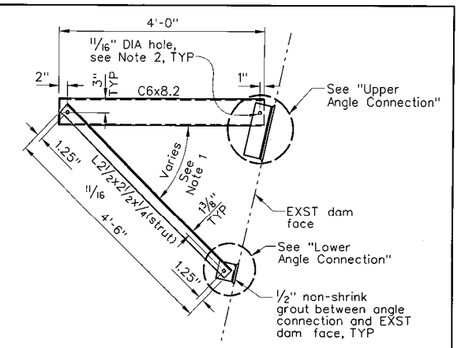
1. All bolts, threaded anchor rods, and G-clip fasteners shall be galvanized, and include nut and washer.



TYPICAL PLATFORM FRAMING PLAN

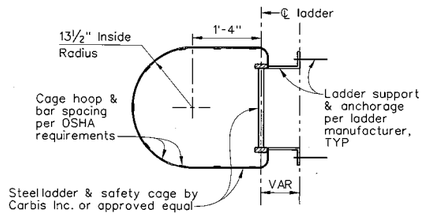
Scale: 3/4" = 1'-0"

Note: For clarity, platform struts and angle connections are not shown.



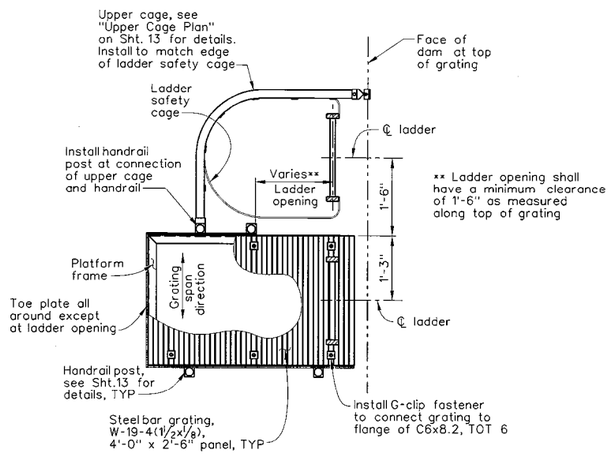
TYPICAL PLATFORM FRAMING SIDE ELEVATION

Scale: 3/4" = 1'-0"



SECTION 11-11

Scale: 3/4" = 1'-0"

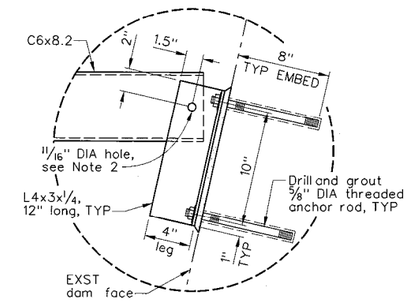


SECTION 12-12

Scale: 3/4" = 1'-0"

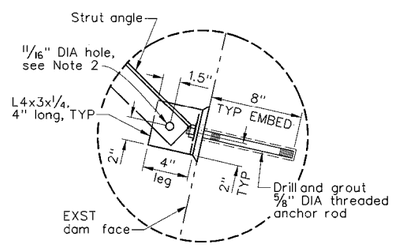
Notes: 1. Platform frame connections not shown for clarity.

2. Upper Platform shown. Lower platform is similiar but ladder, upper cage, and handrail deails opposite.



UPPER ANGLE CONNECTION

Scale: 2" = 1'-0"



LOWER ANGLE CONNECTION

Scale: 2" = 1'-0"

DATE	BY	DESCRIPTION
		REVISIONS

SCOTT D. GREGOWSKE, No. 68087, Exp. 06/30/11, CIVIL, REGISTERED PROFESSIONAL ENGINEER, STATE OF CALIFORNIA

PROJECT ENGINEER — DATE 12/10/09

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

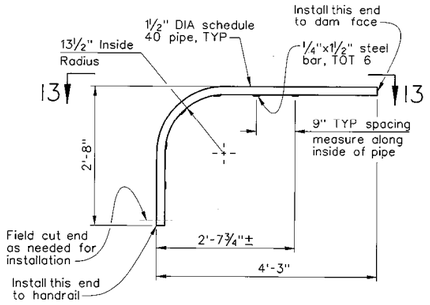
SLUICEWAY RISER LADDER DETAILS #1

PIN WRDD000016 | JOB X5009661 | DWG 55-D92.12 | SHEET 12 OF 21

PS-DB 07/05

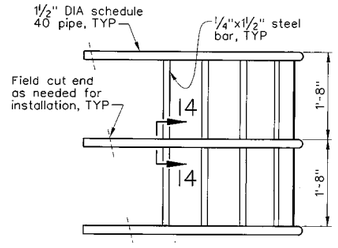
DRAFTER S_GREGOWSKE | DESIGNER S_GREGOWSKE | CHECKER KENNETH_HO | CADD PROJECT FILE NAME WRDD000016-STRUCTURES.DGN | REVIEWED by KENNETH_HO DATE 12/10/09

DATE: $DATE$ TIME: $TIME$ FILE: $FILE$



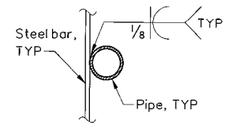
UPPER CAGE PLAN

Scale: ¾" = 1'-0"



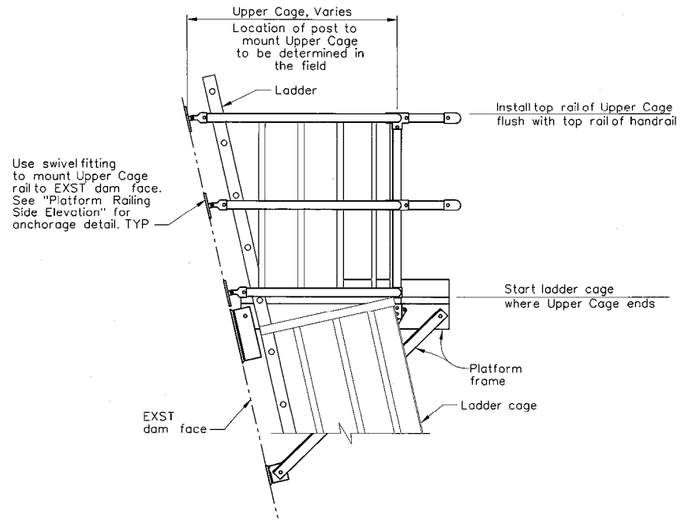
ELEVATION 13-13

Scale: ¾" = 1'-0"



SECTION 14-14

Not to Scale

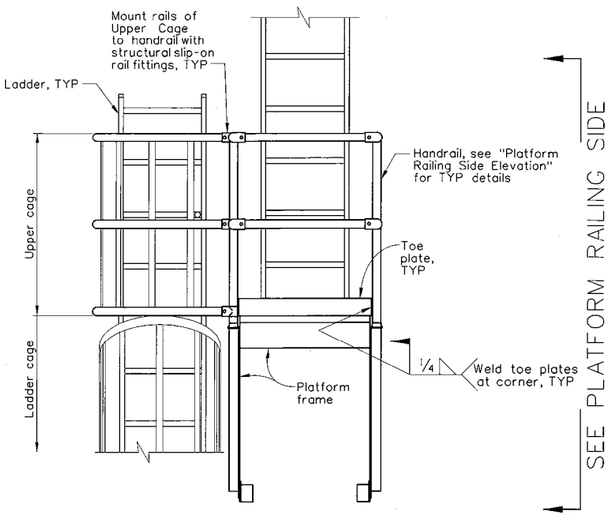


PLATFORM LADDER SIDE ELEVATION

Scale: ¾" - 1'-0"

Notes: 1. Upper Platform shown. Lower platform is similiar but details are opposite.

SEE PLATFORM LADDER SIDE ELEVATION

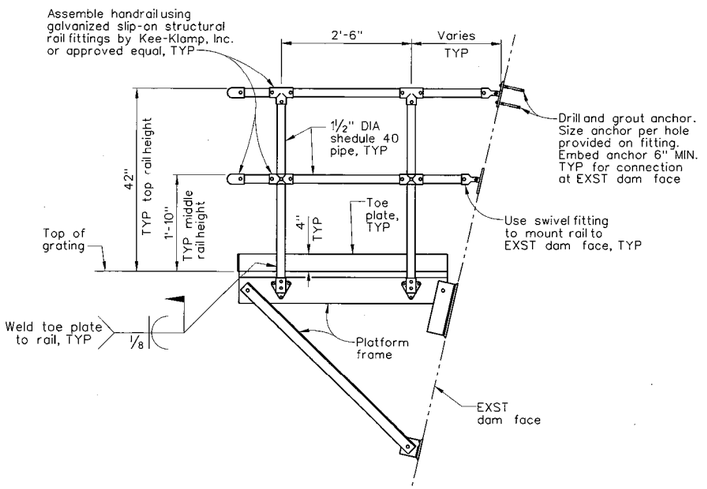


PLATFORM FRONT ELEVATION

Scale: ¾" - 1'-0"

Notes: 1. Upper Platform shown. Lower platform is similiar but details are opposite.

SEE PLATFORM RAILING SIDE ELEVATION



PLATFORM RAILING SIDE ELEVATION

Scale: ¾" - 1'-0"

Notes: 1. Ladders and cages not shown for clarity.
2. Upper Platform shown. Lower platform is similiar but details are opposite.

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

DRAFTER	DESIGNER	CHECKER	CADD PROJECT FILE NAME	REVIEWED	DATE
S. GREGOWSKE	S. GREGOWSKE	KENNETH HO	WRDD000016-STRUCTURES.DGN	By KENNETH HO	12/10/09

DATE	MK	DESCRIPTION
		REVISIONS

REGISTERED PROFESSIONAL ENGINEER
SCOTT D. GREGOWSKE
NO. 68087
EXP. 06/30/11
CIVIL
STATE OF CALIFORNIA

PROJECT ENGINEER — 12/10/09 DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

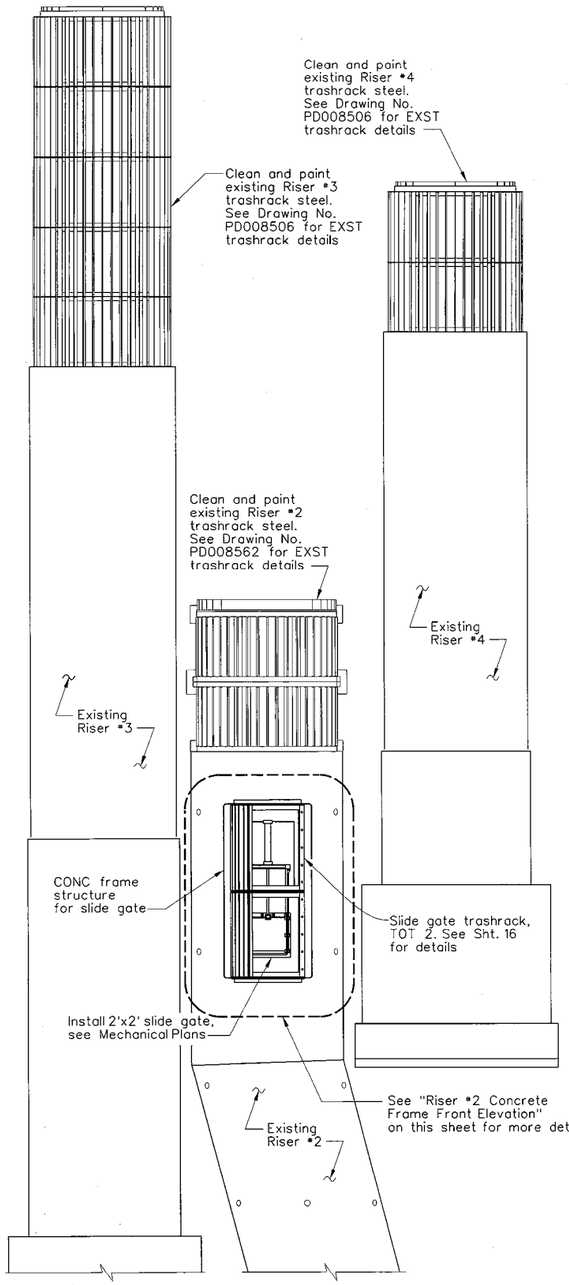
SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

SLUICEWAY RISER LADDER DETAILS #2

PIN WRDD000016	JOB X5009661	DWG 55-D92.13	SHEET 13 OF 21

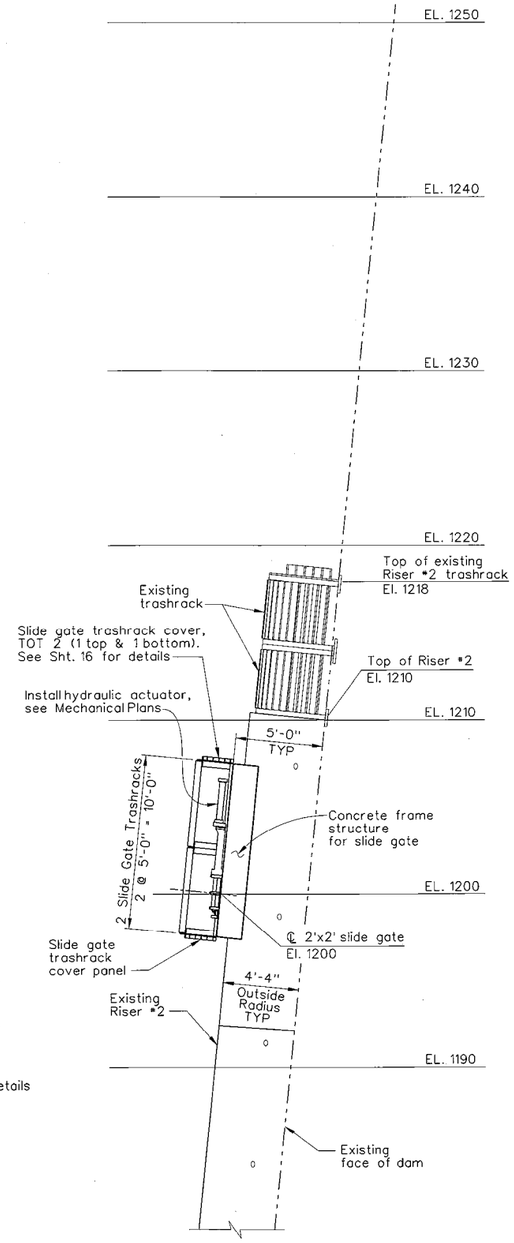
PS-DB 07/05

DATE. $DATE$ TIME. $TIME$ FILE. $FILE$



FRONT ELEVATION

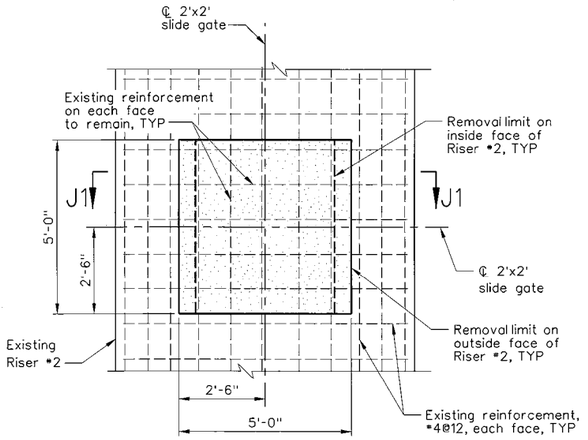
Scale: 1/4" = 1'-0"



RISER #2 SIDE ELEVATION

Scale: 1/4" = 1'-0"

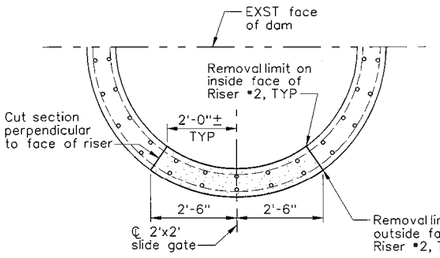
Note: For clarity, not all of the details of the Slide Gate Trashrack are shown.



CONCRETE REMOVAL

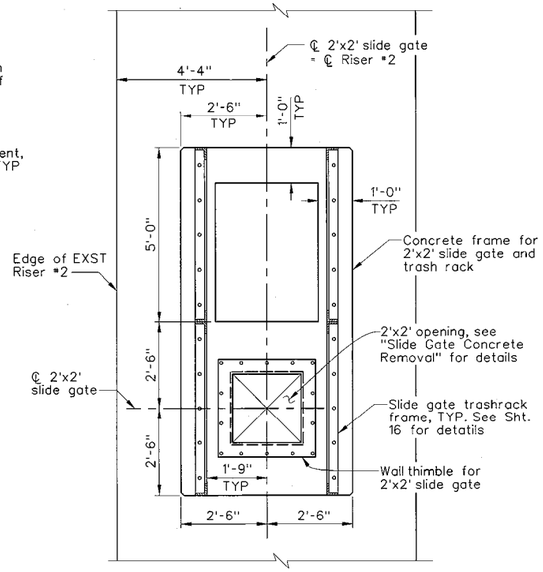
Scale: 1/2" = 1'-0"

Note: ▨ Concrete Removal Limit



SECTION J1-J1

Scale: 1/2" = 1'-0"



RISER #2 CONCRETE FRAME FRONT ELEVATION

Scale: 1/2" = 1'-0"

Note: Trash rack bars and slide gate not shown for clarity

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

DATE	MK	DESCRIPTION
		REVISIONS

REGISTERED PROFESSIONAL ENGINEER — SCOTT D. GREGOWSKE — NO. 68087 — EXP. 06/30/11 — CIVIL — STATE OF CALIFORNIA

PROJECT ENGINEER 12/10/09 DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

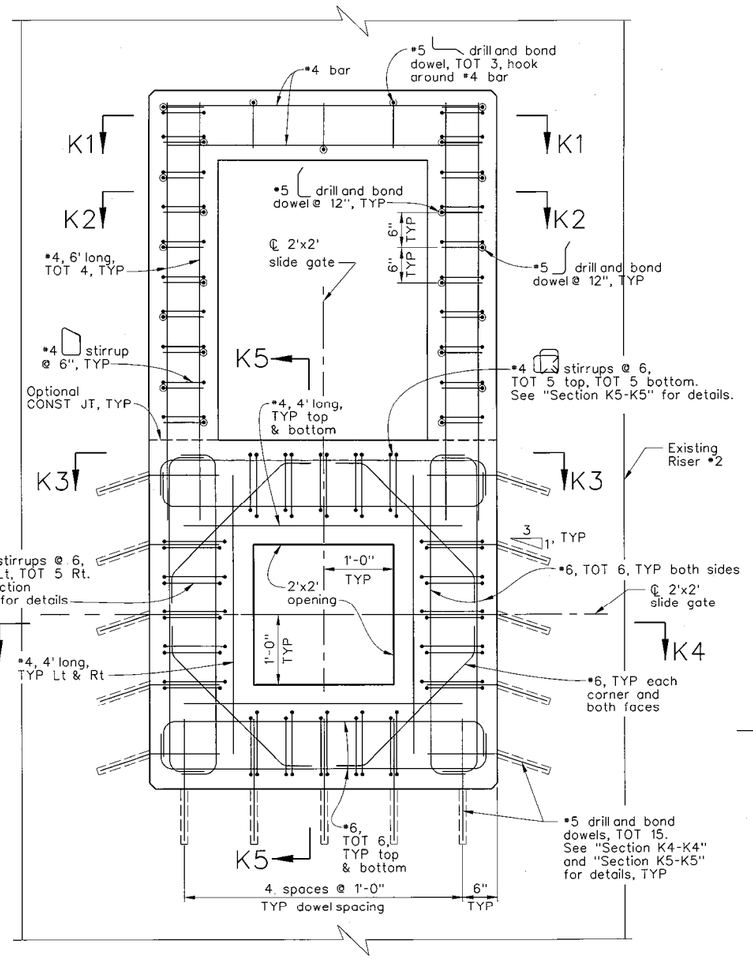
RISER #2 ELEVATIONS AND SECTIONS

PIN WRDD000016 | JOB X5009661 | DWG 55-092.14 | SHEET 14 OF 21

PG-CB 07/00

DRAFTER S. GREGOWSKE | DESIGNER S. GREGOWSKE | CHECKER KENNETH HO | CADD PROJECT FILE NAME WRDD000016-STRUCTURES.DGN | REVIEWED BY KENNETH HO DATE 12/10/09

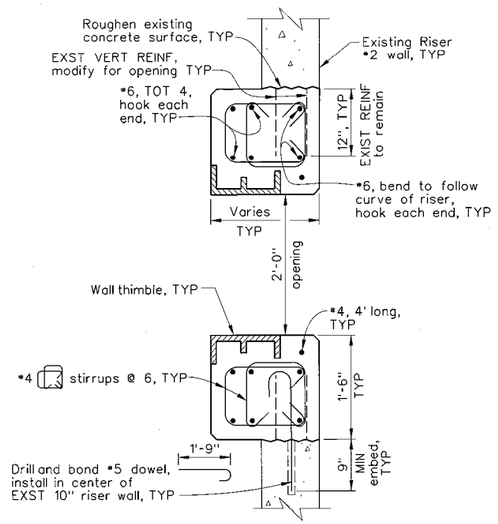
DATE: $DATE$ TIME: $TIME$ FILE: $FILE$



SLIDE GATE REINFORCEMENT LAYOUT

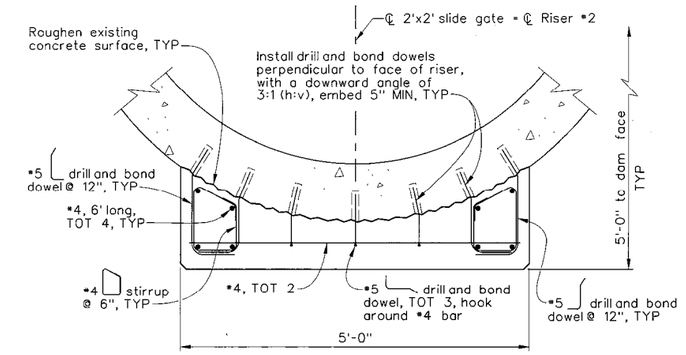
Scale: 1" = 1'-0"

Note: For clarity, wall thimble and existing reinforcement not shown.



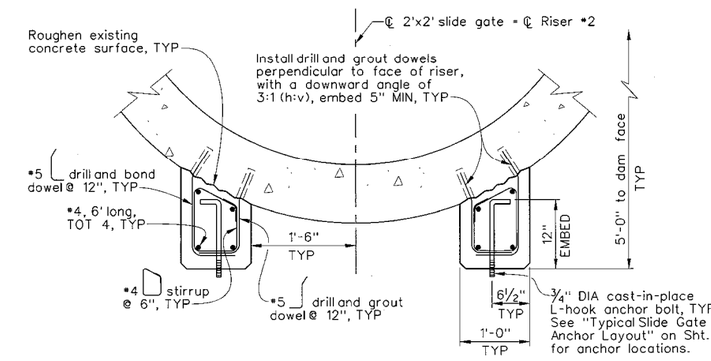
SECTION K5-K5

Scale: 1" = 1'-0"



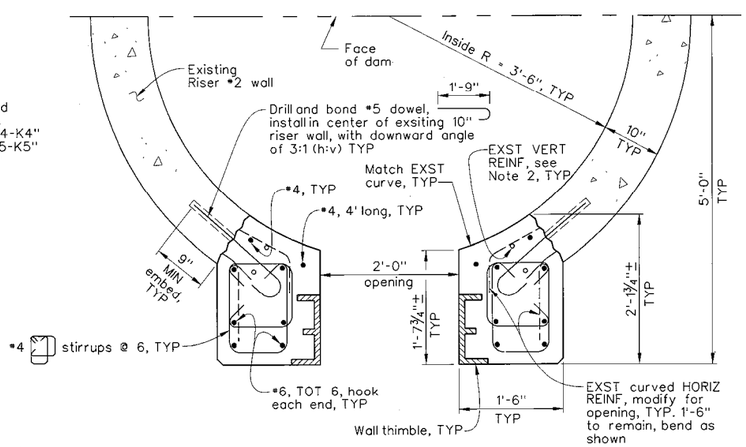
SECTION K1-K1

Scale: 1" = 1'-0"



SECTION K2-K2

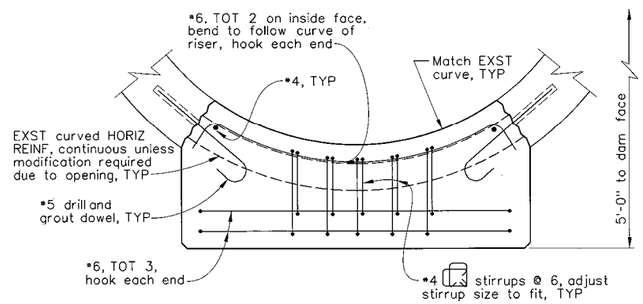
Scale: 1" = 1'-0"



SECTION K4-K4

Scale: 1" = 1'-0"

Notes: 1. For clarity, cast-in-place anchor bolts not shown.
2. Existing vertical reinforcement shall be continuous unless modification required due to opening.



SECTION K3-K3

Scale: 1" = 1'-0"

Note: For clarity, cast-in-place anchor bolts and vertical reinforcement not shown.

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

DATE	BY	DESCRIPTION

REVISIONS

REGISTERED PROFESSIONAL ENGINEER — SCOTT D. GREGOWSKE — No. 68087 — EXP. 06/30/11 — CIVIL — STATE OF CALIFORNIA

PROJECT ENGINEER — DATE 12/10/09

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

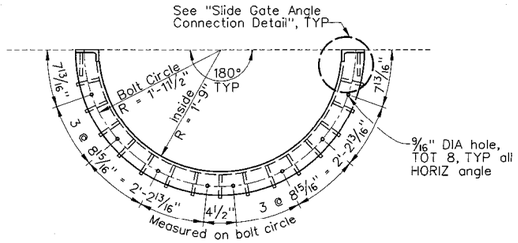
SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

RISER #2 CONCRETE DETAILS

PIN WRDD000016 | JOB X5009661 | DWG 55-D92.15 | SHEET 15 OF 21

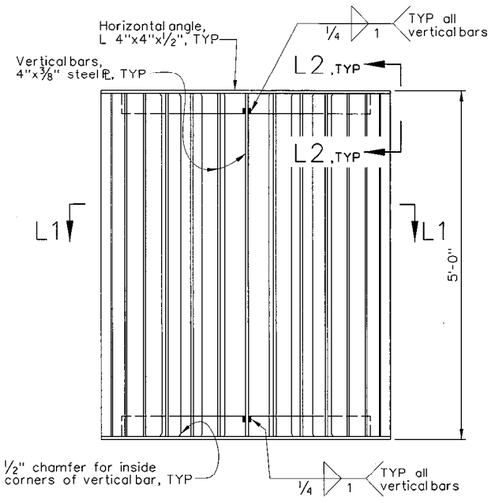
DRAFTER S. GREGOWSKE | DESIGNER S. GREGOWSKE | CHECKER KENNETH HO | CADD PROJECT FILE NAME WRDD000016-STRUCTURES.DGN | REVIEWED By KENNETH HO DATE 12/10/09

DATE, $DATE$ TIME, $TIME$ FILE, $FILE$



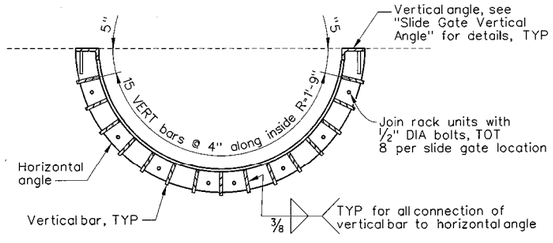
SLIDE GATE TRASHRACK PLAN

Scale: 1" = 1'-0"



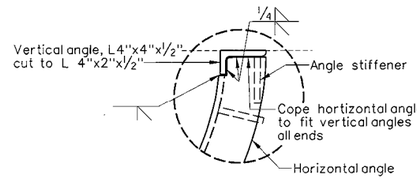
SLIDE GATE TRASHRACK ELEVATION

Scale: 1" = 1'-0"



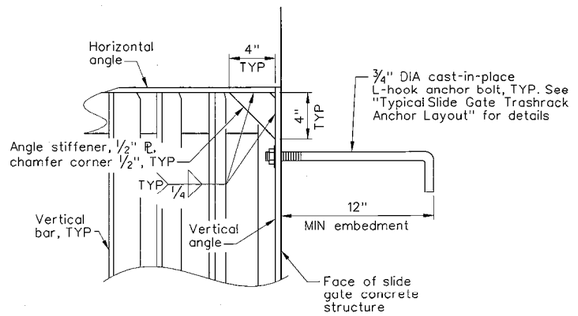
SECTION L1-L1

Scale: 1" = 1'-0"



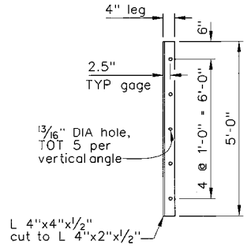
SLIDE GATE ANGLE CONNECTION DETAIL

Scale: 2" = 1'-0"



SECTION L2-L2

Scale: 2" = 1'-0"

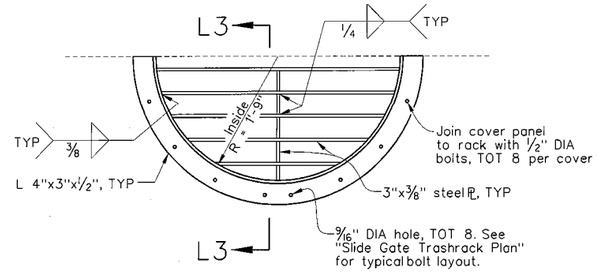


SLIDE GATE VERTICAL ANGLE

Scale: ½" = 1'-0"

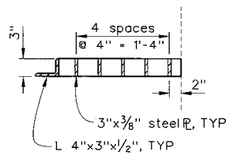
Notes:

1. All bolts, threaded anchor rods, L-hook anchor rods shall be galvanized, and included nut and washer.



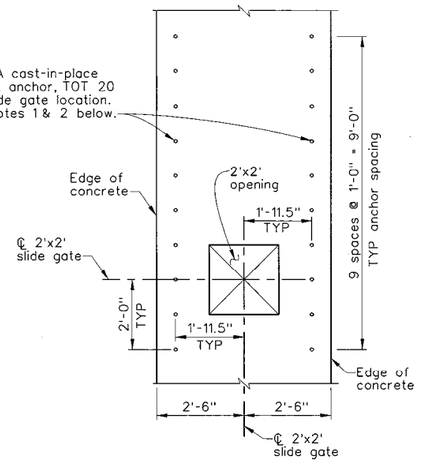
SLIDE GATE TRASHRACK COVER PLAN

Scale: 1" = 1'-0"



SECTION L3-L3

Scale: 1" = 1'-0"



TYPICAL SLIDE GATE TRASHRACK ANCHOR LAYOUT

Scale: ½" = 1'-0"

Notes: 1. See "Section C3-C3" on Sht. 5 for typical anchor embedment for slide gate on the sluiceway riser.

2. See "Section K2-K2" on Sht. 15 for TYP anchor embedment for slide gate on Riser #2.

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

DATE	MK	DESCRIPTION

REVISIONS

REGISTERED PROFESSIONAL ENGINEER
SCOTT D. GREGOWSKE
NO. 68087
EXP. 06/30/11
CIVIL
STATE OF CALIFORNIA

12/10/09
PROJECT ENGINEER DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

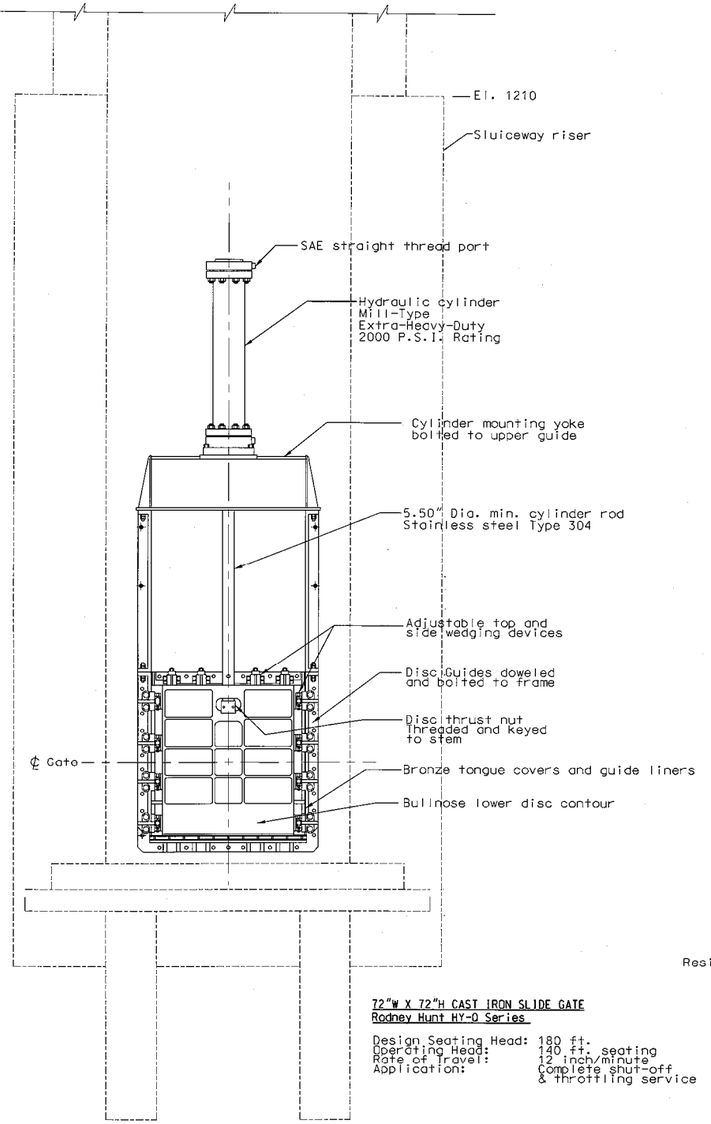
2' x 2' SLIDE GATE TRASHRACK DETAILS

PIN WRDD000016 | JOB X5009661 | DWG 55-D92.16 | SHEET 16 OF 21

PS-DSI 07/03

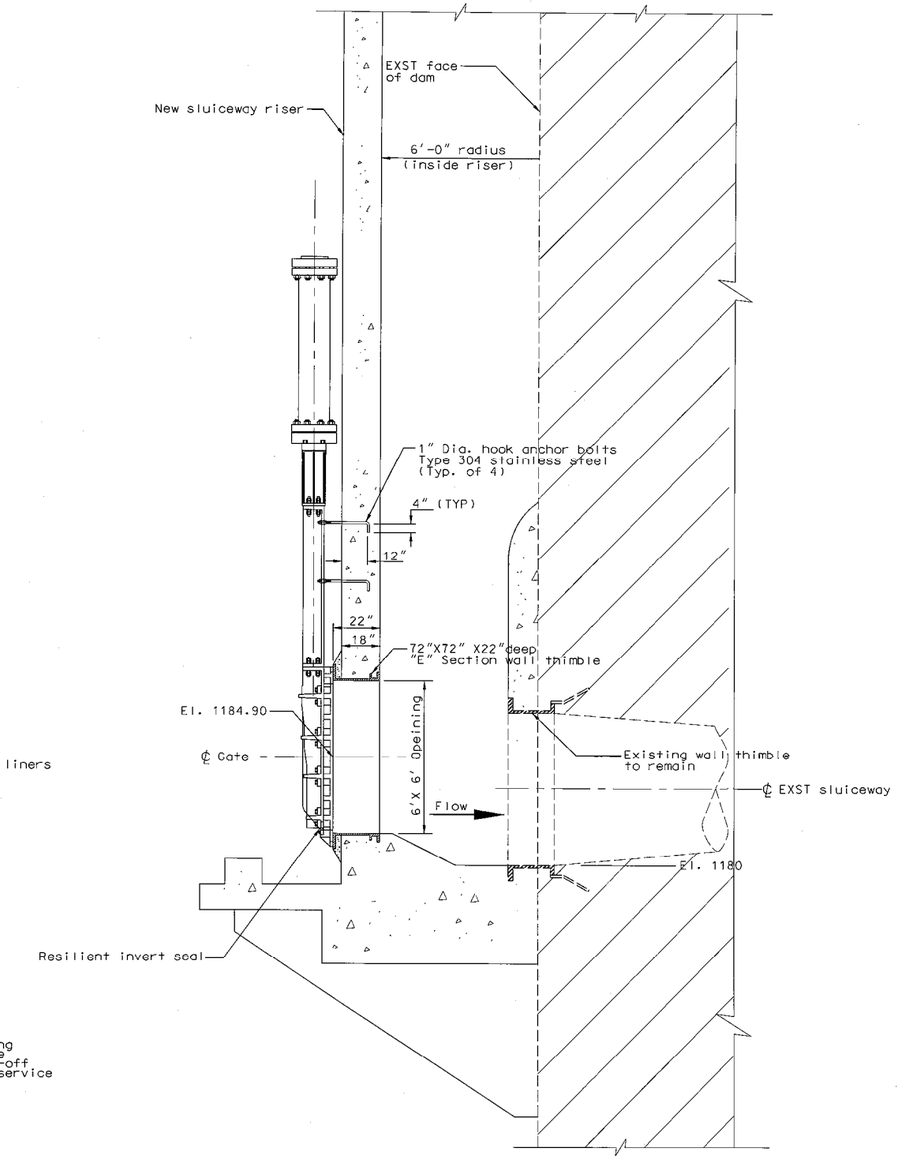
DRAFTER S. GREGOWSKE | DESIGNER S. GREGOWSKE | CHECKER KENNETH HO | CADD PROJECT FILE NAME WRDD000016-STRUCTURES.DGN | REVIEWED By KENNETH HO DATE 12/10/09

DATE: $DATE$ TIME: $TIME$ FILE: $FILE$



72"W X 72"H CAST IRON SLIDE GATE
Rodney Hunt HY-O Series

Design Seating Head: 180 ft.
Operating Head: 140 ft. seating
Rate of Travel: 12 inch/minute
Application: Complete shut-off & throttling service



72" X 72" CAST IRON SLIDE GATE (1 REQ'D)
FRONT ELEVATION AND SIDE SECTION VIEW
SCALE: 3/8" = 1'-0"

DRAFTER	DESIGNER	CHECKER	CADD PROJECT FILE NAME	REVIEWED BY	DATE
F. WONG	F. WONG	P. CHANG	WRD16.FLD.DES.MECH.M2		

DATE	MK	DESCRIPTION
		REVISIONS

REGISTERED PROFESSIONAL ENGINEER
PETER CHANG
No. M 29732
Exp. 6-30-2010
MECHANICAL
STATE OF CALIFORNIA

PROJECT ENGINEER — DATE 6-11-09

M-1

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

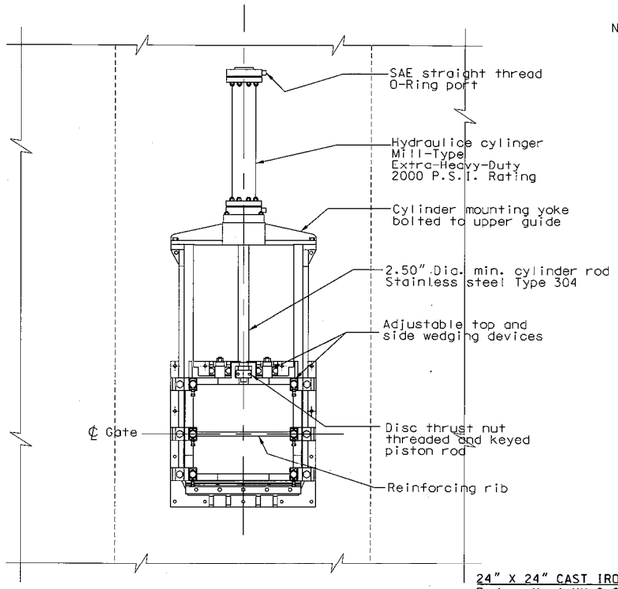
SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

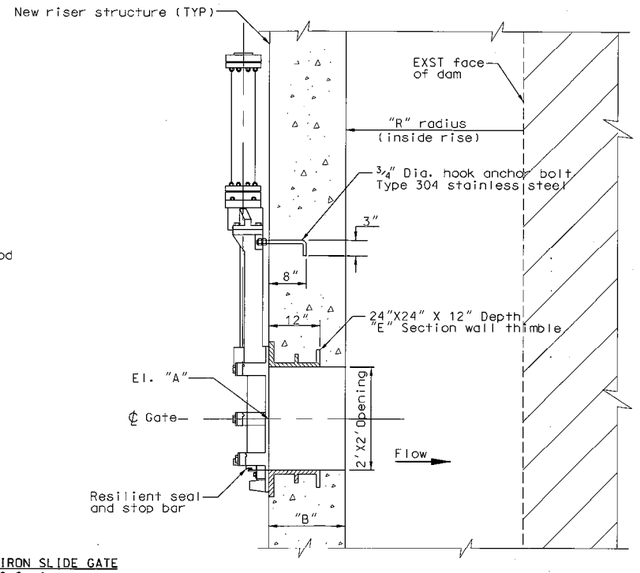
72"X72" SLUICEWAY SLIDE GATE
FRONT ELEVATION AND SIDE SECTION VIEW

WRD0000016	PCA X5009661	DWG 55-D92.17	SHEET 17 OF 21

PS-DF 07/05

DATE: $DATE$ TIME: $TIME$ FILE: $FILE$





LOCATION	El."A"	"B"	"R" (Riser Inside Radius)
Sluiceway Riser	1220.00	±18"	6'-0"
Riser #2	1200.00	±19 3/4"	3'-6"

24" X 24" CAST IRON SLIDE GATE
Rodney Hunt HY-O Series

Design Seating Head: 135 ft.
Operating Head: 120 ft. seating
Rate of Travel: 14 inch/minute
Application: Complete shut-off

24" X 24" CAST IRON SLIDE GATE (2 REQ'D)
FRONT ELEVATION AND SIDE SECTION VIEW
TYPICAL FOR SLUICEWAY RISER & RISER #2
SCALE: 3/4" = 1'-0"

DRAFTER F. WONG | DESIGNER F. WONG | CHECKER P. CHANG | CADD PROJECT FILE NAME WRD16.FLD.DES.MECH.M2 | REVIEWED BY | DATE

DATE	MK	DESCRIPTION
		REVISIONS

REGISTERED PROFESSIONAL ENGINEER — PETER CHANG — No. M 29732 — Exp 6-30-2010 — MECHANICAL — STATE OF CALIFORNIA
PROJECT ENGINEER — DATE 6-11-09

M-2

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

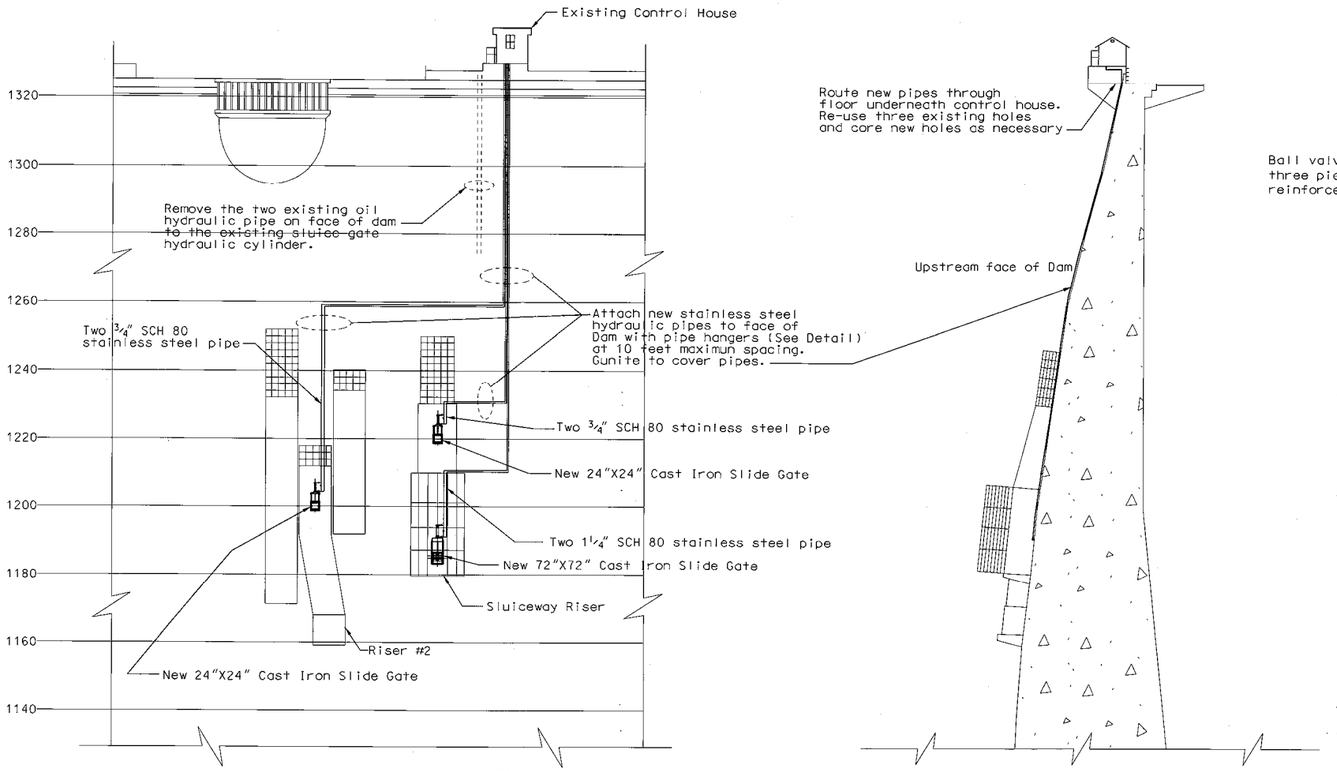
SANTA ANITA DAM AND RESERVOIR
RISER MODIFICATION

24"X24"SLIDE GATE
FRONT ELEVATION AND SIDE SECTION VIEW

WRD000016 | PCA X5009661 | DWG 55-D92.18 | SHEET 18 OF 21

PSLEF4765

DATE. $DATE$ TIME. $TIME$ FILE. $FILEL$

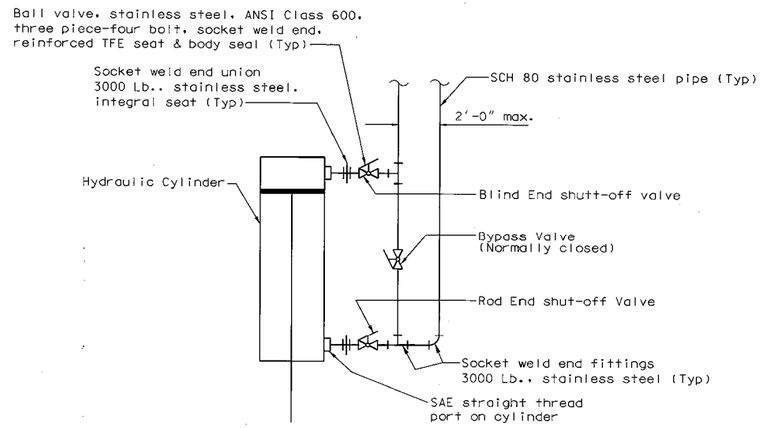


NOTE

1. All hydraulic line piping shall be Schedule 80 stainless Type 316, ASTM A312 Grade TP316,seamless,annealed.
2. Hydraulic line pipe fittings shall be 3000 LBS..socket weld end, stainless steel Type 316. ASTM A182.
3. Welding for stainless steel pipe joints and fittings shall be made by the gas tungsten-arc welding process performed by certified welder.

UPSTREAM FACE ELEVATION
HYDRAULIC OIL LINES
SCALE: 1" = 20'

UPSTREAM FACE ELEVATION
HYDRAULIC OIL LINES
SCALE: 1" = 20'



CYLINDER CONNECTION SCHEMATIC
SCALE: NONE

DRAFTER F. WONG
DESIGNER F. WONG
CHECKER P. CHANG
CADD PROJECT FILE NAME WRD16.FLD.DES.MECH.M3
REVIEWED BY
DATE

DATE	MK	DESCRIPTION
		REVISIONS

REGISTERED PROFESSIONAL ENGINEER — PETER CHANG — NO. M 29732 — EXP. 6-30-2010 — MECHANICAL — STATE OF CALIFORNIA
PROJECT ENGINEER — DATE 6-11-09

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA DAM AND RESERVOIR

RISER MODIFICATION

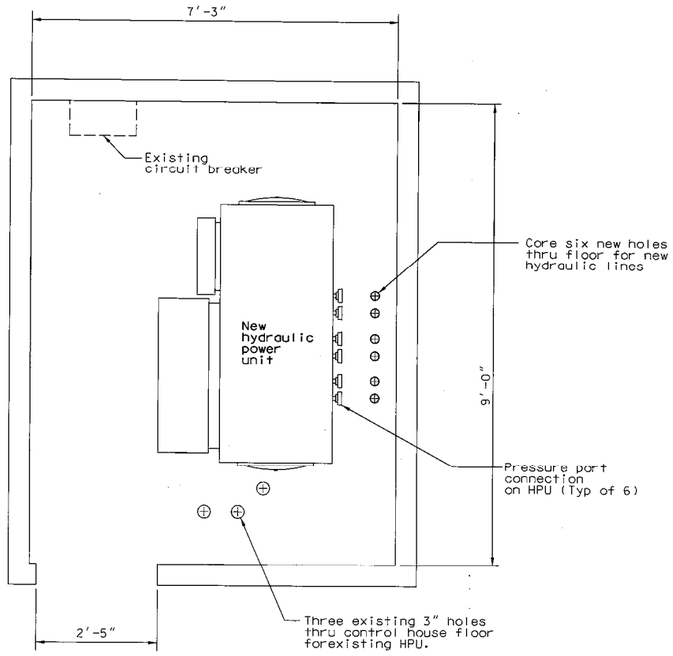
UPSTREAM ELEVATION AND SECTION
HYDRAULIC LINE LAYOUT

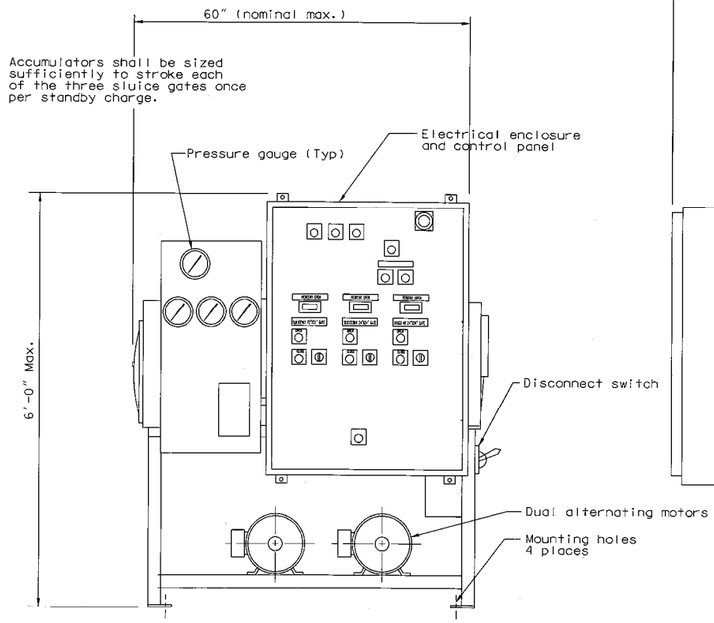
M-3

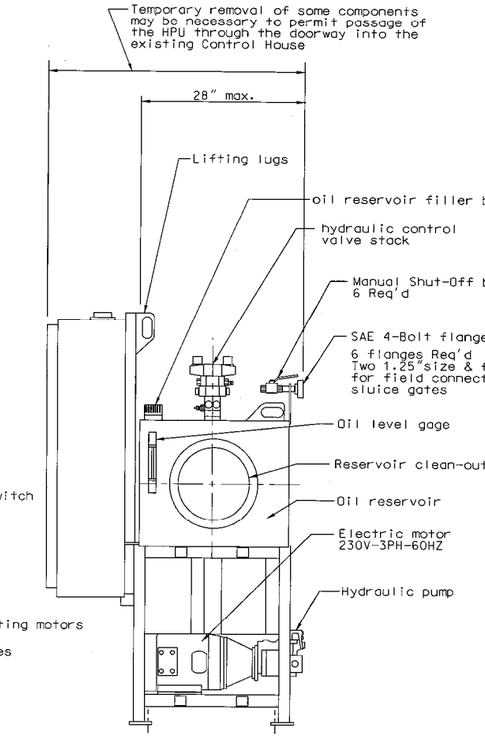
WRDD000016	PCA X5009661	DWG 55-D92.19	SHEET 19 OF 21

PS-DIF 07/05

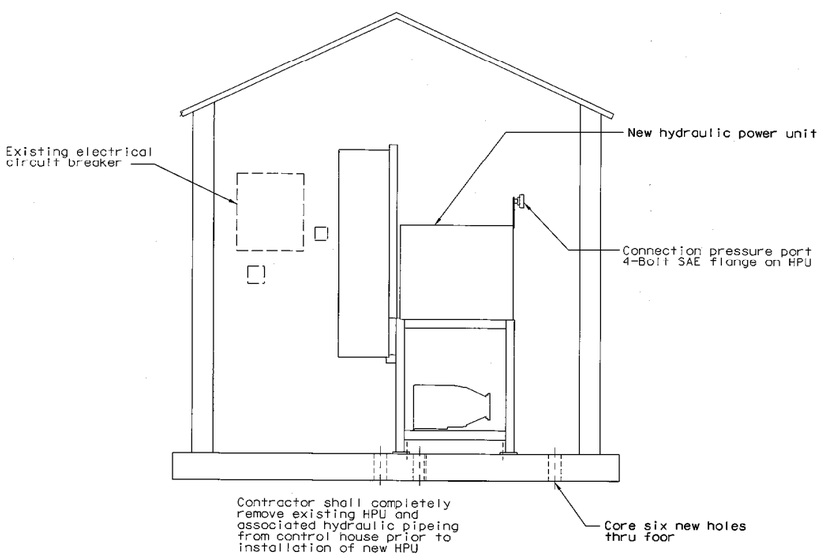
DATE: $DATE$ TIME: $TIME$ FILE: $FILE$







1. 2000 P.S.I. operating pressure
2. HPU shall operate one 72"x72" and two 24"x24" sluice gates at nominal speed of 12" to 14" per minute.
3. Gate position sensor shall be flow-meter based, installed in the hydraulic oil flow path between directional control valve and hydraulic cylinder.
4. Digital display shall show gate position in percentage open.
5. Gate positionn shall be translated via 4-20 mA or a voltage signal.
6. Hydraulic oil shall be Mobil DTE 24.
7. Panel mounted system pressure gauge shall be provided for system pressure and individual cylinder pressures.



EXISTING CONTROL HOUSE
SCALE: 3/4 " = 1'-0"

HYDRAULIC POWER UNIT (TYPICAL PROPOSED)
TO BE INSTALLED IN EXISTING CONTROL HOUSE
SCALE: NONE

DRAFTER	DESIGNER	CHECKER	CADD PROJECT FILE NAME	REVIEWED	DATE
F. WONG	F. WONG	P. CHANG	WRD16.FLD.DES.MECH.M4	BY	

DATE	MK	DESCRIPTION
		REVISIONS

REGISTERED PROFESSIONAL ENGINEER
PETER CHANG
NO. M 29732
EXP. 6-30-2010
MECHANICAL
STATE OF CALIFORNIA

PROJECT ENGINEER — DATE 6-11-09

M-4

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

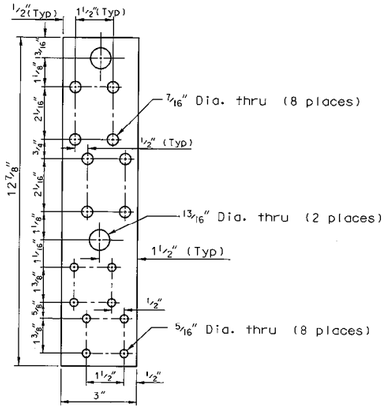
SANTA ANITA DAM AND RESERVOIR

RISER MODIFICATION

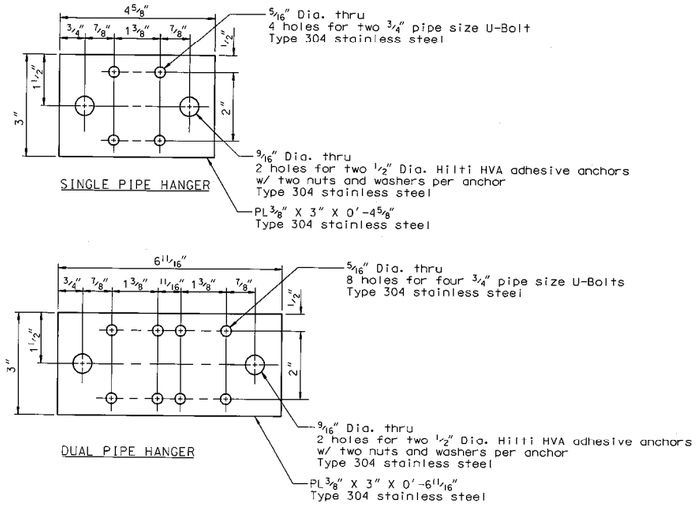
EXISTING CONTROL HOUSE AND NEW HYDRAULIC POWER UNIT

WRDD000016	PCA X5009661	DWG 55-D92.20	SHEET 20 OF 21

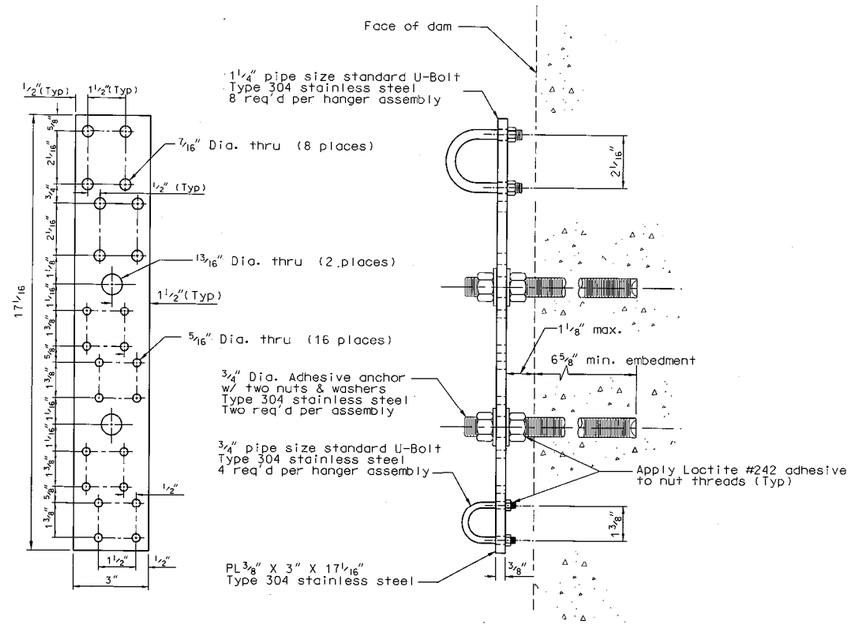
DATE: $DATE$ TIME: $TIME$ FILE: $FILE$



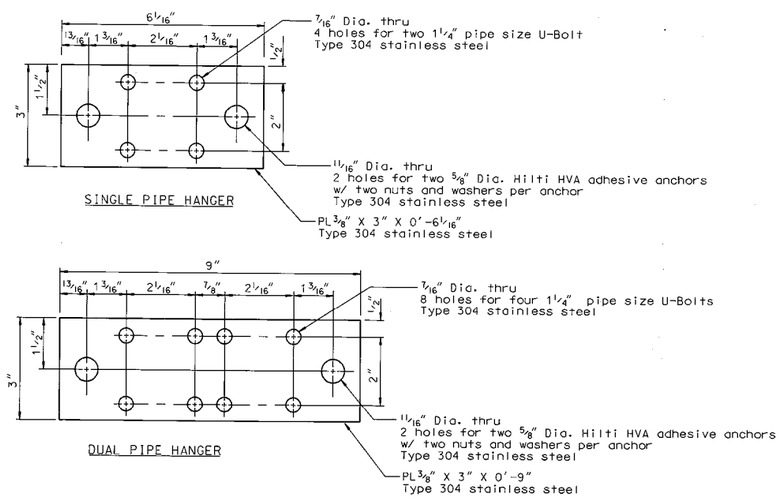
PIPE HANGER - 1 1/4" AND 3/4" COMBO
SCALE: 3/8" = 1"



PIPE HANGER - 3/4" PIPE SIZE
SCALE: HALF SIZE



GANG PIPE HANGER ASSEMBLY
SCALE: 3/8" = 1"



PIPE HANGER - 1 1/4" PIPE SIZE
SCALE: HALF SIZE

DRAFTER	DESIGNER	CHECKER	CADD PROJECT FILE NAME	REVIEWED	DATE
F. WONG	F. WONG	P. CHANG	WRD16-FLD.DES.MECH.M5	BY	

DATE	MK	DESCRIPTION	
REVISIONS			

REGISTERED PROFESSIONAL ENGINEER
PETER CHANG
NO. M 29732
EXP. 6-30-2010
MECHANICAL
STATE OF CALIFORNIA
PROJECT ENGINEER — DATE

M-5

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA DAM AND RESERVOIR

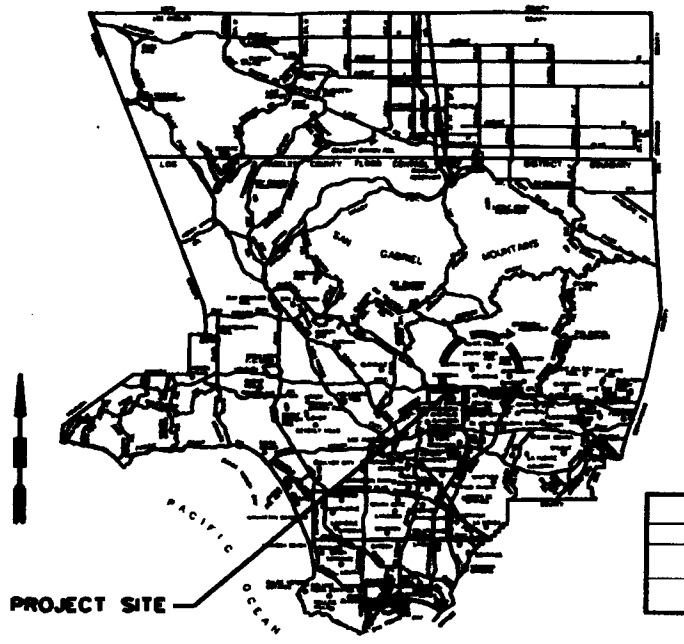
RISER MODIFICATION

HYDRAULIC LINE - PIPE HANGER DETAIL

WRD0000016	PCA X5009661	DWG 55-D92.21	SHEET 21 OF 21

DATE: $DATE$ TIME: $TIME$ FILE: $FILEL$

COUNTY OF LOS ANGELES
DEPARTMENT OF PUBLIC WORKS
SANTA ANITA SPREADING GROUNDS
MAINTENANCE PROJECT



Thomas Bros. Pgs. 567 D1

LOCATION MAP

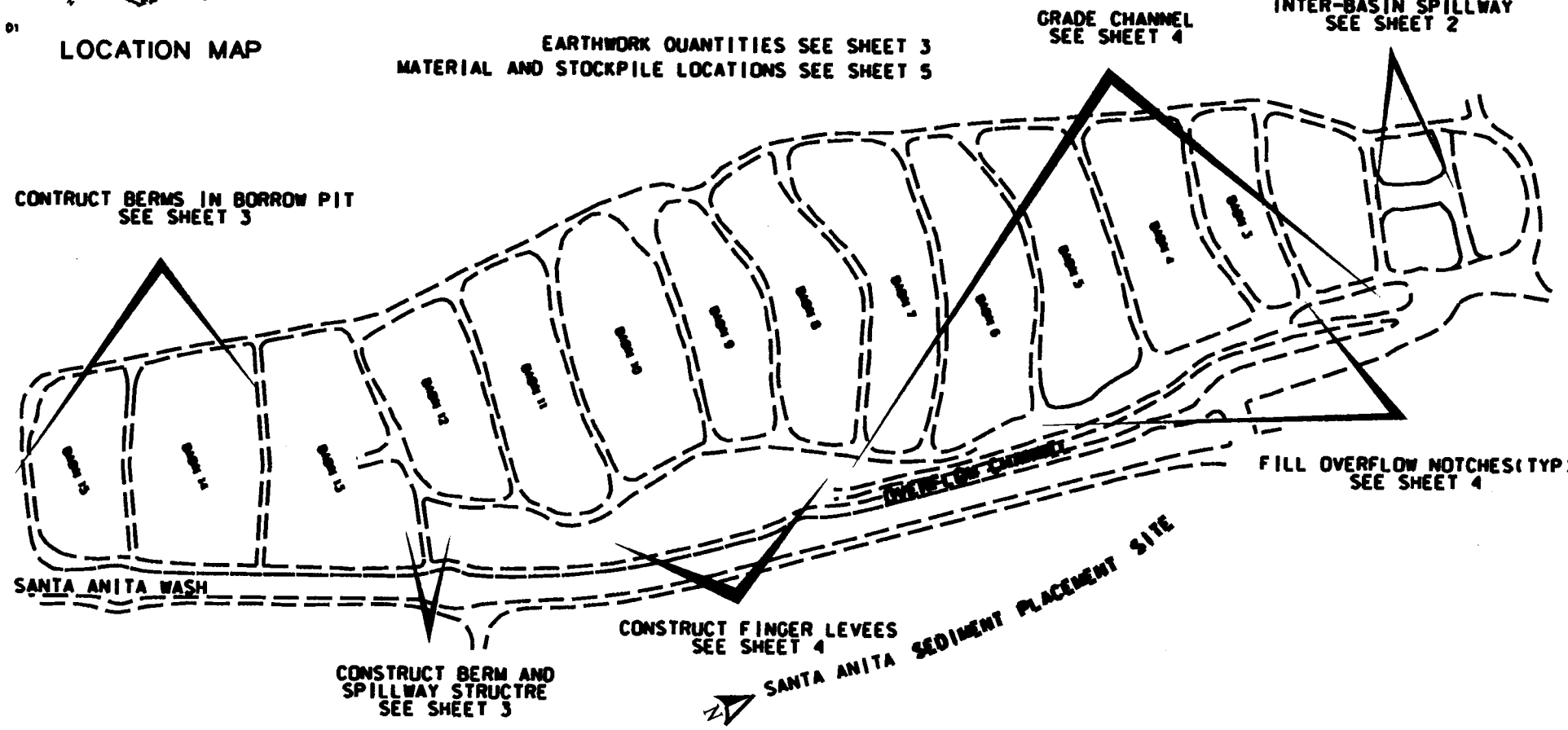
BASIN VOLUMES																
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	TOTAL
EXISTING (Acre-Feet)	1.23	2.76	2.20	2.66	2.46	2.96	4.80	2.66	2.66	5.60	5.11	3.11	N/A	N/A	N/A	38.22
PROPOSED (Acre-Feet)	0.56	2.76	2.20	2.66	4.25	2.96	4.80	3.55	3.30	6.54	5.84	5.59	6.25	5.45	4.32	61.11

REFERENCES

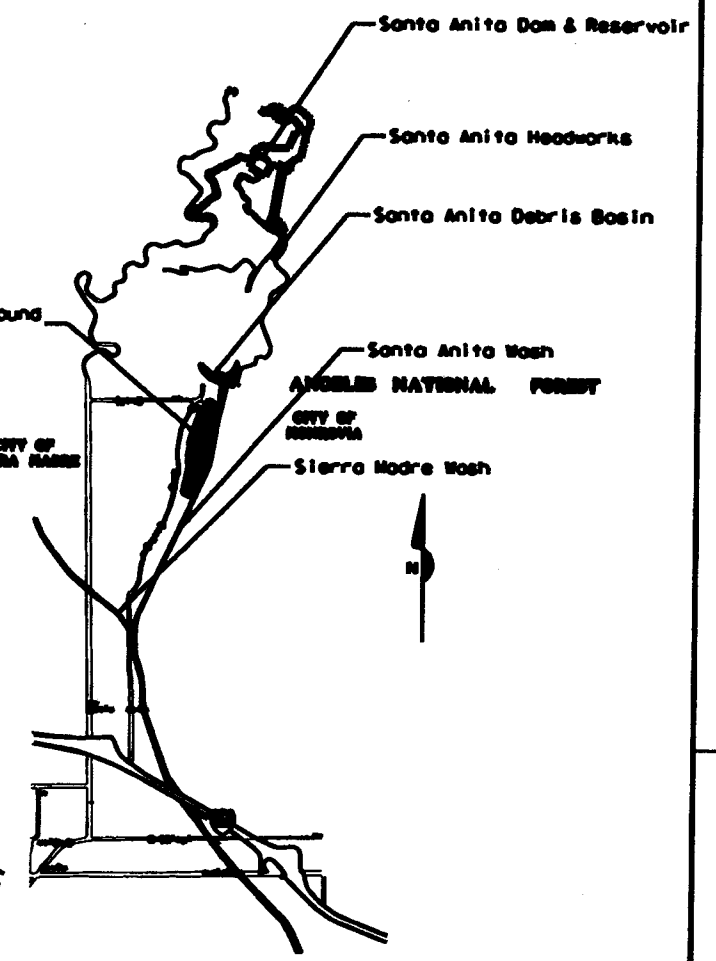
SantaAnitaSG-SOB-36-T31_July2007

Santa Anita Wash Spreading Grounds
General Plan : January 1951
Drawing No. : 36-021

EARTHWORK QUANTITIES SEE SHEET 3
MATERIAL AND STOCKPILE LOCATIONS SEE SHEET 5



SANTA ANITA SPREADING GROUNDS
PLAN VIEW
NTS



SANTA ANITA WASH SYSTEM
NOT TO SCALE

CONSTRUCTION NOTES:

1. Remove rocks and boulders (6" and above) from Basins 1 through 12 to a depth of 3 feet.
2. Use the existing rocks up to 12 inches diameter as rip rap for the spillway and weir structures.
3. Use borrow material from Santa Anita SPS for earthwork required to fill the overflow notches in basins 1 through 12.
4. All work shall be done to the satisfaction of DPW.

DATE	MK	DESCRIPTION

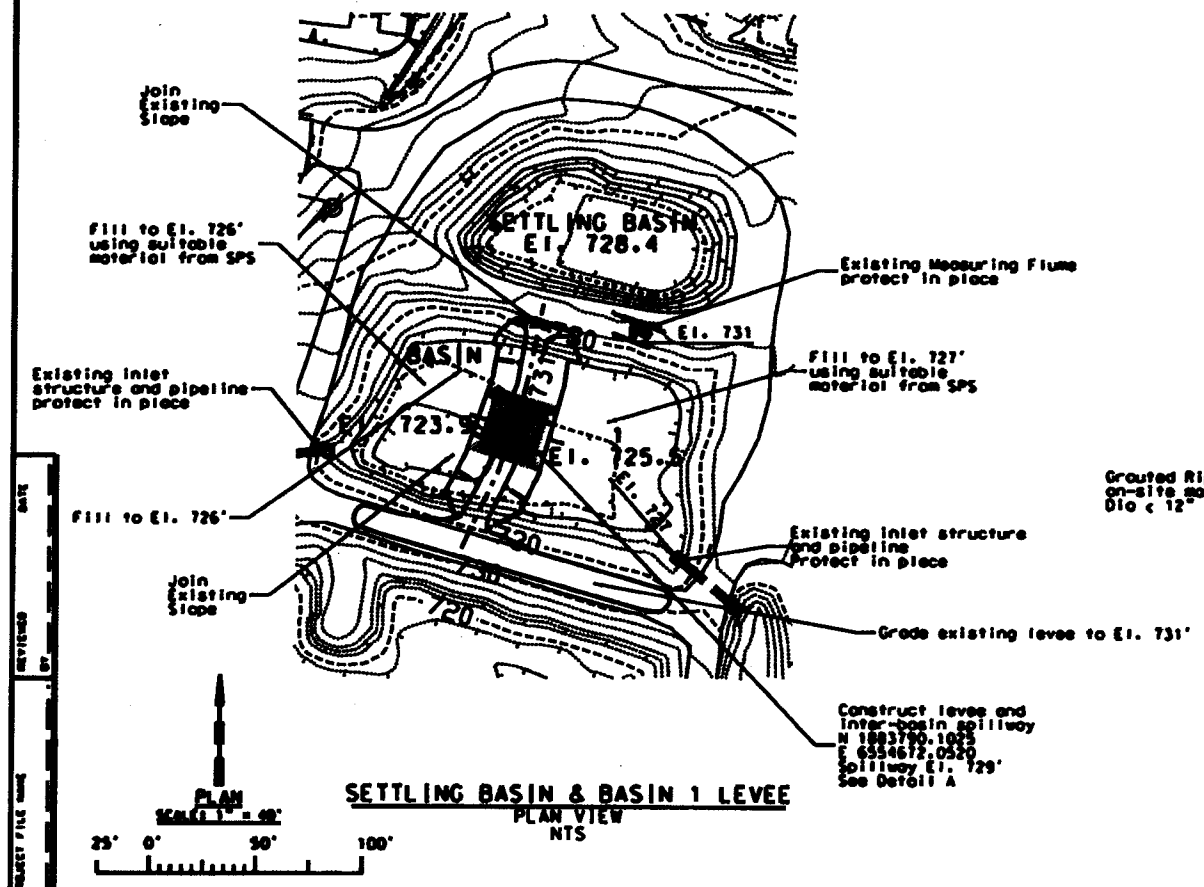
REVISIONS

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

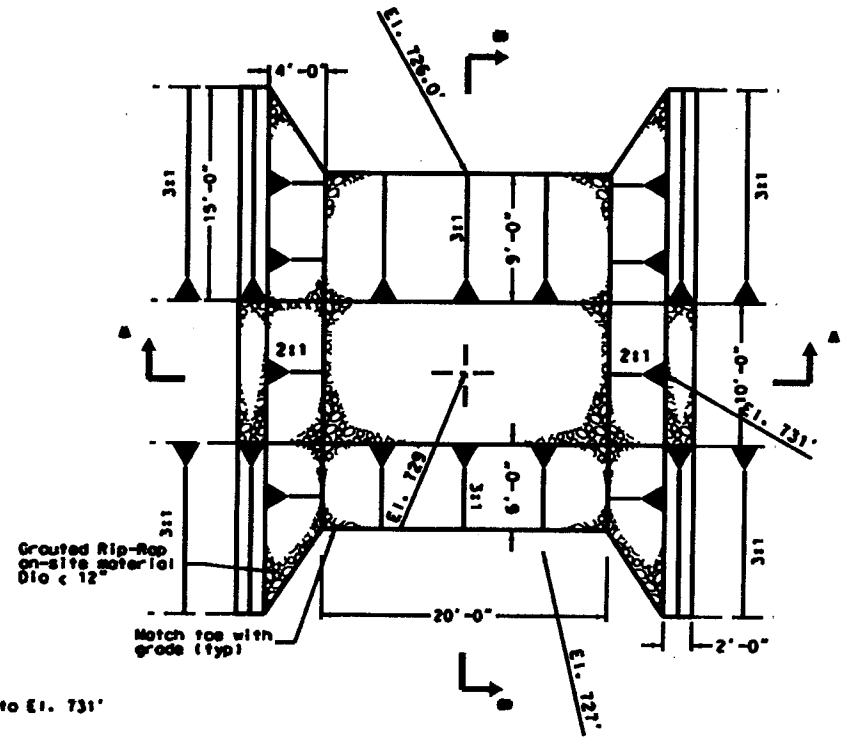
SANTA ANITA SPREADING GROUNDS
MAINTENANCE PROJECT

TITLE SHEET

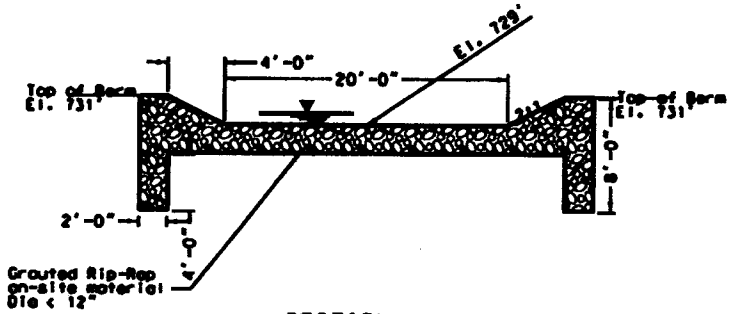
6/29/2009 | JOB H0321857 | DWG 36D-55.1 | SHEET 1 OF 4



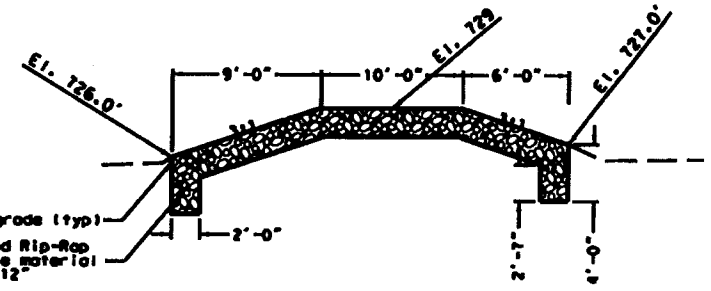
SETTLING BASIN & BASIN 1 LEVEE
PLAN VIEW
NTS



DETAIL A
INTERBASIN SPILLWAY STRUCTURE
NTS



SECTION A-A
NTS



SECTION B-B
NTS

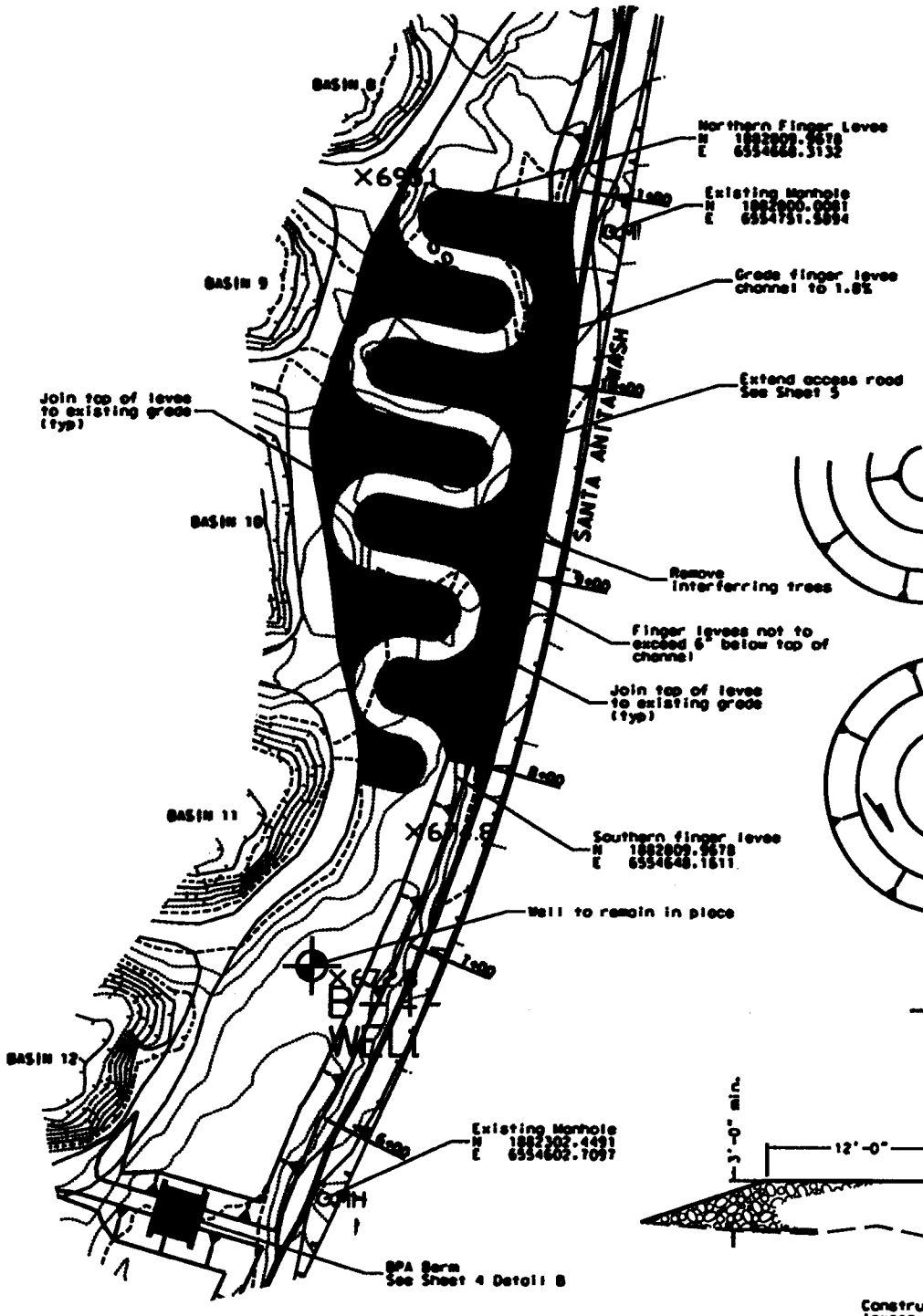
DATE	BY	DESCRIPTION
		REVISIONS

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA SPREADING GROUNDS
MAINTENANCE PROJECT

SETTLING BASIN & BASIN 1 DETAIL

6/29/2009	JOB H03218B7	DWG 360-55.2	SHEET 2 OF 4

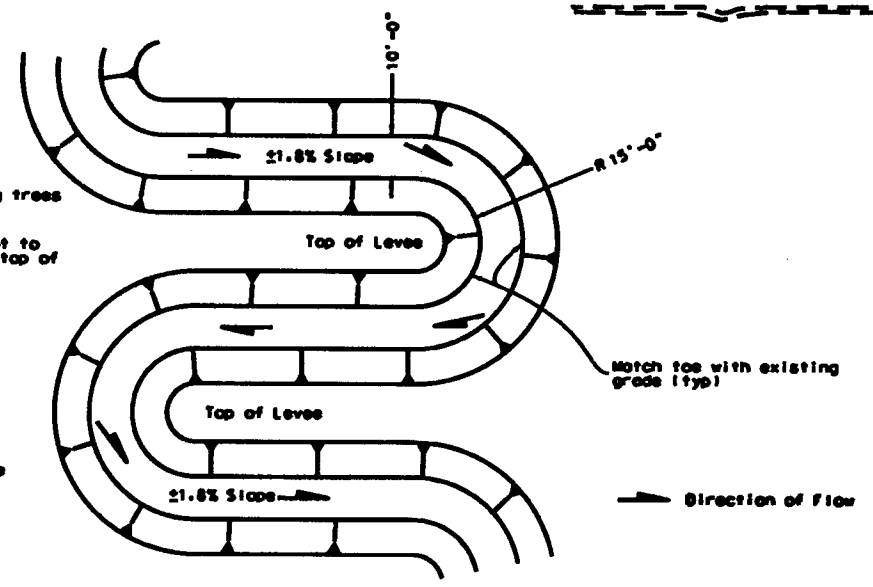


Protect trees that do not obstruct construction

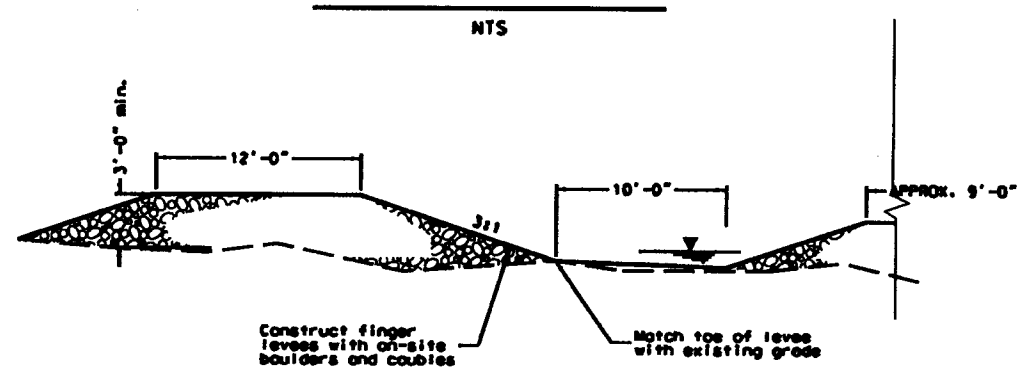
PLAN
SCALE: 1" = 40'

25' 0' 50' 100' 150'

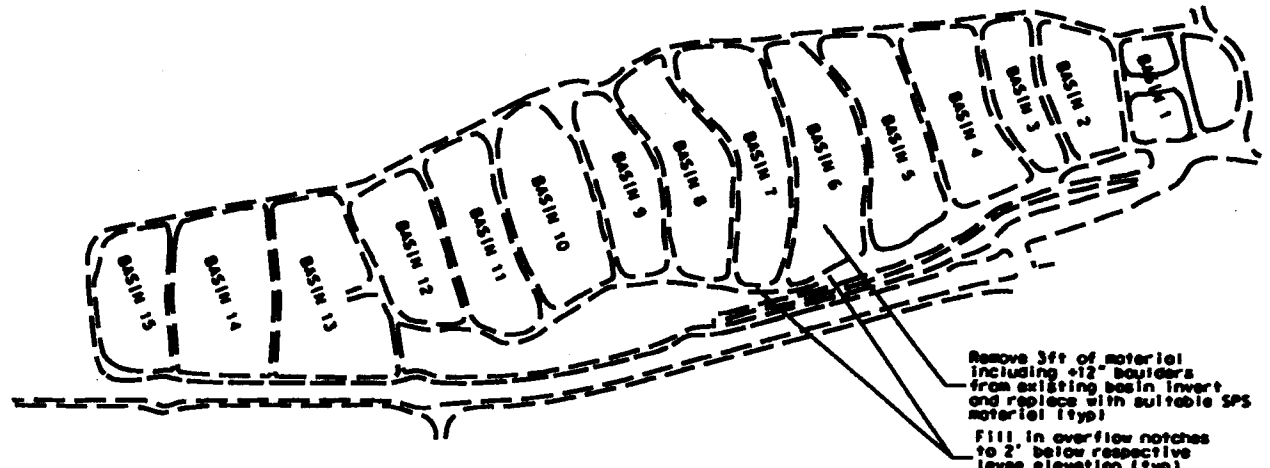
FINGER LEVEE
PLAN VIEW



TYPICAL FINGER LEVEE
NTS



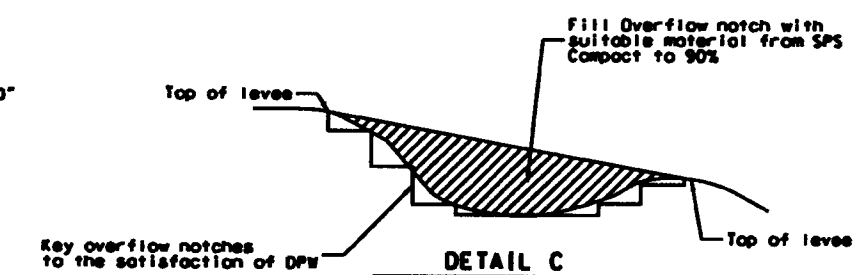
TYPICAL PROFILE OF FINGER LEVEE
NTS



PLAN VIEW OF BASIN CLEANOUT
NTS

APPROXIMATE EARTH WORK QUANTITIES

	FILL	CUT	GROUTED RIP-RAP
FINGER LEVEES	2.500 CY	1.000 CY	
BASIN 1 BERM	300 CY		200 CY
BPA BERM	800 CY		300
BASIN 2	3.480 CY	2.670 CY	
BASIN 3	2.130 CY	2.130 CY	
BASIN 4	5.460 CY	4.200 CY	
BASIN 5	5.630 CY	4.330 CY	
BASIN 6	6.210 CY	4.780 CY	
BASIN 7	6.000 CY	4.620 CY	
BASIN 8	5.580 CY	4.290 CY	
BASIN 9	4.030 CY	3.100 CY	
BASIN 10	5.850 CY	4.500 CY	
BASIN 11	4.615 CY	3.550 CY	
BASIN 12	3.900 CY	3.000 CY	
BASIN 13 BERM	2.250 CY		400 CY
BASIN 14 BERM	1.500 CY		400 CY
BASIN 15 BERM	1.500		
TOTAL	61.735 CY	43.170 CY	1.300 CY



DETAIL C
TYPICAL OVERFLOW NOTCH CROSS-SECTION
NTS

DATE	NO.	DESCRIPTION

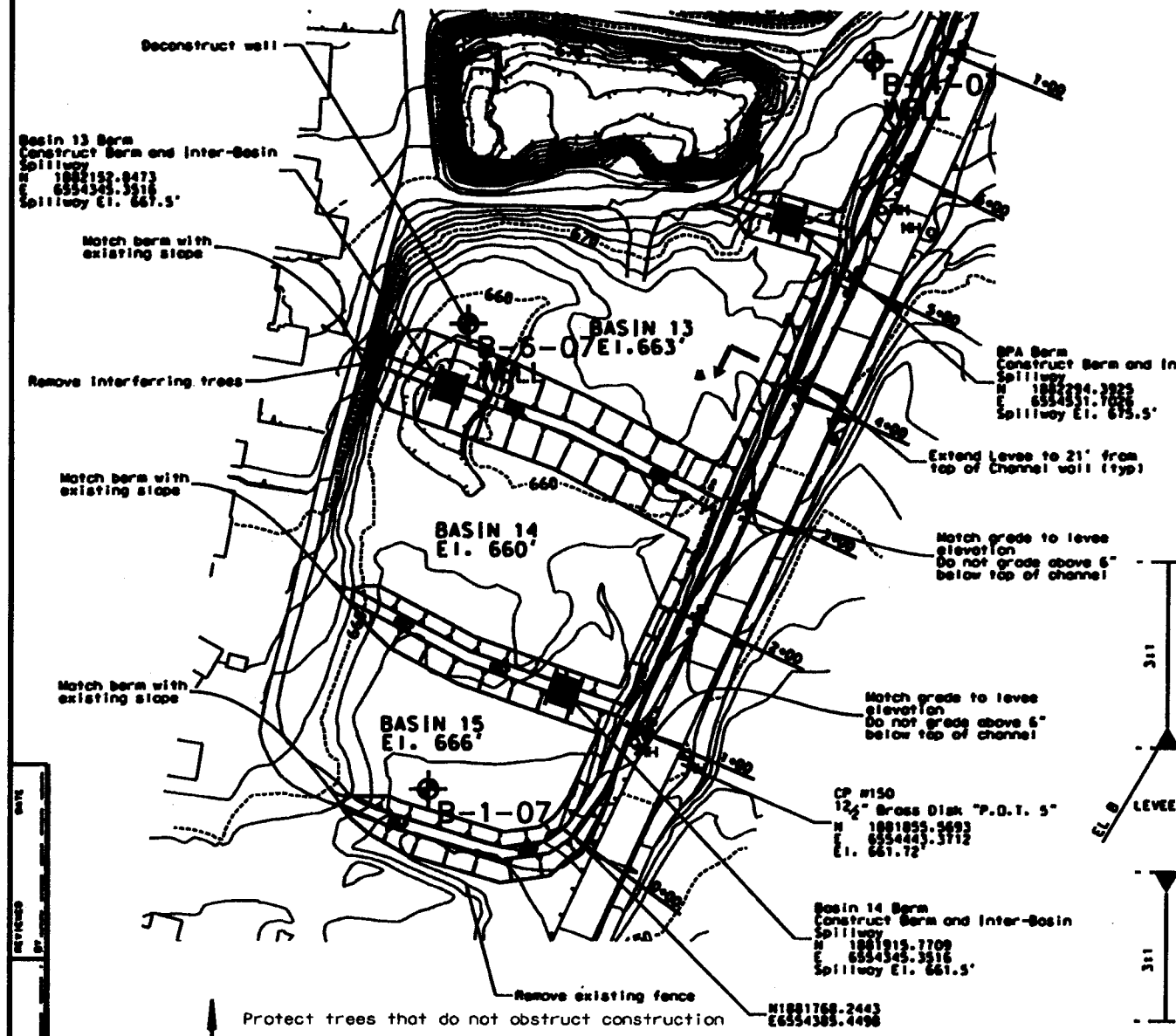
REVISIONS

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA SPREADING GROUNDS
MAINTENANCE PROJECT

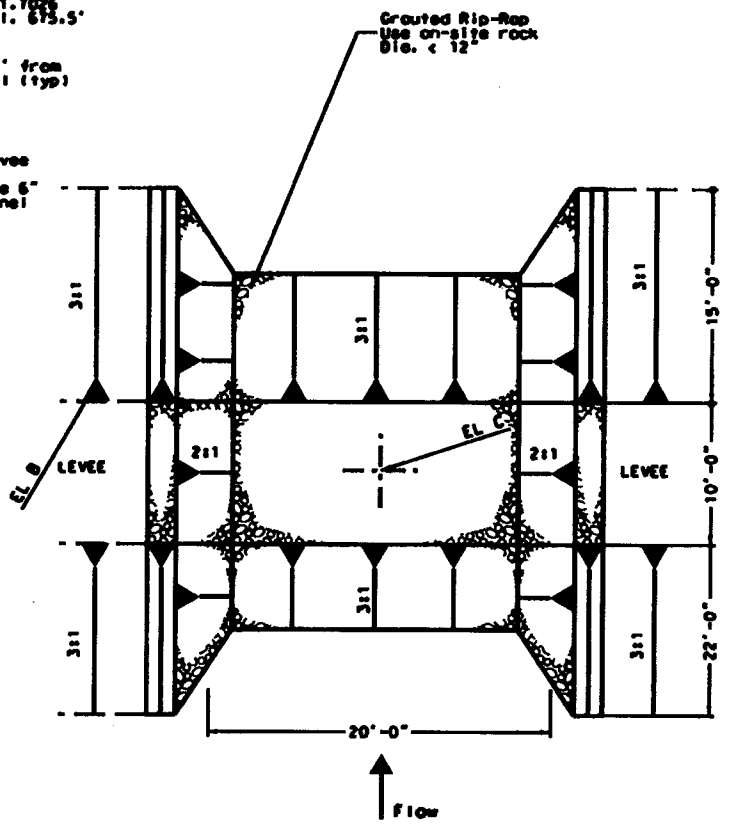
FINGER LEVEE AND BASIN CLEANOUT

6/29/2009 | JOB H0321857 | DWG 360-55.3 | SHEET 3 OF 4

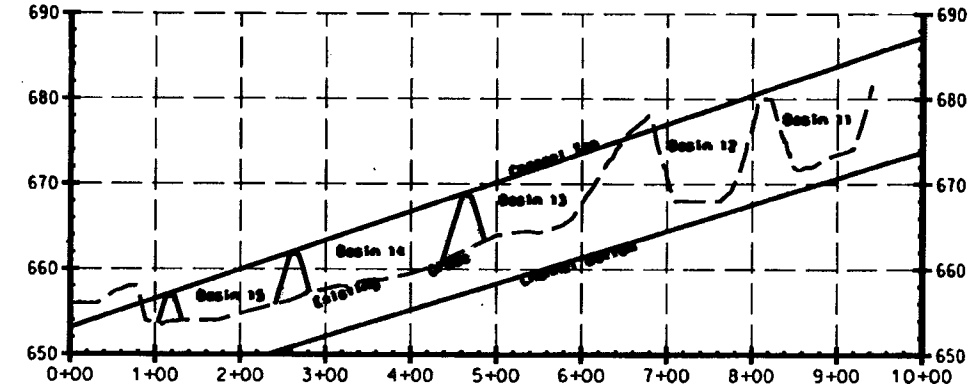


BORROW PIT AREA BERM
PLAN VIEW

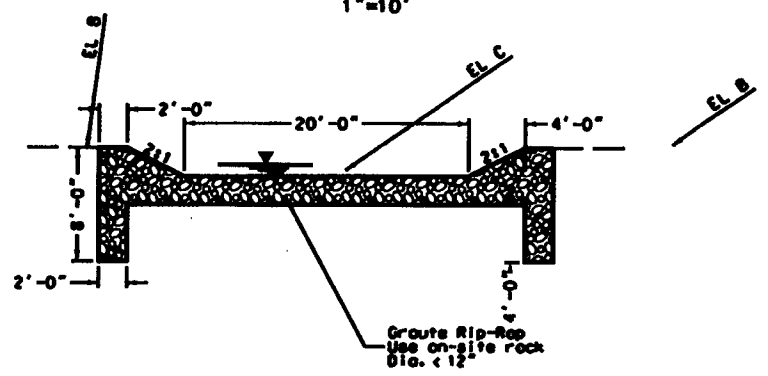
PLAN
SCALE: 1" = 50'
25' 0' 50' 100' 200'

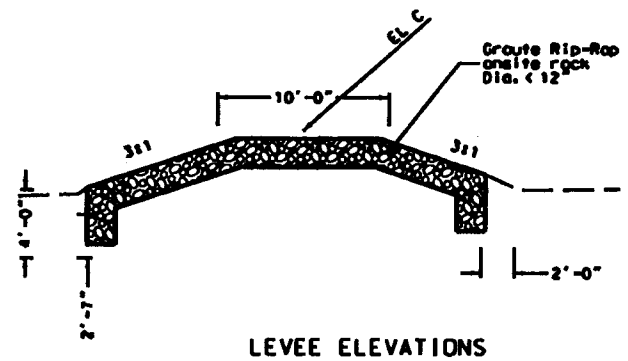


DETAIL B
INTERBASIN SPILLWAY STRUCTURES
NTS



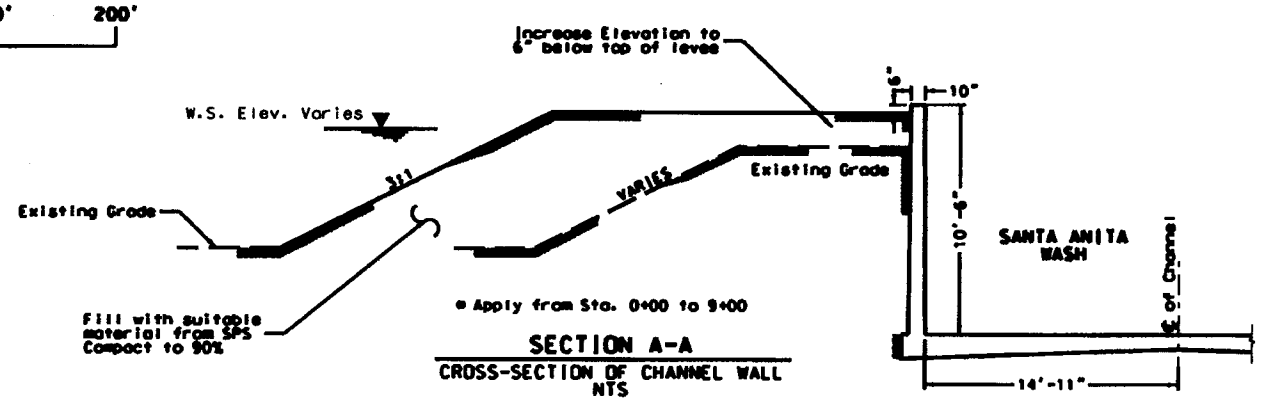
BORROW PIT AREA PROFILE
SCALE: 1"=100'
1"=10'





LEVEE ELEVATIONS

Levee	Top Levee Elevation B	Spillway Elevation C	W.S. Elevation
BPA	677.0	675.5	676.0
13	669.0	667.5	668.0
14	663.0	661.5	662.0
15	657.0	No Spillway	



* Apply from Sta. 0+00 to 9+00

SECTION A-A
CROSS-SECTION OF CHANNEL WALL
NTS

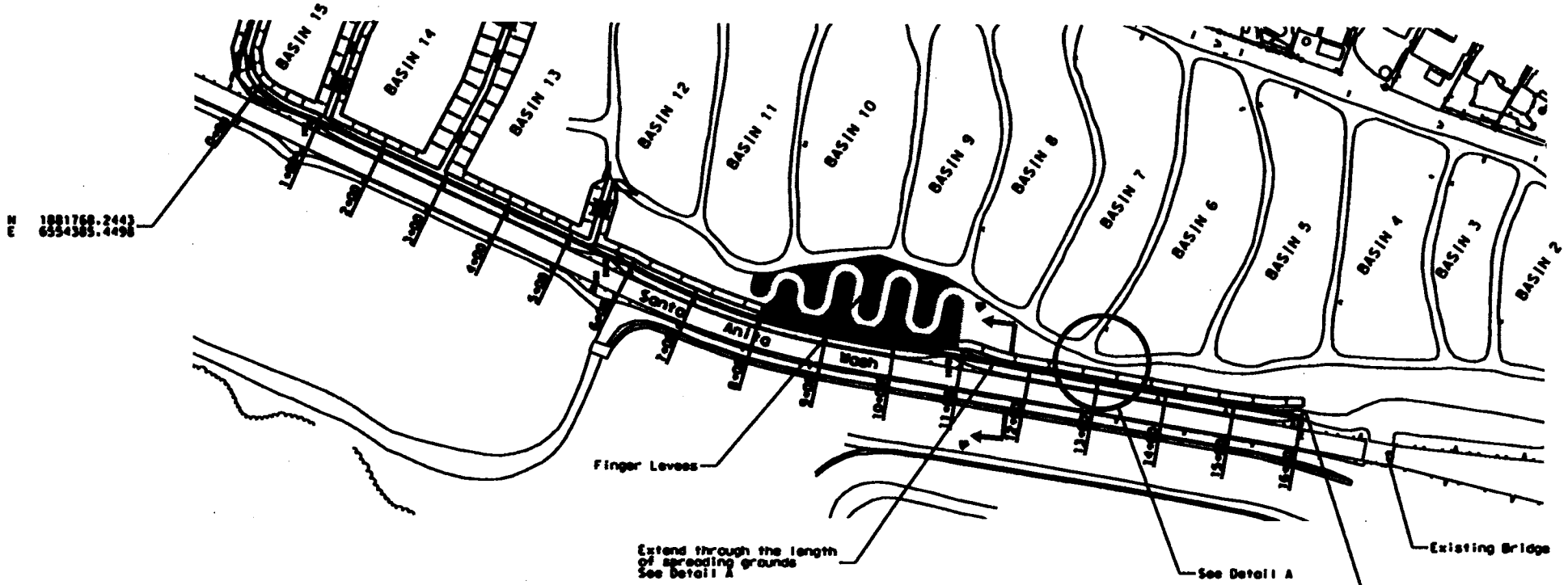
DATE	BY	DESCRIPTION
		REVISIONS

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA SPREADING GROUNDS
MAINTENANCE PROJECT

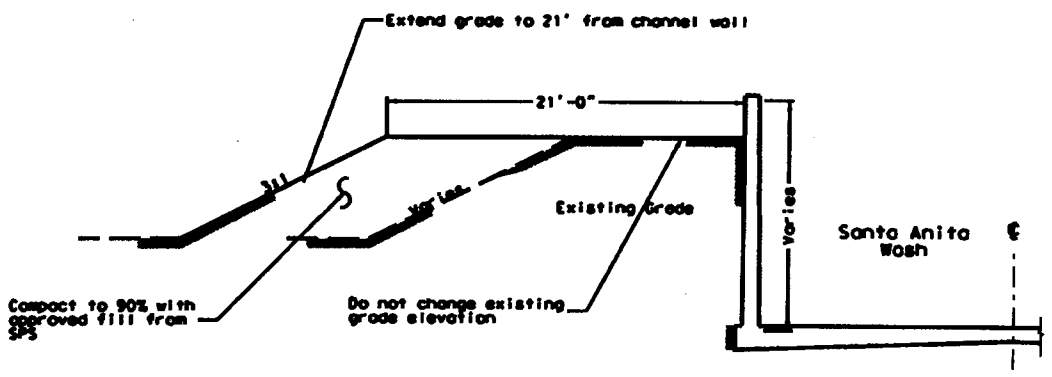
BORROW PIT AREA DETAIL

6/29/2009	JOB H0321857	DWG 360-33.4	SHEET 4 OF 5



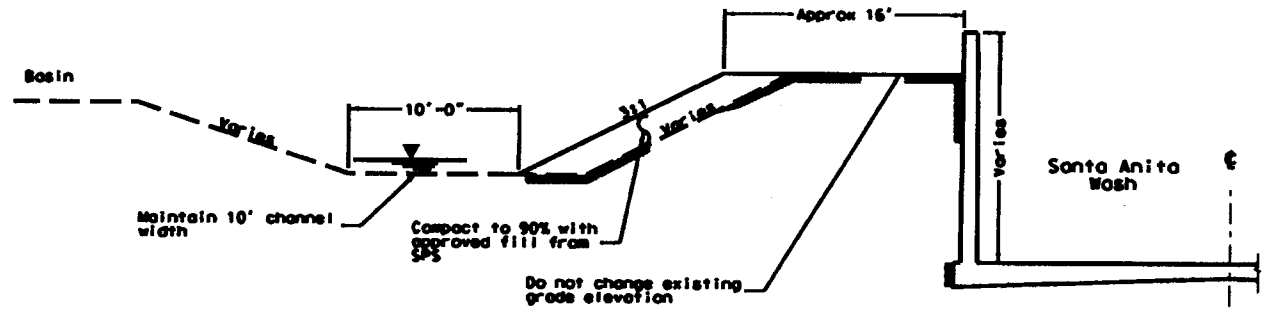
PLAN
SCALE: 1" = 100'
50' 0' 100' 200'

ACCESS ROAD GRADING
PLAN VIEW
NTS



• Apply to Sta. 8+00 to 11+00

SECTION A-A
CROSS-SECTION OF CHANNEL WALL
NTS



• Apply to Sta. 11+00 to Existing Bridge

DETAIL A
CROSS-SECTION OF CHANNEL WALL
NTS

DATE	BY	DESCRIPTION
		REVISIONS

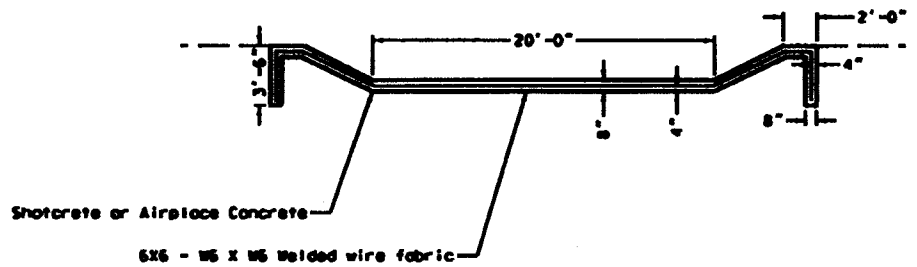
CIVIL

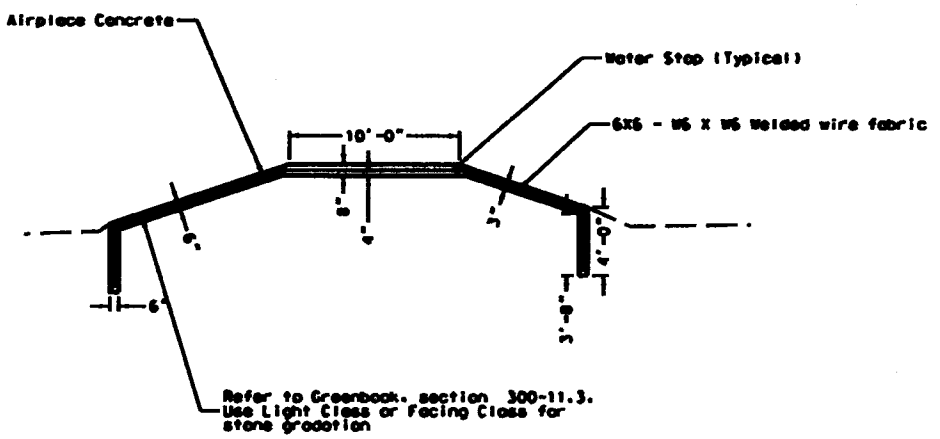
COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

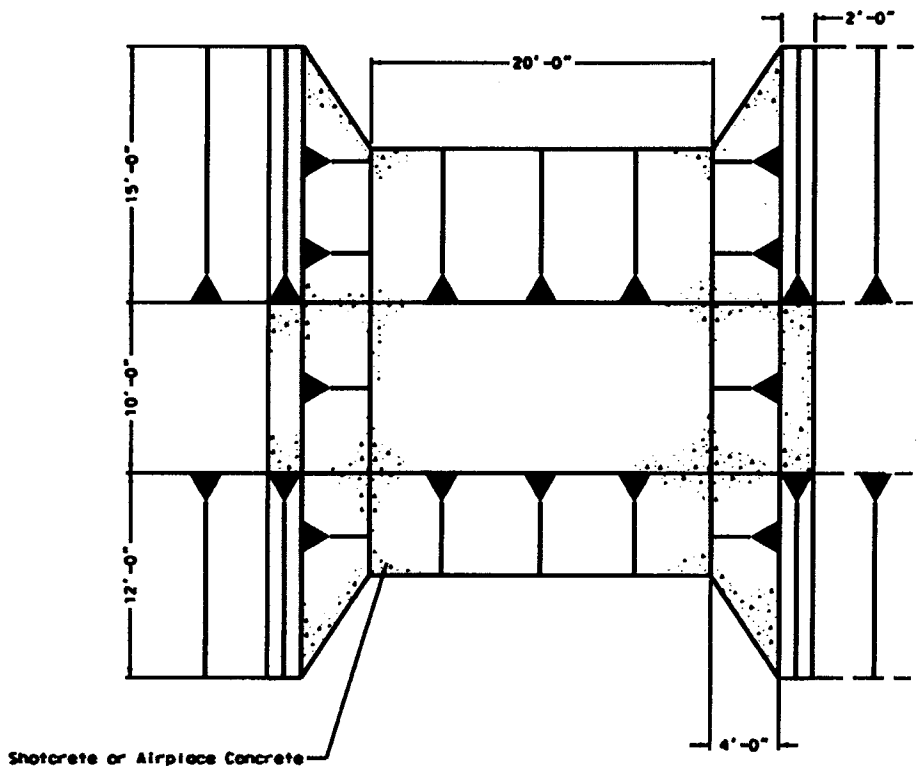
SANTA ANITA SPREADING GROUNDS
MAINTENANCE PROJECT

ACCESS ROAD EXTENSION

6/29/2009 | JOB NO321057 | DWG 360-55.5 | SHEET 5 OF 5







DATE	BY	DESCRIPTION
		REVISIONS

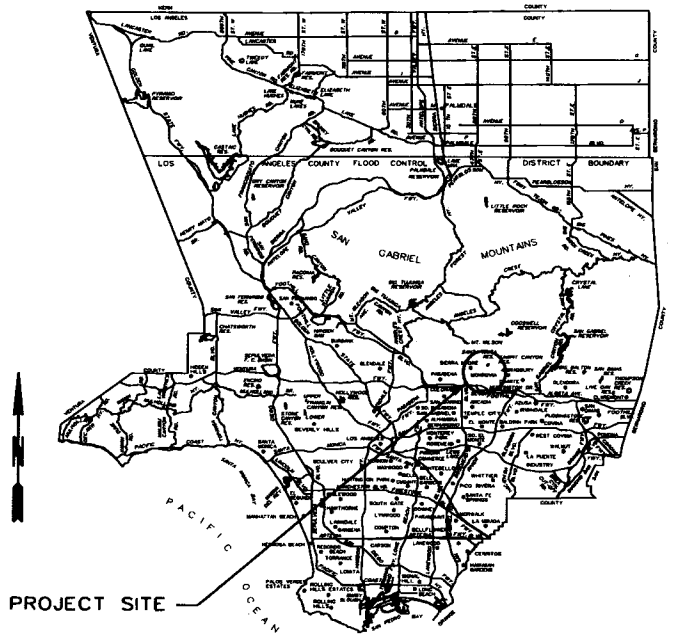
COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA SPREADING GROUNDS
MAINTENANCE PROJECT

ALTERNATIVE 2 SPILLWAY DESIGN

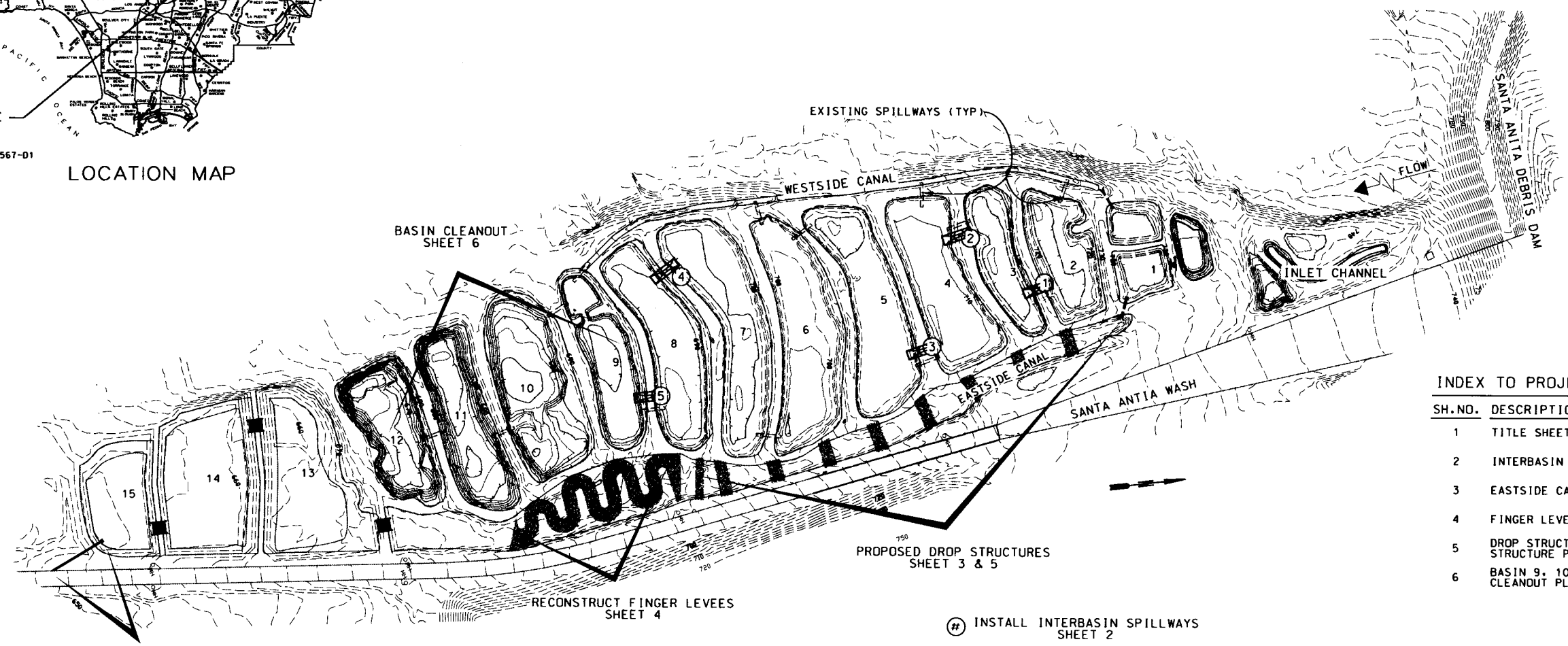
6/29/2009	JOB NO321057	DWG 360-55.6	SHEET 6 OF 6

COUNTY OF LOS ANGELES
DEPARTMENT OF PUBLIC WORKS
SANTA ANITA SPREADING GROUNDS
IMPROVEMENT PROJECT



Thomas Bros. Pgs. 567-D1

LOCATION MAP



PLAN VIEW
SANTA ANITA SPREADING GROUNDS
NOT TO SCALE

INDEX TO PROJECT PLANS

SH.NO.	DESCRIPTION
1	TITLE SHEET
2	INTERBASIN SPILLWAYS PLAN AND DETAILS
3	EASTSIDE CANAL PLAN AND PROFILE
4	FINGER LEVEE PLAN AND DETAIL
5	DROP STRUCTURE, BASIN 15 OVERFLOW STRUCTURE PLAN AND DETAILS
6	BASIN 9, 10, 11, AND 12 CLEANOUT PLAN AND DETAILS

DRAFTER H. TREMMEL | DESIGNER H. TREMMEL | CHECKER | CADD PROJECT FILE NAME

DATE	MK	DESCRIPTION

REVISIONS

REGISTERED PROFESSIONAL ENGINEER — K. ZIMMER — NO. — EXP. — CIVIL — STATE OF CALIFORNIA

PROJECT ENGINEER — DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA SPREADING GROUNDS
IMPROVEMENT PROJECT

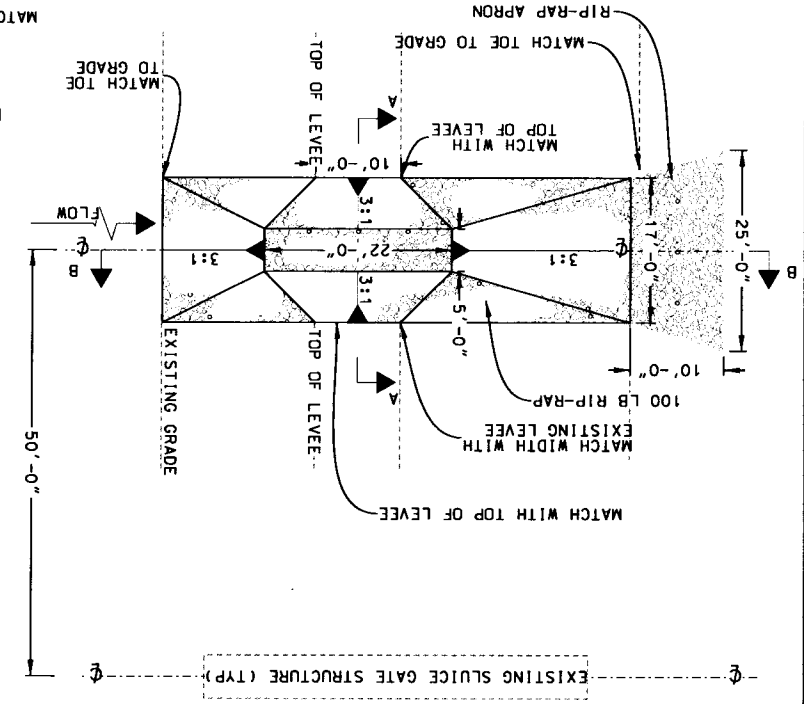
TITLE SHEET

JOB H0321857 | DWG | SHEET 1 OF 6

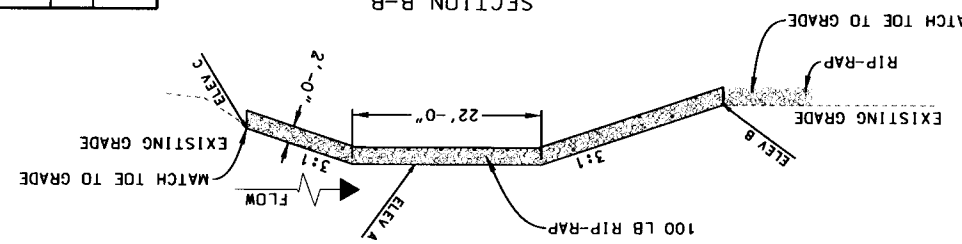
DATE- $DATE$ TIME- $TIME$ FILE- $FILE$

DRAWN BY: H. TREMMEL | DESIGNED BY: H. TREMMEL | CHECKED | CADD PROJECT FILE NAME | REVIEWED BY | DATE

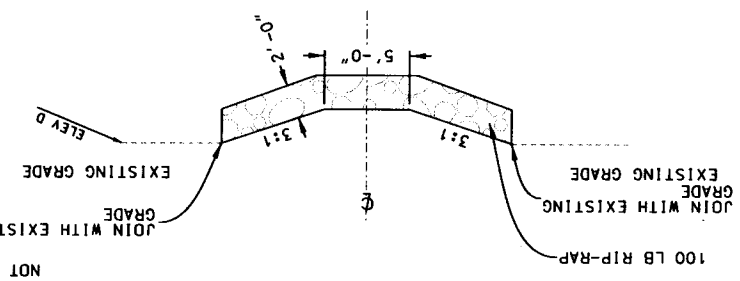
DETAIL A
INTERBASIN SPILLWAY STRUCTURES (TYP)
NOT TO SCALE



SECTION B-B
INTERBASIN SPILLWAY STRUCTURES (TYP)
NOT TO SCALE



SECTION A-A
INTERBASIN SPILLWAY STRUCTURES (TYP)
NOT TO SCALE

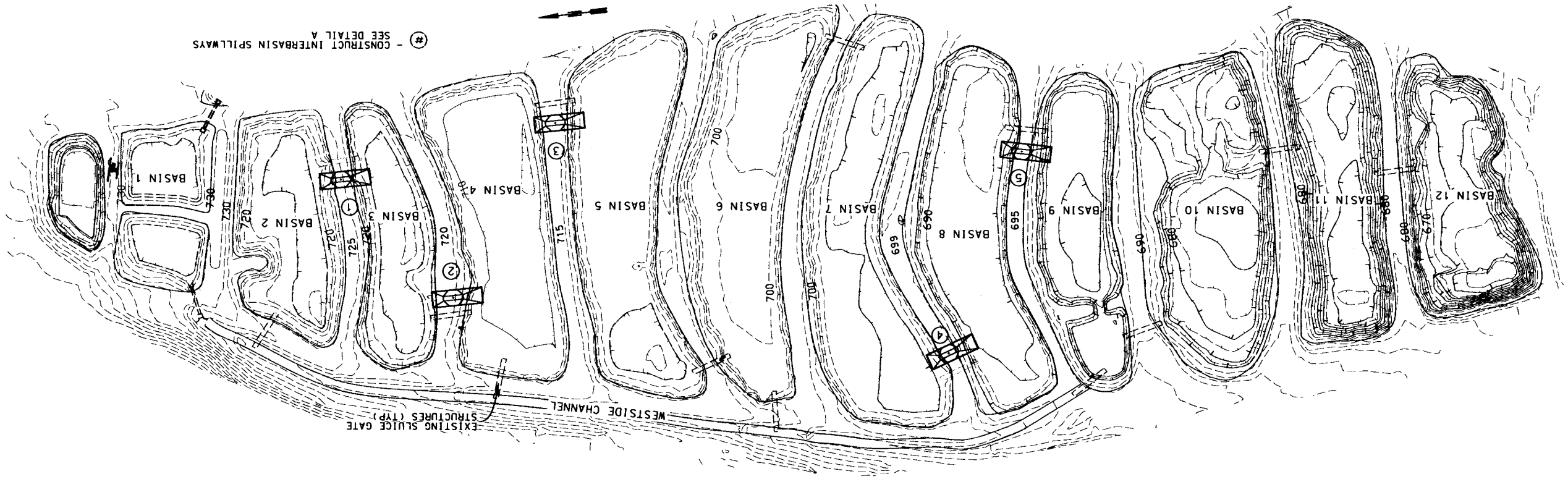


INTERBASIN SPILLWAY ELEVATIONS

SPILLWAY NO.	SPILLWAY CREST EL. ELEV A	DOWNSTREAM TOE EL. ELEV B	UPSTREAM TOE EL. ELEV C	EX. TOP OF LEVEE ELEV D	EX. BASIN INVERT EL.
①	722	714	718	725	718
②	717	710	716	720	714
③	712	704	711	715	710
④	697	690	695	699	692
⑤	693	683	691	695	688

INTERBASIN SPILLWAY RIP-RAP QTY APPROX: 60 CY/EA

PLAN VIEW
SANTA ANITA SPREADING GROUNDS
NOT TO SCALE



REVISIONS		
DATE	MK	DESCRIPTION

PROJECT ENGINEER _____ DATE _____

R. ZIMMER — REGISTERED PROFESSIONAL ENGINEER — STATE OF CALIFORNIA — CIVIL

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA SPREADING GROUNDS
IMPROVEMENT PROJECT

INTERBASIN SPILLWAYS PLAN AND DETAIL

JOB H032185Z DWG | SHEET 2 OF 6

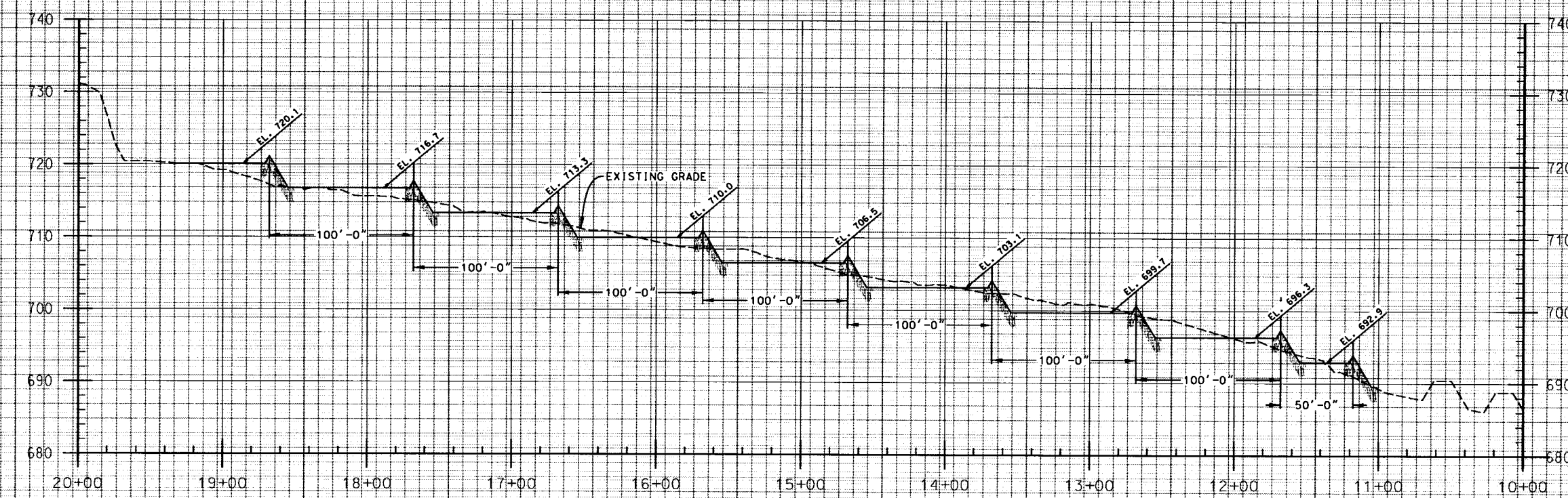
PROC.DWG

DATE: 00/00/19 | TIME: 07:TIME | FILE: BY:FILE.S

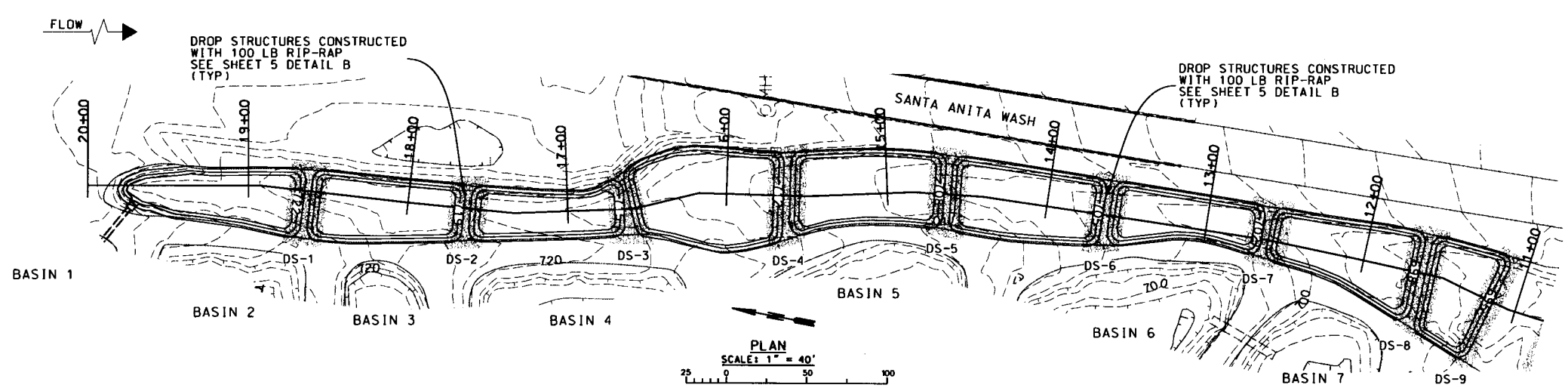
750

SCALE: HOR. 1"= 40'
VERT. 1"= 8'

REVIEWED BY:
DATE
CONSULTANT DESIGN SECTION



PROFILE
EASTSIDE CANAL DROP STRUCTURES



PLAN
SCALE: 1" = 40'
25 0 50 100

PLAN VIEW
EASTSIDE CANAL DROP STRUCTURES

RIP-RAP QTY TABLE

DROP STRUCTURE	RIP-RAP QTY
DS-1	115 CY
DS-2	100 CY
DS-3	110 CY
DS-4	140 CY
DS-5	125 CY
DS-6	110 CY
DS-7	95 CY
DS-8	150 CY
DS-9	175 CY
TOTAL	1120 CY

DATE	MK	DESCRIPTION

REVISIONS

REGISTERED PROFESSIONAL ENGINEER
K. ZIMMER
NO.
EXP.
CIVIL
STATE OF CALIFORNIA

PROJECT ENGINEER DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA SPREADING GROUNDS
IMPROVEMENT PROJECT

EASTSIDE CANAL PLAN AND PROFILE

JOB H0321857 DWG SHEET 3 OF 6

DRAFTER H. TREMMEL
DESIGNER H. TREMMEL
CHECKER
CADD PROJECT FILE NAME
REVIEWED BY
DATE

DATE: $DATE$ TIME: $TIME$ FILE: $FILEL$

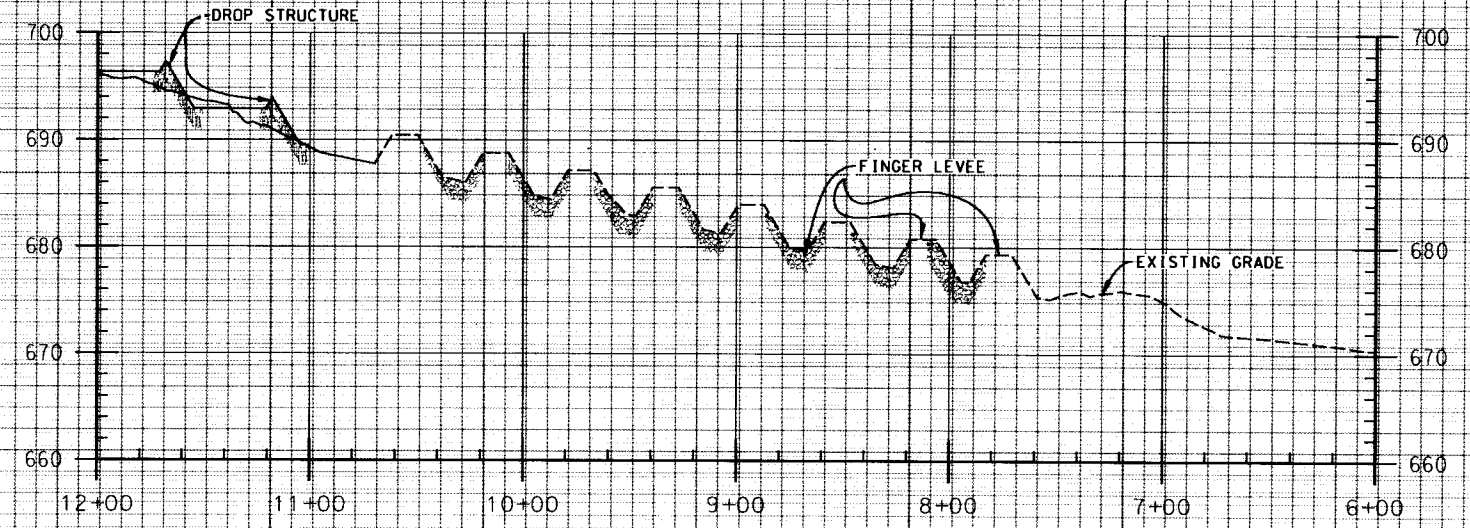
750

SCALE: HOR. 1"= 40'
VERT. 1"= 8'

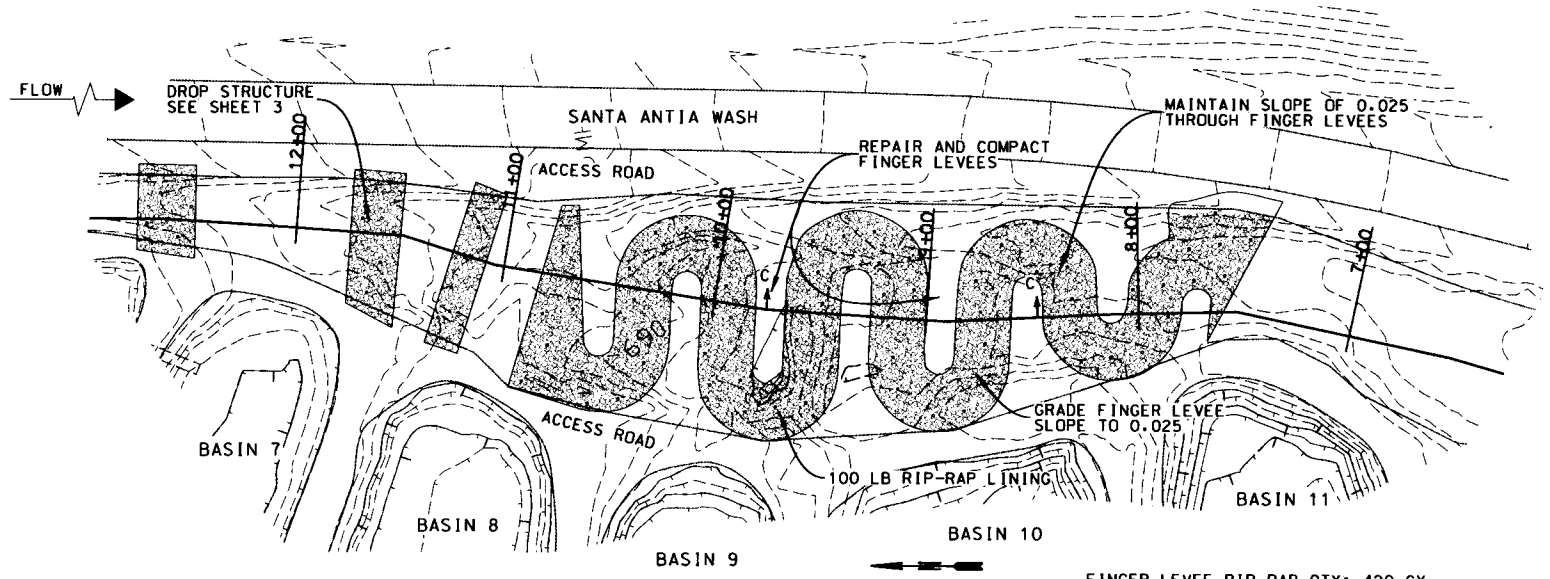
REVIEWED BY:

DATE

CONSULTANT DESIGN SECTION



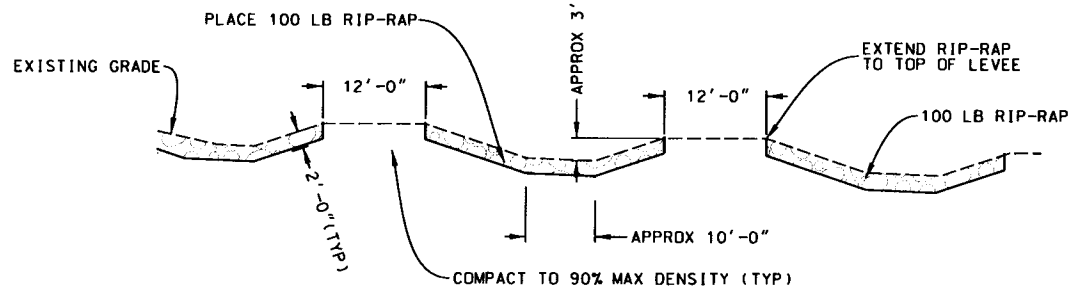
PROFILE
FINGER LEVEES



PLAN
SCALE: 1" = 40'
25 0 50 100

PLAN VIEW
FINGER LEVEES

FINGER LEVEE RIP-RAP QTY: 420 CY



SECTION C-C
FINGER LEVEE (TYP)
NOT TO SCALE

DATE	BY	DESCRIPTION
		REVISIONS

REGISTERED PROFESSIONAL ENGINEER
K. ZIMMER
NO.
EXP. CIVIL
STATE OF CALIFORNIA

PROJECT ENGINEER DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

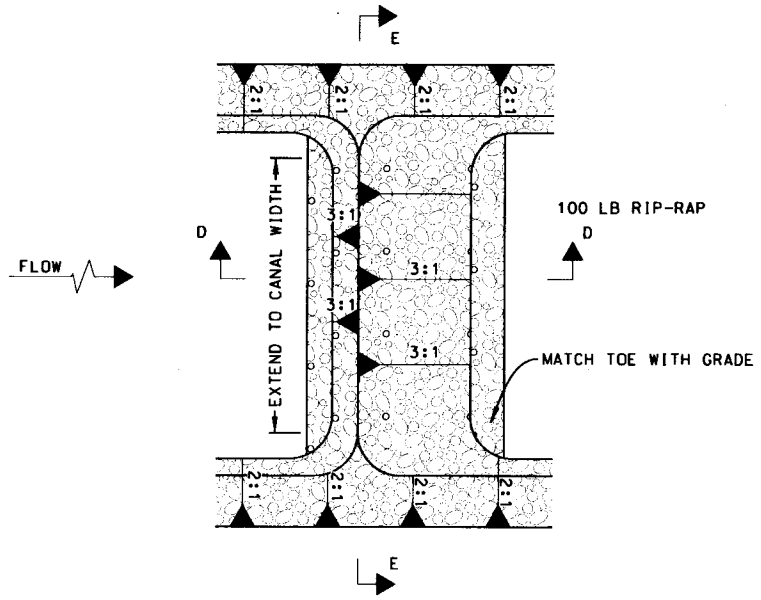
SANTA ANITA SPREADING GROUNDS IMPROVEMENT PROJECT

FINGER LEVEE PLAN AND DETAIL

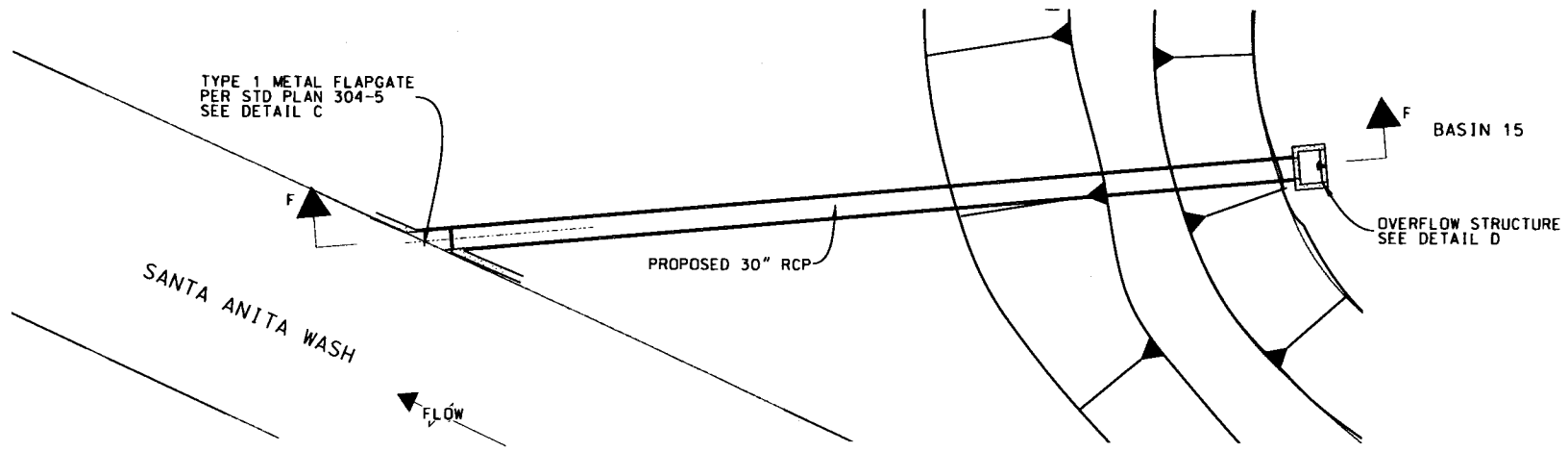
JOB H0321857 | DWG | SHEET 4 OF 6

DRAFTER | DESIGNER | CHECKER | CADD PROJECT FILE NAME | REVIEWED BY | DATE

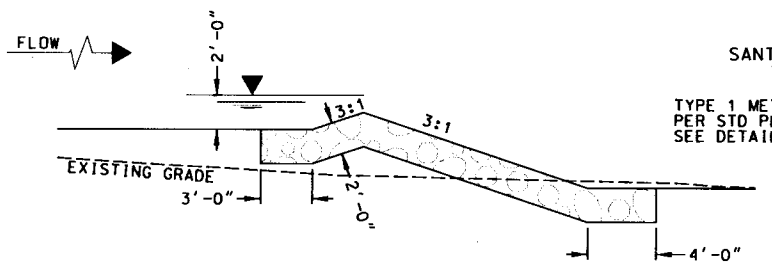
DATE: %DATE% TIME: %TIME% FILE: %FILE%



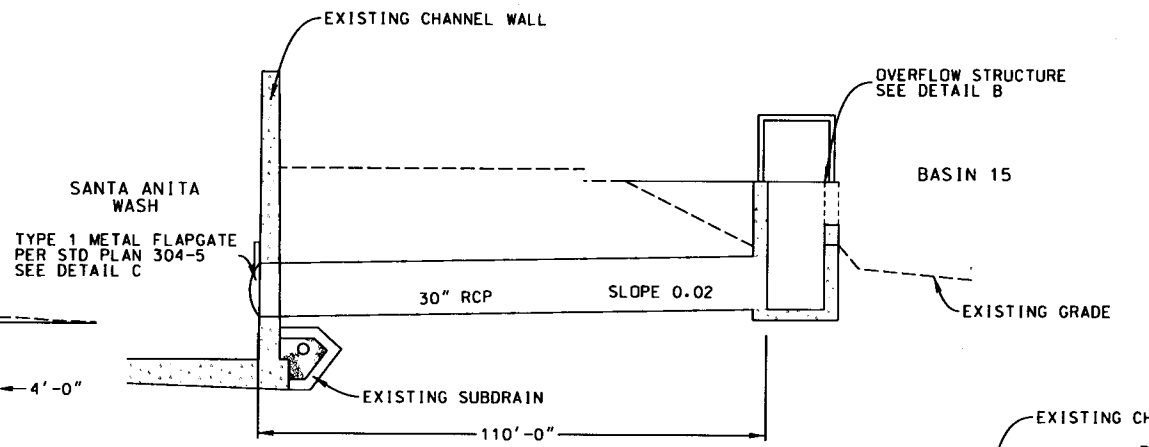
DETAIL B (SH.3)
DROP STRUCTURE (TYP)
NOT TO SCALE



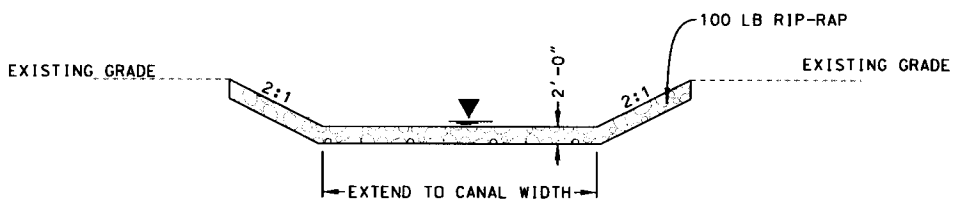
PLAN VIEW
BASIN 15 OVERFLOW STRUCTURE
NOT TO SCALE



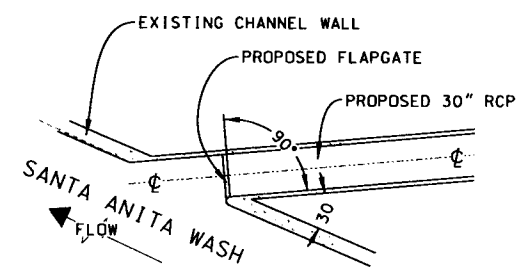
SECTION D-D
DROP STRUCTURE (TYP)
NOT TO SCALE



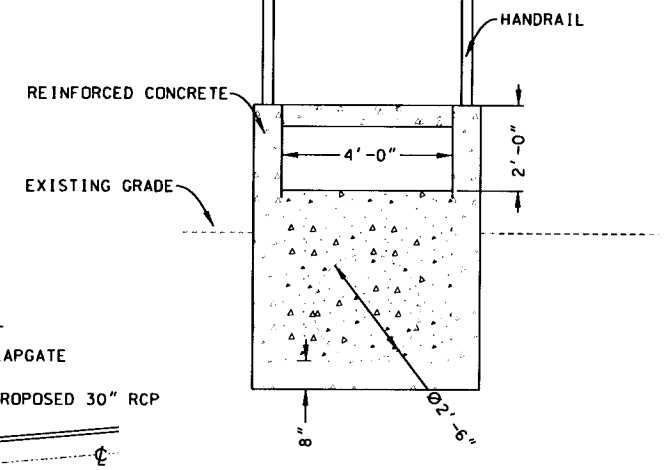
SECTION F-F
BASIN 15 OVERFLOW STRUCTURE
NOT TO SCALE



SECTION E-E
DROP STRUCTURE (TYP)
NOT TO SCALE



CHANNEL JUNCTION STRUCTURE DETAIL "C"
BASIN 15 OVERFLOW STRUCTURE
NOT TO SCALE



DETAIL D
OVERFLOW WEIR
NOT TO SCALE

DRAFTER	DESIGNER	CHECKER	CADD PROJECT FILE NAME	REVIEWED BY	DATE
H. TREMMEL	H. TREMMEL				

DATE	MK	DESCRIPTION
		REVISIONS

K. ZIMMER
CIVIL
PROJECT ENGINEER DATE

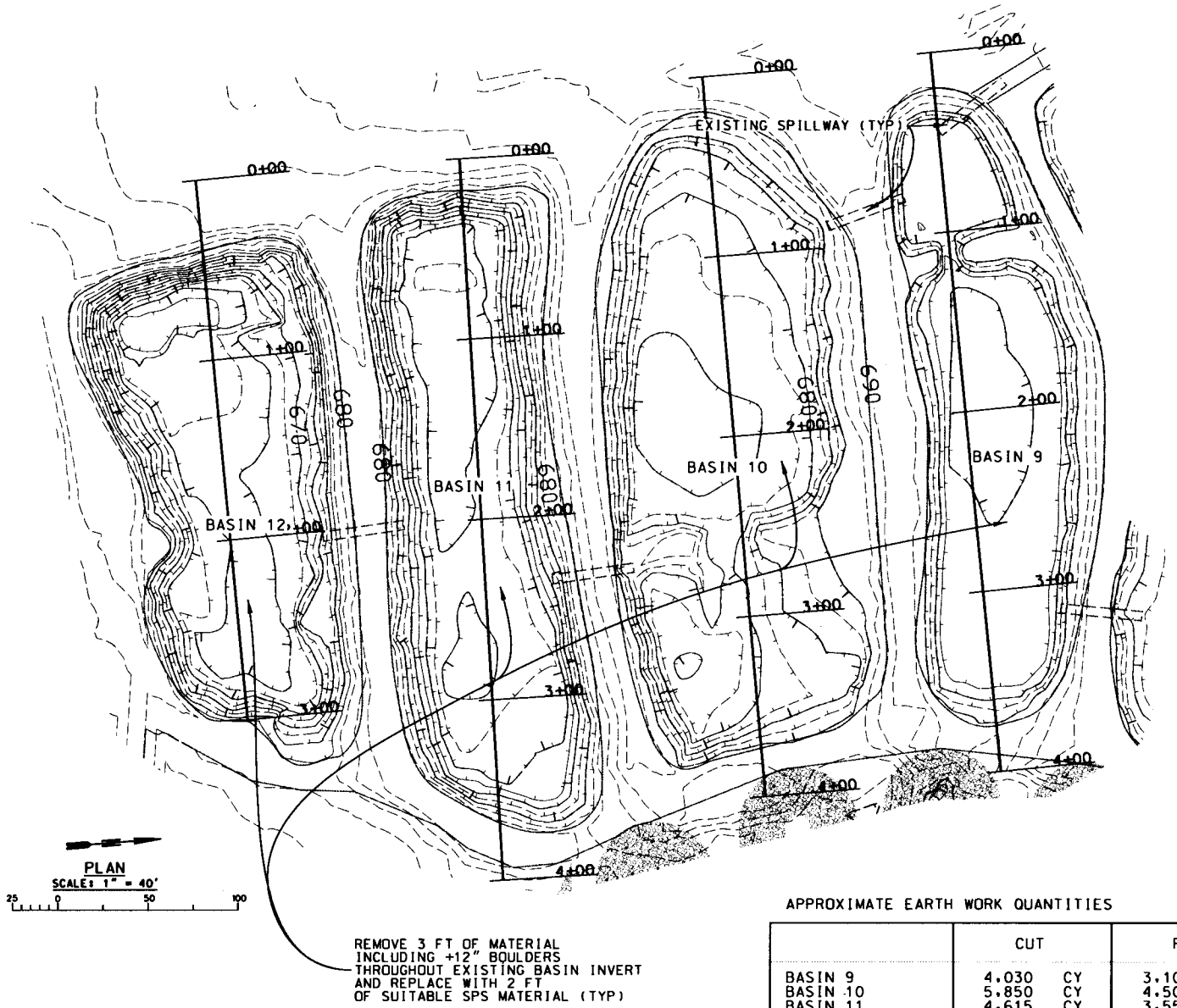
COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA SPREADING GROUNDS IMPROVEMENT PROJECT

DROP STRUCTURE & BASIN 15 OVERFLOW STRUCTURE PLAN AND DETAIL

JOB H0321857 DWG SHEET 5 OF 6

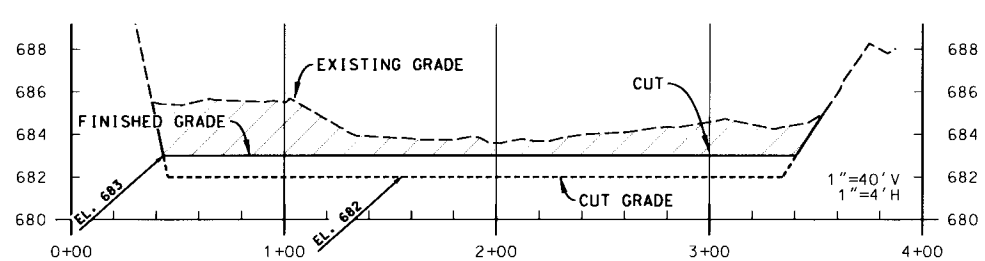
DATE: 8DATE8 TIME: 8TIME8 FILE: 8FILE8



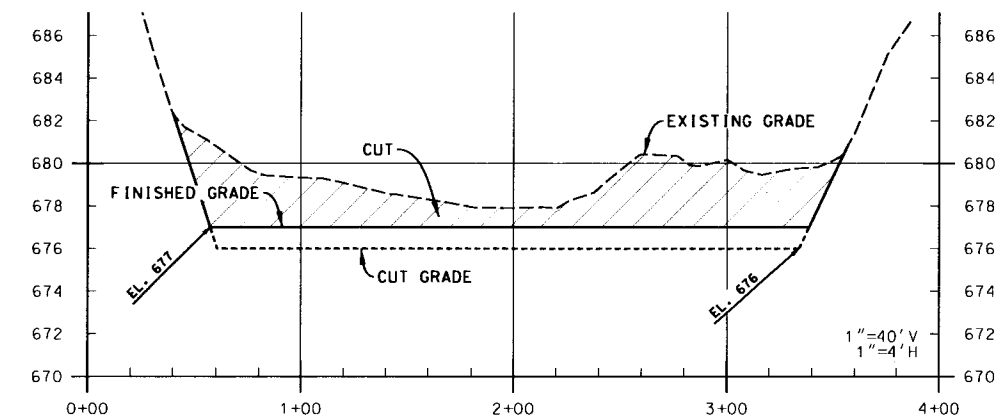
PLAN VIEW
SANTA ANITA SPREADING GROUNDS

APPROXIMATE EARTH WORK QUANTITIES

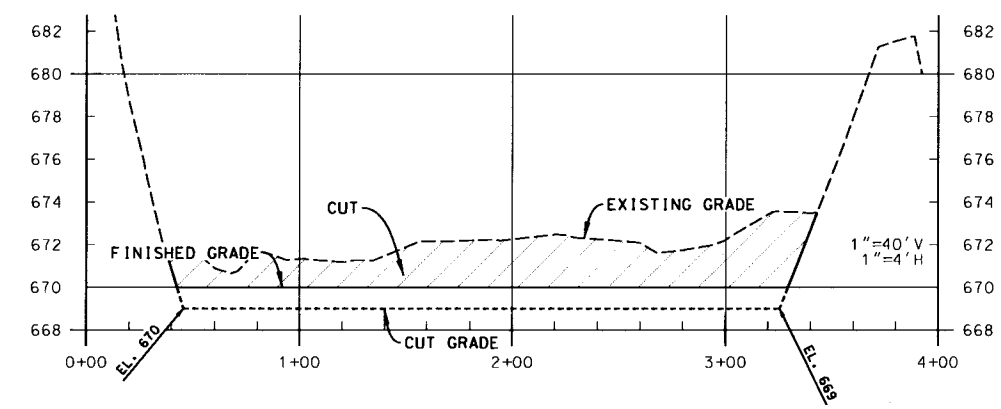
	CUT	FILL
BASIN 9	4,030 CY	3,100 CY
BASIN 10	5,850 CY	4,500 CY
BASIN 11	4,615 CY	3,550 CY
BASIN 12	3,900 CY	3,000 CY
TOTAL	18,395 CY	14,150 CY



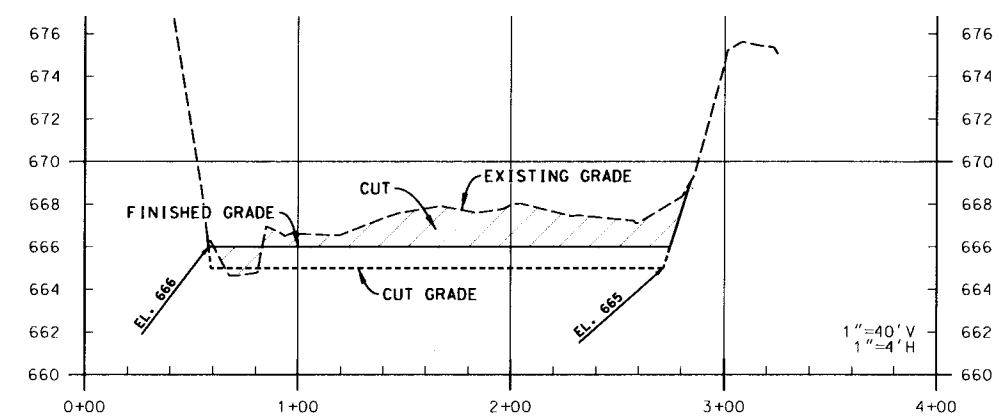
CROSS-SECTION
BASIN 9



CROSS-SECTION
BASIN 10



CROSS-SECTION
BASIN 11



CROSS-SECTION
BASIN 12

DATE	MK	DESCRIPTION
REVISIONS		

REGISTERED PROFESSIONAL ENGINEER K. ZIMMER NO. EXP. CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA SPREADING GROUNDS
IMPROVEMENT PROJECT

BASIN 9, 10, 11, AND 12
CLEANOUT PLAN AND DETAIL

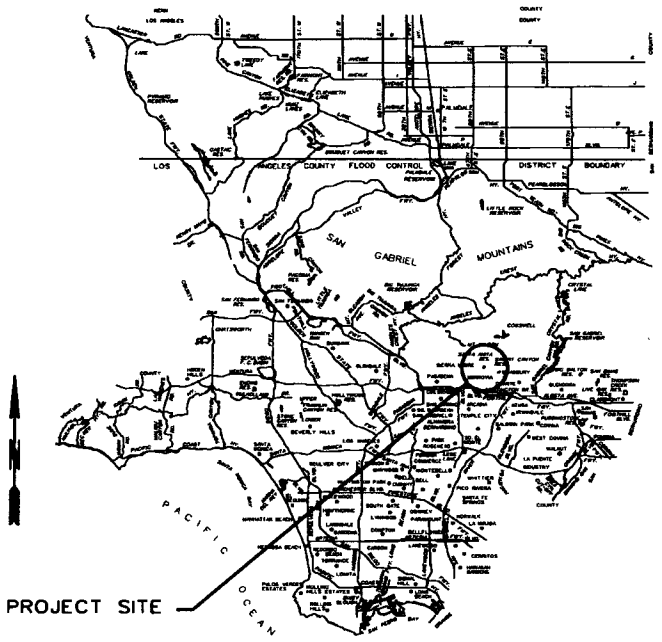
JOB H0321857 DWG SHEET 6 OF 6

DRAFTER H. TREMMEL | DESIGNER H. TREMMEL | CHECKER | CADD PROJECT FILE NAME | REVIEWED BY | DATE

DATE: $DATE$ TIME: $TIME$ FILE: $FILEL$

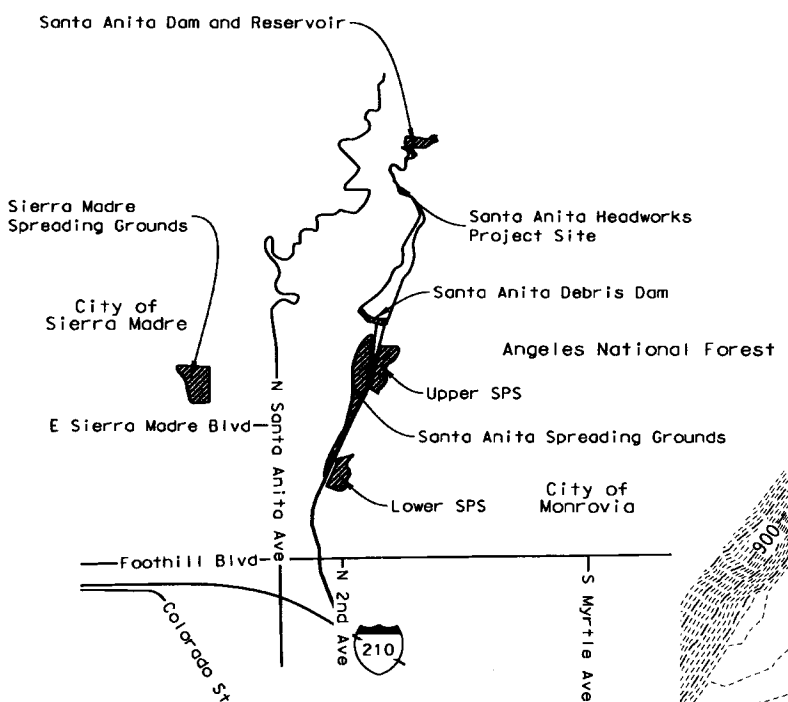
COUNTY OF LOS ANGELES
DEPARTMENT OF PUBLIC WORKS

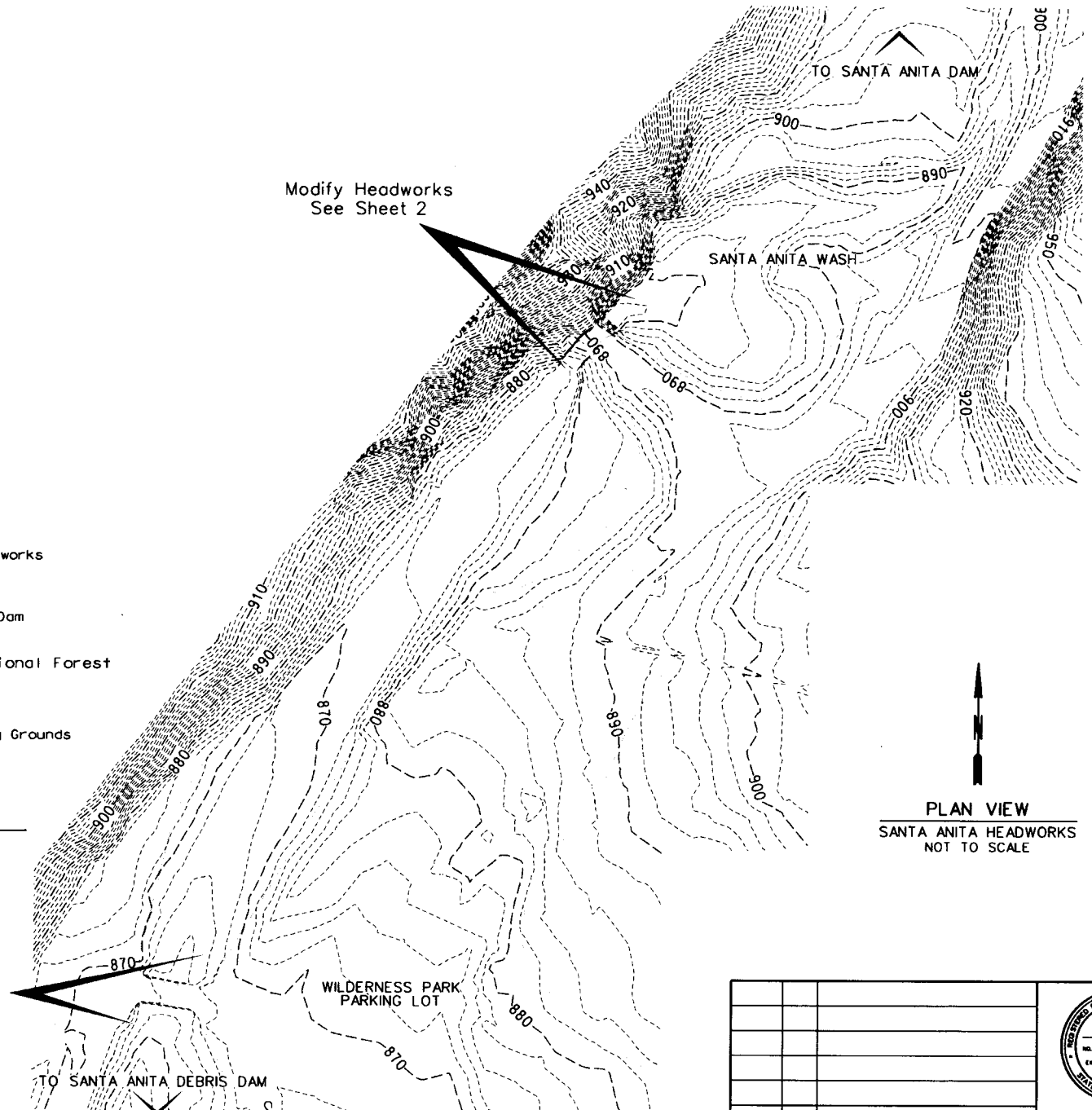
SANTA ANITA SPREADING GROUNDS
HEADWORKS IMPROVEMENT CONCEPT



Thomas Bros. Pgs. 567-D1

LOCATION MAP





VICINITY MAP
NOT TO SCALE

PLAN VIEW
SANTA ANITA HEADWORKS
NOT TO SCALE

REFERENCES

1 SantaAnitaSG-SDB-36-T31-July2007

INDEX TO PROJECT PLANS

SH. NO.	DESCRIPTION
1	TITLE SHEET
2	HEADWORKS PLAN AND PROFILE
3	BRIDGE PLAN

DATE	MK	DESCRIPTION

REVISIONS

REGISTERED PROFESSIONAL ENGINEER K. ZIMMER NO. EXP. CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER DATE

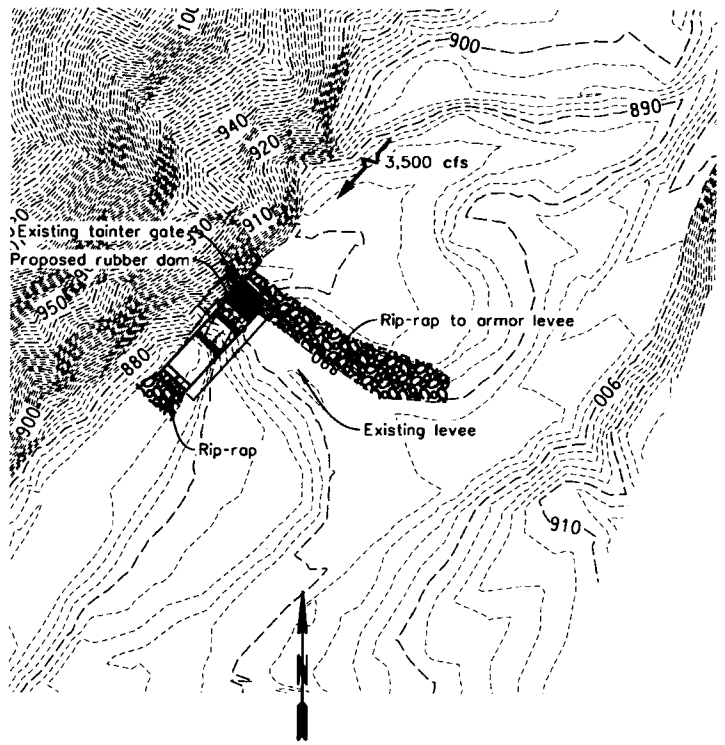
COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA HEADWORKS

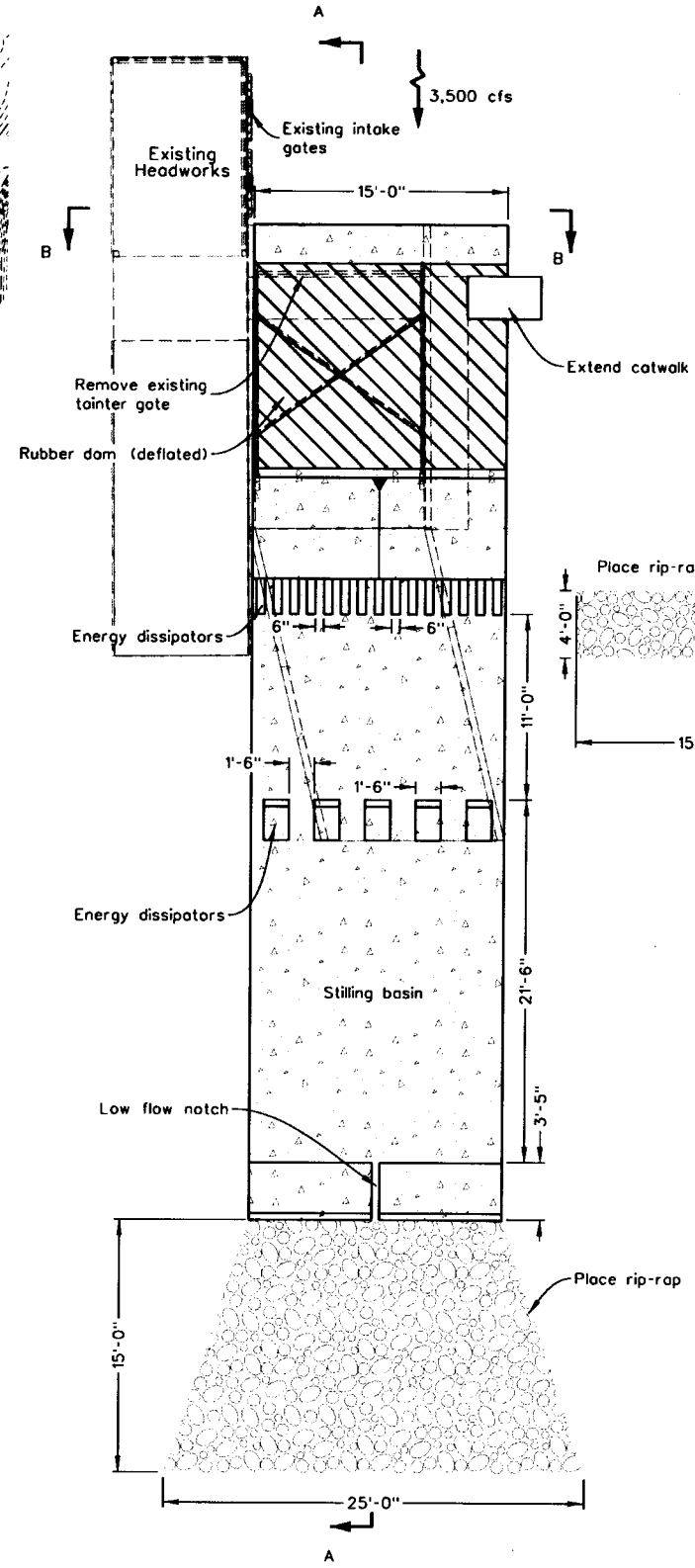
TITLE SHEET

1/20/2011 | JOB H0321857 | DWG | SHEET 1 OF 3

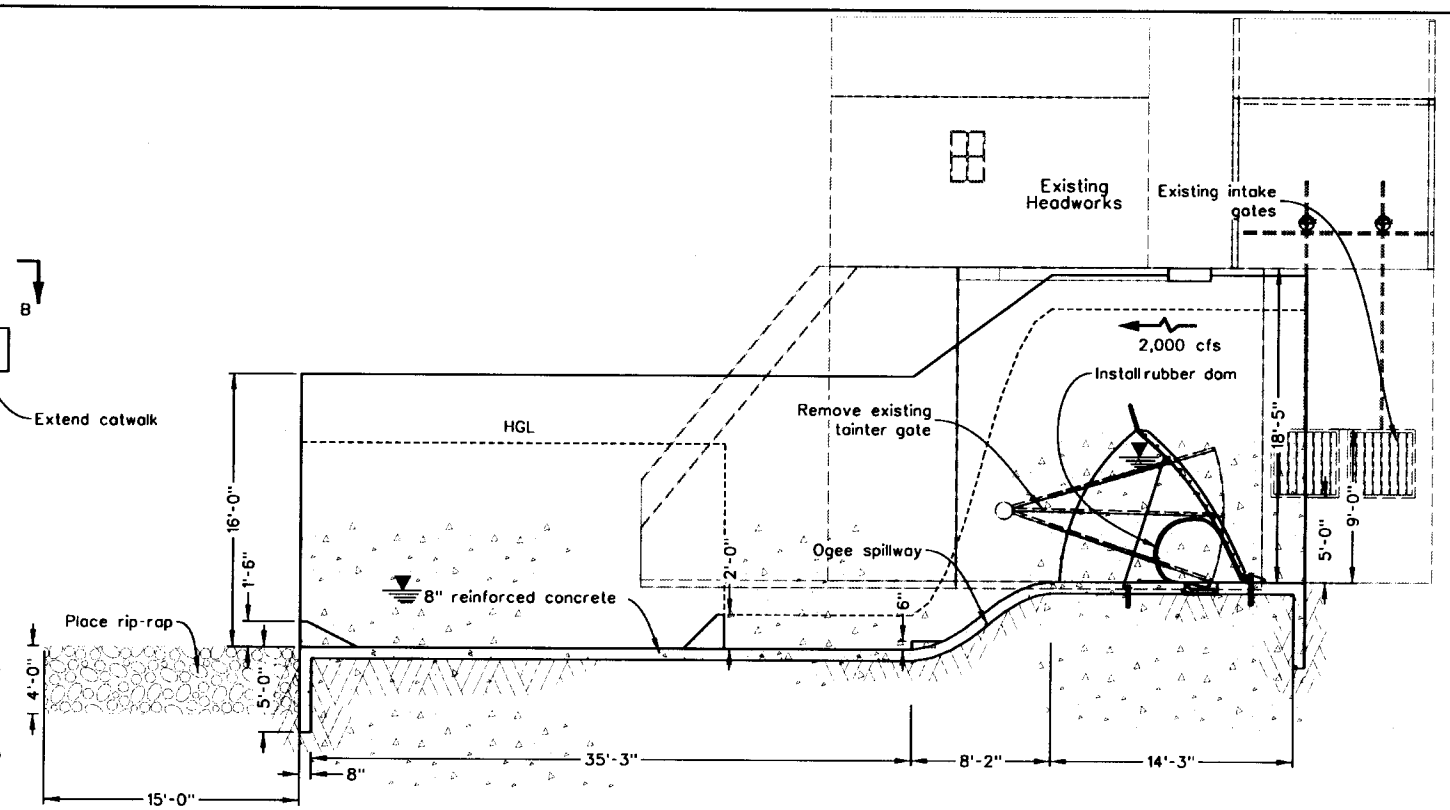
DRAFTER H. TREMMEL | DESIGNER H. TREMMEL | CHECKER | CADD PROJECT FILE NAME



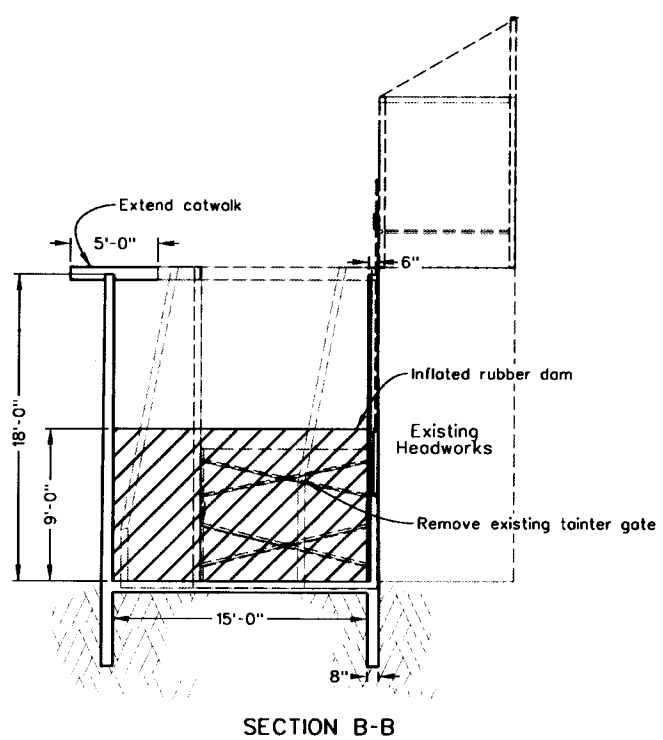
PLAN VIEW
RUBBER DAM ALTERNATIVE 2
NOT TO SCALE



PLAN VIEW
RUBBER DAM
NOT TO SCALE



SECTION A-A
RUBBER DAM
NOT TO SCALE



SECTION B-B
RUBBER DAM
NOT TO SCALE

Santa Anita Headworks
Rating Table

Head from Headworks Invert (ft)	Flow, Q (cfs)	Notes
1	81	
2	217	
3	380	
4	559	
5	749	
6	947	
7	1,151	
8	1,361	
9	1,574	
10	1,790	
11	2,008	
12	2,229	
13	2,451	
14	2,675	
15	2,900	
16	3,126	
17	3,353	
18	3,580	Headwork's maximum capacity
19	3,780	Water begins to overtop breakaway levee
20	4,137	
21	4,604	
22	5,156	
23	6,476	
23.4	6,882	50yr 3rd day design storm

DATE	MK	DESCRIPTION

REVISIONS

REGISTERED PROFESSIONAL ENGINEER K. ZIMMER NO. EXP. CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

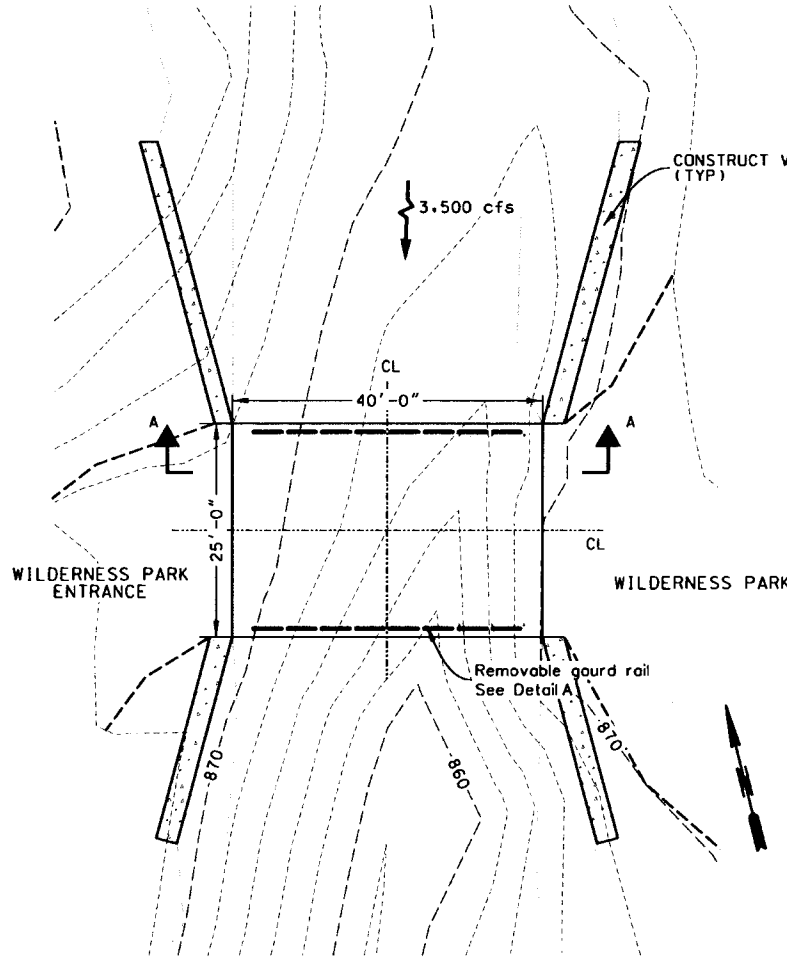
SANTA ANITA HEADWORKS

HEADWORKS
PLAN AND PROFILE

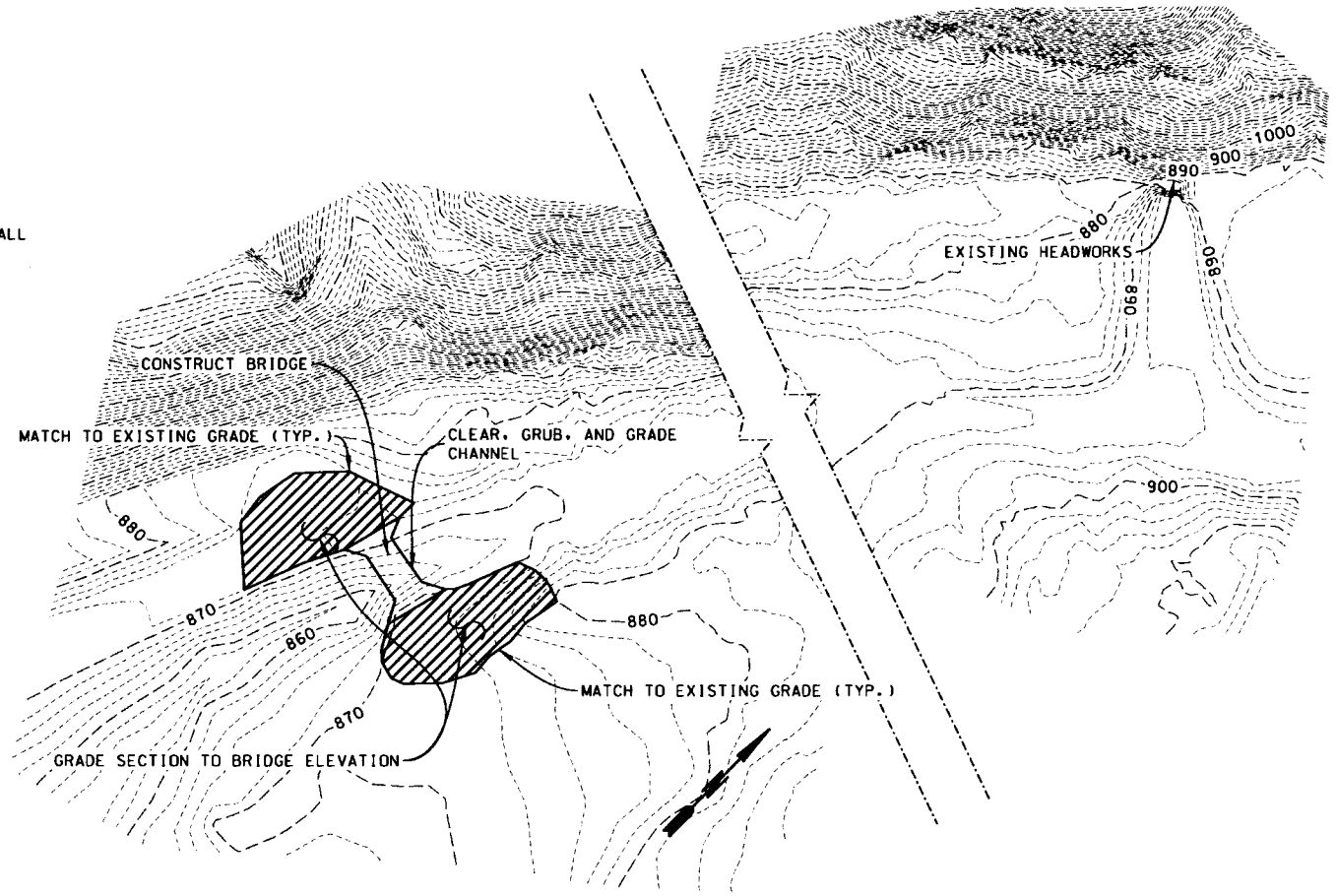
1/20/2011 | JOB H0321857 | DWG | SHEET 2 OF 3

DRAFTER H. TREMMEL | DESIGNER H. TREMMEL | CHECKER | CADD PROJECT FILE NAME | REVIEWED BY | DATE

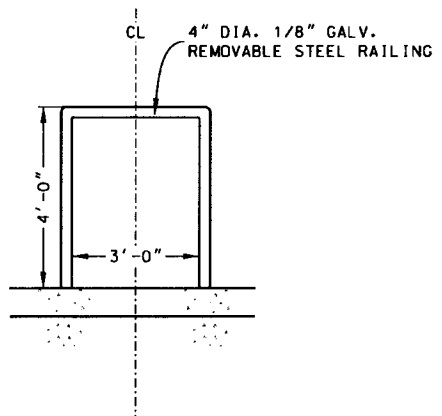
DATE: 00ATE0 TIME: 0TIME0 FILE: 0FILEL0



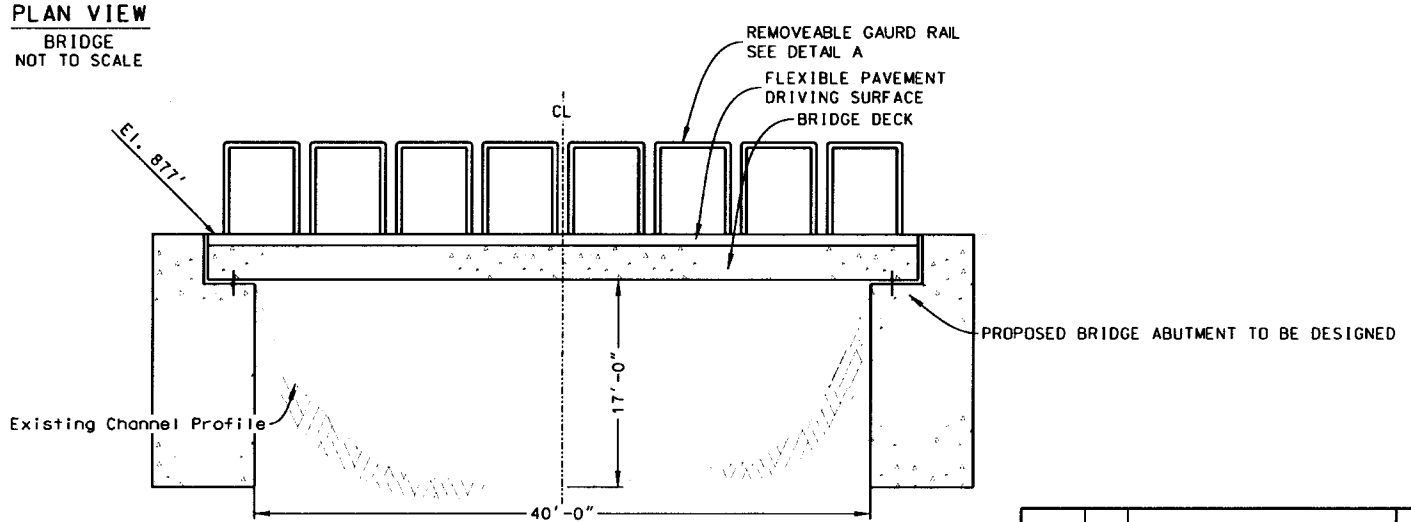
PLAN VIEW
BRIDGE
NOT TO SCALE



PLAN VIEW
BRIDGE AND CHANNEL
NOT TO SCALE



DETAIL A
REMOVABLE GUARD RAIL
NOT TO SCALE



SECTION A-A
CROSS-SECTION OF BRIDGE
NOT TO SCALE

Proposed Bridge
Rating Table

Head from Channel Invert (ft)	Flow, Q (cfs)
1.5	75
2.5	300
6.0	1,000
9.0	2,200
15.0	6,900

DRAFTER	DESIGNER	CHECKER	CADD PROJECT FILE NAME	REVIEWED BY	DATE
H. TREMMEL	H. TREMMEL				

DATE	MK	DESCRIPTION

REVISIONS

K. ZIMMER
No. ____
Exp. ____
CIVIL
REGISTERED PROFESSIONAL ENGINEER
STATE OF CALIFORNIA

PROJECT ENGINEER — DATE

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

SANTA ANITA HEADWORKS

BRIDGE PLAN

DATE 01/20/2011 | JOB H0321857 | DWG | SHEET 3 OF 3

DATE: $DATE$ TIME: $TIME$ FILE: $FILE$