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SYNOPSIS: Stevens Creek Dam is a rolled earthfill structure with a height of about 120 feet. A seismic safety evaluation in 1978 concluded that the dam would probably be severely damaged if subjected to a maximum credible earthquake originating on the nearby San Andreas fault. Normal reservoir operation was immediately restricted, pending decisions as to the fate of the project. Conceptual design studies were completed in 1982 to identify the most promising alternative remedial concepts. The most promising concepts were chosen and final design of modifications to the dam and its appurtenant hydraulic structures were completed in 1984. Modification of the dam was completed in 1986, involving the placement of massive buttresses on both the upstream and downstream slopes and installation of internal drains to control the phreatic line. This required extension of the existing outlet conduit and construction of new inlet and outlet control structures.

INTRODUCTION

Stevens Creek Dam, owned by the Santa Clara Valley Water District, is located on the west side of the highly populated Silicon Valley in California (Fig. 1). The dam was designed and constructed in 1935, according to then-modern standards. It is located 2½ miles from the San Andreas fault, which is capable of a maximum credible earthquake (MCE) of Magnitude 8½. No major problems were reported over the 43 years following construction.

This paper describes site conditions, selected aspects of the 1978 seismic safety evaluation, alternative remedial design concepts that were evaluated, details of the final design that was ultimated selected, results of field, laboratory and engineering analyses on which the evaluations and the final design were based, and selected aspects of construction.

SITE CONDITIONS

Stevens Creek Reservoir, with a capacity of about 3,500 acre-feet, impounds water for ground water recharge and also provides some flood control and recreation benefits. The dam is constructed of a gravelly, clayey sand, referred to as the Santa Clara formation, which underlies the dam and reservoir area. The dam section reportedly consists of an upstream "impervious" zone and a downstream "pervious" zone, separated by a contact inclined downward and upstream at 1:1 (horizontal to vertical) from about the midpoint of the crest. The foundation of the dam consists of Santa Clara Formation, overlain by up to 15 feet of alluvium and terrace deposits. A cutoff trench was excavated up to 40 feet to bedrock beneath the "impervious zone".

SEISMIC SAFETY EVALUATION

A state-of-the-art seismic safety evaluation of Stevens Creek Dam was completed by Wahler Associates in 1978, as part of a comprehensive investigation of the Santa Clara Valley Water District's seven major dams.

Field and Laboratory Investigations

Because the cobbly materials in the embankment had defied previous, conventional "undisturbed" sampling (by others), an innovative

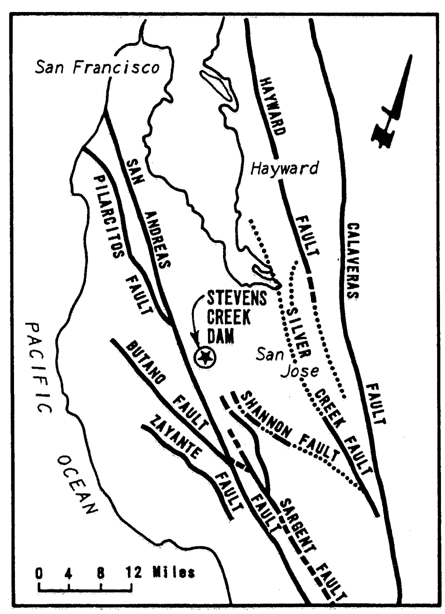

Fig.1. Vicinity Map

technique was developed by Wahler Associates for taking relatively undisturbed samples from large-diameter bucket auger holes. A specially -designed bucket auger drilling machine was suspended from a 70-ton truck-mounted construction crane. Two holes were drilled, on the crest and the upstream face of the dam. Vertical samples were obtained by driving 6-inch-diameter heavy-walled sample tubes, with the kelly, at approximately 5-foot intervals as each hole was advanced, and the amount of energy required to advance the sampler was recorded for the purpose of estimating the equivalent penetration resistance of the materials. Six-inch-diameter horizontal push samples were also obtained from each of the drill holes, using a new horizontal sampling device, designed and fabricated by Wahler Associates. The new device uses a fixed-piston sampling technique to hydraulically push a relatively thin-walled tube. The sampler was mounted in a cage, designed to carry an operator, which was lowered into the 48-inch-diameter hole. The temporary casing that lined the hole had open "windows" along its length to permit selection of representative sampling locations within the dam. Several samples from each depth were obtained by rotating the casing to expose new sampling locations. Approximate in-place density determinations of the embankment materials were made in the 48-inch drill holes by weighing the entire cylinder of soil removed between known depths. Sand cone field density tests were conducted in exploratory trenches in the surficial foundation soils.

Laboratory testing included classification, density, permeability, static and dynamic strength and resonant column tests. Basic material properties determined from those tests are summarized in Table 1. It should be noted that only slight differences were found to exist in the gradations of the "impervious" and "pervious" materials.

Table 1 - Summary of Material Properties from 1978 Study

Property	Existing Dam	Terrace Gravel	Alluvium
Ave. Unit Dry Wt. (pcf)	120	131	106
Ave. Gradation (%)			
No. 4	72	53	30
No. 200	25	8	14
Liquid Limit (%)	25-30	N.P.	N.P.
Plastic Index (%)	7-13		
Permeability (cm/sec)			
Horiz.	$5x10^{-6}$	--	--
Vert.	$4x10^{-7}$	--	--
Strength			
ø' (deg.)	37	--	36
c' (ksf)	0.4	--	ø

Analyses and Results

A dynamic finite element analysis was performed on the maximum section of the dam. The finite element model was subjected to the expected motions from an MCE with a peak bedrock acceleration of 0.72 gravity and a duration of 100 seconds. The accelerogram and shear modulus values used in the analysis were adopted from previous studies by others (Woodward-Clyde Consultants, 1976).
The results of the seismic stability evaluation indicated that strain potentials in most of the embankment would be much greater than 10 percent. However, meaningful quantitative estimates of deformation were not possible because the exact magnitude of the strain potentials in excess of 10 percent could not be determined (few of the laboratory cyclic triaxial tests could be carried out to cyclic strains greater than 10 percent). It was concluded that the dam had a high potential for lateral spreading and crest settlement, which could result in a reduction in freeboard below the reservoir level if the reservoir were full or nearly full.

STUDY OF CONCEPTUAL ALTERNATIVES

In 1982, a study was completed (Wahler Associates, 1982), in which conceptual alternatives were evaluated and selected alternatives were detailed at the preliminary design level. Basic data was collected, from additional field geophysical surveys, limited cyclic triaxial and resonant column tests on samples taken from the upstream "impervious" zone of the embankment, and basic tests on potential borrow materials. Comparative, simplified dynamic analyses were performed on several trial embankment modification sections. The objectives of this study were to arrive at one or two basic remedial design concepts for restoring full reservoir capacity and to develop preliminary cost estimates for them. Two categories of remedial modification were initially considered: (1) improvement of seismic stability by the addition of buttresses; and (2) control of the phreatic line such as to minimize the areas of seismic concern. The second approach would depend entirely on the effectiveness of internal drains, which are not always reliable, and construction would be far more complicated and costly than the first approach. For those reasons, the second category was not pursued further.

Field and Laboratory Investigations

A primary objective of the field investigation was to better define the shear moduli of both the existing and potential new fill materials. Geophysical surveys were conducted and a limited number of undisturbed samples taken at two, 3 hole arrays; from the crest and the upstream slope. Cross-hole measurements were made with a downhole shear wave hammer at 5-foot intervals. Measured shear modulus (K_{2max}) values varied from 95 to 165 in the "pervious" zone of the dam and from 160 to 210 in the "impervious" zone. Shear modulus values in the "pervious" zone were found to be substantially lower than those measured in

1976. Although values of shear modulus from laboratory resonant column tests were found to consistently underestimate those from the field geophysical surveys, the differences were within the range normally attributed to sample disturbance and the differences in boundary conditions between the two types of test (Arango, et al., 1978, Drnevich, 1977). From laboratory resonant column tests, an average shear modulus value of 85 was calculated for new fill compacted to 95 percent relative compaction (ASTM D1557-78, modified to yield 20,000 ft.-lbs/ft^3 of compactive effort).

Conceptual Alternatives

A total of five conceptual modification alternatives were evaluated for the embankment section. The alternative modifications consisted of (1) upstream and downstream berms of various sizes to provide confinement; (2) partial excavation of the existing very stiff, overly-compacted upstream "impervious" zone and replacement with recompacted fill with a lower shear modulus; (3) similar to (2) above, but with a larger upstream zone; (4) similar to (3) above but including a wider dam crest to lower the phreatic line, and a small berm on the downstream slope to intercept seepage and provide confinement; and (5) similar to (4) above without excavation and replacement of the existing upstream "impervious" zone.

Analyses and Results

One-dimensional seismic response analyses, using the computer program SHAKE (Schnabel et. al., 1972), were carried out on three or four selected soil columns for each section. In addition, simplified deformation analyses were performed on Modification No. 4 using the Makdisi-Seed (1978) method. The strain potential evaluation indicated that most of the embankment materials in all of the schemes would have strain potentials greater than 10 percent. However, the exact magnitudes of strain potentials could not be accurately determined, as had been the case in the 1978 study. On the positive side, the evaluation indicated a substantial reduction in seismically-induced stresses within the existing upstream "pervious" zone (compared to the 1978 study, and due primarily to the reduced shear modulus derived from the 1982 field geophysical survey) and also within the replaced embankment zone (modification alternatives 2 through 4 above). Because it was suspected that the dynamic strength of the upstream zone was probably being underestimated from the limited number of tests available, it appeared that additional laboratory triaxial tests should be carried out during final design before any further thought should be given to replacing this material with new fill. Based upon the above results, two remedial schemes were selected for preliminary design detailing and cost estimating (Fig. 2).

Selected Alternatives

The first scheme consisted of upstream and downstream buttresses, similar to modification alternative 5 above, with the objective of improving the dynamic stability of both the upstream and downstream slopes. Additional freeboard was also be provided to compensate

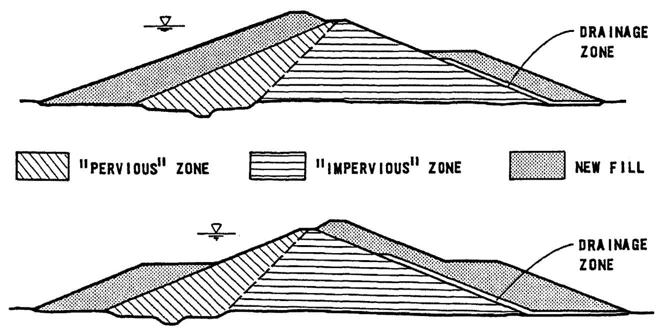

Fig. 2. Selected Design Concepts

for crest slumping, and a drainage zone was included between the existing downstream face and the new buttress to intercept seepage. The first scheme assumed no excavation and replacement of the existing upstream "impervious" zone. However, two optional sub-schemes were also evaluated: one involving partial excavation and replacement of existing upstream "impervious" material with lower density, new fill to reduce the shear moduli and thus improve the seismic resistance of the materials; and another utilizing high strength rockfill in the upstream berm to improve the seismic resistance of that slope.

The philosophy of the second scheme was entirely different from that of the first scheme. Instead of focusing on improving the dynamic stability of the upstream slope, the second approach was to provide additional mass and confinement downstream, to preclude catastrophic failure if there were severe damage to the upstream slope. A limited upstream buttress was, nevertheless, incorporated, to provide mass at the toe.

FINAL DESIGN

Field and Laboratory Investigations

Additional sampling of existing embankment and potential borrow materials was necessary for final design. Laboratory testing focused on better defining the cyclic and post-cyclic shear strength of the "impervious" and "pervious" materials in the dam, and the compaction, static, cyclic and post-cyclic strength, and shear modulus of potential fill materials. Selected cyclic strength data are presented on Figure 3. Post-cyclic strengths of existing and new fill was derived from samples strained to 6 to 10 percent under cyclic loading, then sheared, statically, to a post-cyclic level of 10 percent. The cyclic shear strength of borrow materials was interpreted from fabricated samples compacted to 95 percent relative compaction (ASTM D1557-78, modified to yield 20,000 ft.-lbs/ft^3 of compactive effort). Post-cyclic strength of new fill was derived from samples compacted to 98 percent relative compaction. The interpreted parameters were $\phi'=31°$ and 11.5°, and $c'=0$ and 10.4 psi, respectively, for existing

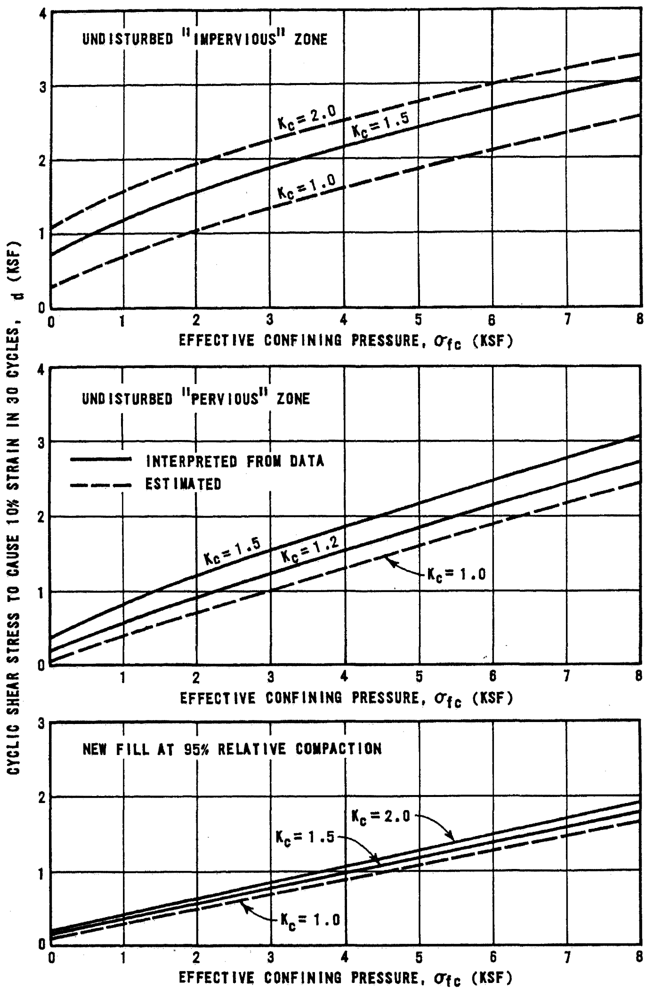

Fig. 3. Cyclic Shear Strengths

and new fill materials, plotted in terms of shear stress on the failure plane at failure vs. normal stress on the failure plane during consolidated.

Analysis of Alternatives

Final design analyses were carried out on the two selected preliminary design alternatives (Fig. 2), using static and dynamic finite element methods. The static non-linear finite element computer program FEADAM (Duncan, Wong and Ozawa, 1980) was used to simulate the embankment stresses and strains created by construction of the original dam and its alternative modifications. Seepage forces were then superimposed on the basis of flownets, assuming a ratio of horizontal to vertical permeabilities equal to 16. The dynamic response of the alternative modifications was analyzed for the same design earthquake as used in the 1978 study, using the non-linear finite element computer program QUAD-4 (Idriss, et al, 1973). The shear modulus values used in the analyses were the same as developed in the 1982 study. A simple average of the curves for sand and clay (Seed and Idriss, 1970) was used to represent the modulus and damping attenuation characteristics of the existing and new embankment materials. The calculated maximum crest accelerations were 1.63 g and 1.25 g, respectively, for the two alternatives, indicating amplifications of 2.25 and 1.74 times the baserock acceleration. Fundamental periods were calculated to be 0.77 and 0.78 second, respectively. Stresses from the response analyses were converted to equivalent uniform cyclic shear stresses, using standard weighting curves (Seed, et al, 1975). Based on these curves, the irregular shear stress time histories in each element were represented by 30 cycles of uniform shear stress equal to two-thirds of the maximum induced shear stress. The strain potential for each element was calculated by comparing the stress required to cause specific strains, with the induced equivalent uniform cyclic shear stresses. The strain potential evaluation indicated that the strain potentials in the central and downstream portions of both remedial alternatives would be relatively low; however, the strain potentials in the entire upstream "impervious" zone of the existing embankment and the entire upstream buttress for both cases would be greater than 20 percent. These levels of strain potential indicated that the upstream materials in both alternatives would be subject to severe spreading and slumping during the MCE. The second alternative was considered to be more acceptable, because the more favorable downstream buttressing of the second alternative, and the fact that the first alternative considerably increased the load on the outlet conduit, which could result in an unsafe condition where the conduit crosses the existing core trench. A post-earthquake analysis was conducted on the second alternative, assuming that the upstream slope provided no support to the remaining embankment. This was a worst-case scenario, since the upstream buttress would, in fact, provide additional mass at a critical location, and the final upstream configuration would probably be merely slumped down from its original condition, on the order of, at worst, a few tens of feet. The results of that analysis indicated a minimum factor of safety of 1.3 for the upstream slope, calculated for a shallow circle that would not undermine the crest of the modified dam. Therefore, it was concluded that the modified embankment, while not necessarily remaining completely intact, would not fail catastrophically if subject to the MCE. In addition, a simplified estimate of permanent deformation was made, using the method of Makdisi and Seed (1978). The calculated total deformation was approximately 10 feet. The modified dam, with its increased normal freeboard (approximately 19 feet), is designed to retain sufficient residual freeboard, following the MCE, to prevent overtopping.

Details of the Final Design

A plan and section of the final design, as constructed, are shown on Figure 4. The dam section, as designed, was essentially the same as was analyzed, except that the bench at Elevation 490 on the downstream face was

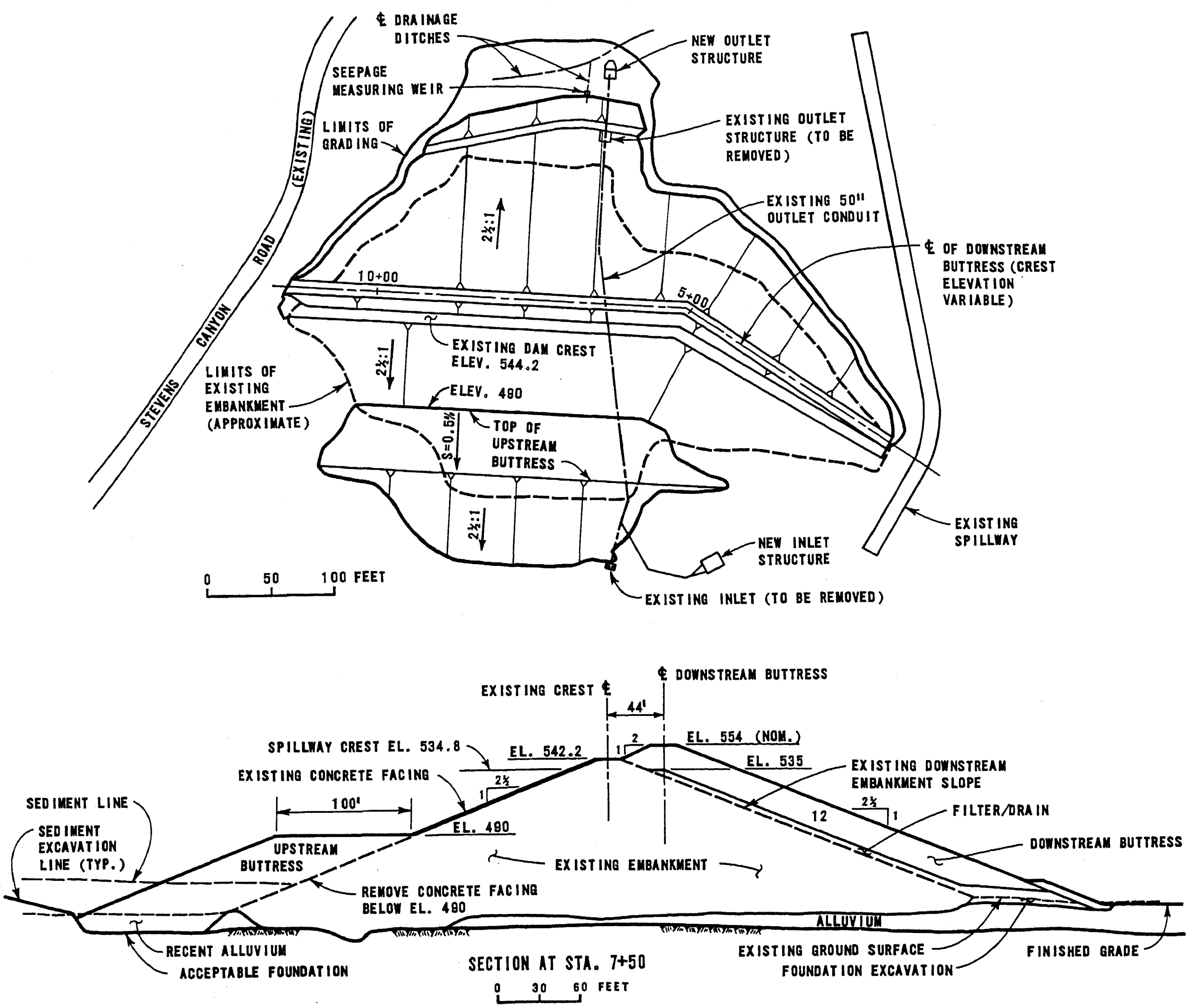

Fig. 4. Plan and Section of Modified Dam

deleted from the final design because it did little to improve the seismic performance of the dam. A 12-foot-thick, inclined chimney drain was incorporated at the interface of the existing downstream embankment face and overlying buttress, to prevent saturation of the buttress and to serve as a crack-stopping zone. The base of the chimney drain is in contact with a blanket drain in the channel, which terminates at an inverted filter at the toe of the dam.

The existing inlet and outlet structures were relocated by extending the 50-inch I.D. outlet conduit upstream and downstream. A new inclined inlet structure on the right abutment was designed to replace the former vertical tower. An inclined structure was chosen due to ease of maintenance and a lower susceptibility to damage from earthquake-induced deformations in the upstream part of the dam. The inlet consists of a wye inlet pipe, controlled by two 42-inch diameter, hydraulically-operated sluice gates, located in a box structure with trashrack-covered ports on the top and sides. The new outlet structure consists of a valve chamber containing two 30-inch butterfly valves, immediately downstream of a wye branch, with dual energy dissipating chambers and stilling basins, and a common outlet apron with warped wing walls.

CONSTRUCTION AND OPERATION

Because of its extreme vulnerability to winter inflows, construction of the modifications took place in a single dry season "window", between June and December 1985. The only significant construction problem was the dewatering and removal of about 70,000 yd³ of very soft sediment (silt) in the bottom of the reservoir, adjacent to the upstream toe of the dam. Lacking definitive information on its depth (over 20 feet) and extent, the contractor relied on backhoe excavation of the silt, without subsurface dewatering. Most of the excavation was carried out under very wet conditions, because the silt was underlain by pervious stream deposits, which created high seepage gradients into the excavations and resulted in several large slides. Several alternative means of handling the wet silt were considered, such as deflocculation and pumping in slurry form, but were not implemented. After a lengthy delay, the silt was removed, leaving final excavation slopes that averaged somewhat steeper than 6:1 (horizontal to vertical). The remarkable steepness of those slopes was due primarily to the stabilizing effect of downward seepage gradients into the underlying pervious stream deposits.

The reservoir filled and spilled shortly after completion of the remedial construction. Data from piezometers installed during the modification work will be carefully monitored to verify the design assumptions regarding the phreatic line within the modified dam.

CONCLUSIONS

The most significant conclusions drawn from the evaluation, remedial design and modification of Stevens Creek Dam were:

The excessive deformations that were predicted to occur in the existing dam in response to the MCE were due, principally, to its very close proximity to the San Andreas fault, the steep embankment slopes and the absence of effective internal drainage.

The very coarse materials in the existing dam required large diameter undisturbed samples for testing. The samples, however, limited the range of cyclic strains that could be induced in the laboratory and the consequent meaningfullness of predicted deformations.

The shear moduli of existing embankment materials could only be determined realistically from cross-hole geophysical tests. Resonant column test results significantly understated the shear moduli due to sample disturbance and other factors.

The response of the saturated upstream part of the existing dam could not be significantly improved by any practical scheme involving replacement of existing materials with more highly compacted materials, or placing buttresses on the upstream slope.

Due to the predicted excessive deformations within the saturated upstream slope of the dam, the most reliable solution was to place massive buttresses on the downstream slope and increase the available freeboard, in order to prevent catastrophic failure. Damage to the upstream part of the dam can be expected, but is not expected to undermine the crest.

Dewatering and excavation of reservoir sediments during construction were found to be the most difficult aspects of construction.

REFERENCES

Arango, I., Moriwaki, U., and Brown, F., (1978), "In-situ and Laboratory Shear Velocity and Modulus", Proc., Earthquake Engineering and Soil Dynamics Specialty Conference, ASCE, June, Pasadena, California.

Drnevich, V.P., (1977), "Resonant Column Testing - Problems and Solutions", Symposium on Dynamic Geotechnical TEsting, ASTM Special Technical Publication 654, June, Denver, Colorado.

Duncan, J.M., Wong, and Ozawa, (1980), "FEADAM: A Computer Program for Finite Element Analysis of Dams:, Report No. UCB/GT80-02, Dept. of Civil Eng., Univ. at California, Berkeley.

Idriss, I.M. Lysmer, J., Hwang, R., and Seed, H.B., (1973), "QUAD-4, A Computer Program for Evaluating the Seismic Response of Soil Structures by Variable Damping Finite Element Procedures", EERC 73-6, Univ. of California, Berkeley.

Makdisi, F.I., and Seed, H.B., (1978), "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations", Journ. Geotechnical Engineering Div., ASCE, Vol. 104, No. GT7 849-867.

Schnabel, P.B., Lysmer, J., and Seed, H.B., (1972), "SHAKE, a Computer Program for Earthquake Response Analysis for Horizontally Layered Sites", EERC 72-12, Univ. of California, Berkeley.

Seed, H.B., and Idriss, I.M., (1970), "Soil Moduli and Damping Factors for Dynamic Response Analysis", EERC, Univ. of California, Berkeley.

Seed, H,B., and Idriss, I.M., Makdisi, F., and Banerjee, N., (1975), "Representation of Irregular Stress Time Histories by Equivalent Uniform Stress Series in Liquefaction Analyses", EERC 75-29, Univ. of California, Berkeley.

Wahler Associates, (1978), "Seismic Safety Evaluation, Stevens Creek Dam", Report to Santa Clara Valley Water District.

Wahler Associates, (1982), "Preliminary Remedial Design Investigation, Stevens Creek Dam, Report to Santa Clara Valley Water District.

Wahler Associates, (1984), "Remedial Design Report, Stevens Creek Dam", Report to Santa Clara Valley Water District.

Woodward-Clyde Consultants, (1976), "Part 1 - Geology and Seismicity - Seismic Stability Assessment of Four Dams", Report to the Santa Clara Valley Water District.