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and Symposium in Honor of Clyde Baker

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ABSTRACT

Crane Valley Dam is located on the North Fork of Willow Creek in Madera County, California, and is owned by Pacific Gas & Electric Company (PG&E). The results of seismic stability analyses performed in 2005 and 2006 showed that the dam's hydraulic fill embankments would experience large deformations during and after the earthquake shaking postulated for the site. To improve the seismic stability and performance of the dam, PG&E initiated the Crane Valley Dam Seismic Retrofit Project (Project), which includes placing new rockfill buttresses on the upstream and downstream slopes of the dam, constructing internal drainage improvements, reinforcing portions of the dam's concrete core wall, and raising the dam crest.

Project components were designed to meet seepage control and seismic stability criteria and to accommodate existing facilities, limited site access, seasonal reservoir operations, and environmentally sensitive areas within and adjacent to the Project site. Engineering analyses included static, seepage, and dynamic finite element analyses to evaluate the potential for liquefaction of hydraulic fill materials and post-earthquake stability of the retrofitted dam embankment. Construction of the Project began in October 2010 and was completed in November 2012.

INTRODUCTION

This paper describes the design and engineering analyses that were performed by AMEC Environment & Infrastructure, Inc. (AMEC) in support of the Crane Valley Dam Seismic Retrofit Project (Project). The Project, which is owned by Pacific Gas and Electric Company (PG&E), of San Francisco, California, is located in the Sierra Nevada foothills, about 40 miles northeast of Fresno, California. The purpose of the Project is to protect public safety by strengthening and improving the dam to meet current standards for seismic performance.

In 2002, the California Department of Water Resources, Division of Safety of Dams (DSOD) reviewed previous seismic stability studies performed for Crane Valley Dam and concluded that the shear strength parameters used for the hydraulic fill portions of the embankment were inconsistent with low standard penetration test (SPT) blowcounts obtained during earlier field investigations. Based on their review, and considering the potential for liquefaction of the dam's hydraulic fill, the DSOD advised PG&E that a new in-depth geotechnical analysis was needed to evaluate the dam's overall seismic stability.

Earlier evaluations of the seismic stability of the dam were performed in 1974 and 1980 and concluded that the dam's performance was adequate for the earthquake ground motions specified at the time. However, these studies relied heavily on the results of cyclic laboratory tests (performed on "undisturbed" samples obtained from borings drilled through the dam) to estimate the cyclic strength of the generally cohesionless hydraulic fill. Because of issues related to sample disturbance, the results of these tests are no longer considered reliable in the current state-of-practice. Current state-of-practice approaches rely on field SPT results to estimate the cyclic resistance and predict the likely behavior of relatively cohesionless soils during and after earthquake shaking.

In 2004, to accommodate changes in the state-of-practice in seismic stability analyses and an increase in the estimated local seismic hazard over the nearly 25-year interval since the last evaluation, PG&E initiated a re-evaluation of the seismic stability of the dam. The dam was re-evaluated for earthquake ground shaking caused by a local (random) earthquake with moment magnitude, M_w, of 6¼, and a magnitude 8.0 event on the distant San Andreas Fault. Results of these analyses showed that the dam's hydraulic fill embankments would liquefy and experience large deformations during and after the earthquake shaking postulated for the site. The magnitude of the deformations was found to be excessive and would likely cause an uncontrolled release of the reservoir water at the current normal maximum operating level. Accordingly, it was concluded that measures were necessary to improve the seismic stability and performance of the dam.

This paper describes the field investigations, engineering analyses, and design of remedial measures to improve the seismic performance of the dam. The field investigations included both onshore and offshore exploration programs. The engineering analyses included static, seepage, and dynamic finite element analyses to evaluate the potential for liquefaction of hydraulic fill materials and the post-earthquake stability of the retrofitted dam embankment.

DESCRIPTION OF CRANE VALLEY DAM

Crane Valley Dam is located on the North Fork of Willow Creek, a tributary of the San Joaquin River, in Madera County, California. Water impounded by the approximately 1,880-foot-long, 145-foot-high dam forms the 4-mile-long Bass Lake. Figure 1 presents an aerial view of the dam. At its normal maximum water surface elevation of about 3,377 feet, Bass Lake has a surface area of 1,165 acres and provides about 45,410 acre-feet of gross and useable storage.

Fig. 1. Aerial View of Crane Valley Dam and Spillway.

Built between 1901 and 1911, the dam is composed of an earth and rockfill embankment with a thin, central concrete core wall. The dam varies in cross-section and includes full hydraulic fill sections (i.e., hydraulic fill embankments on both sides of the core wall) near the west and east ends of the dam. In the mid-section of the dam, where the maximum height occurs, the embankments on the upstream and downstream sides of the core wall are composed of hydraulic fill and dumped rockfill, respectively.

Construction History

Crane Valley Dam was initially constructed in 1901 by the San Joaquin Light and Power Corporation. The original dam, located across Willow Creek in the deepest part of the valley, was constructed of hydraulic fill to a crest elevation of approximately 3,315 feet. Figure 2 shows a photo of the original dam embankment.

Fig. 2. Original (1901) Dam Embankment.

Between 1909 and 1911, the dam embankment was enlarged to its present alignment and general configuration. The enlargement involved a downstream raise that was accomplished by constructing a thin, central concrete core wall and raising the crest of the dam to an elevation of approximately 3,378 feet. The original dam was incorporated into the new hydraulic fill and rockfill embankment. The enlarged embankment extended from the deepest part of the valley over a rocky knoll and across a smaller tributary valley located west of Willow Creek. Figure 3 shows a photo of the upstream side of the enlarged dam during construction.

Fig. 3. Enlarged (1911) Dam Embankment.

Embankment Configuration

In its current configuration, Crane Valley Dam consists of four distinct components: the main dam, the east abutment of the main dam (i.e., the east dam), the transition section between the main dam and the west dam (i.e., the transition section), and the west dam. Figure 4 shows a plan view of the dam, with a stationing line along the crest for reference.

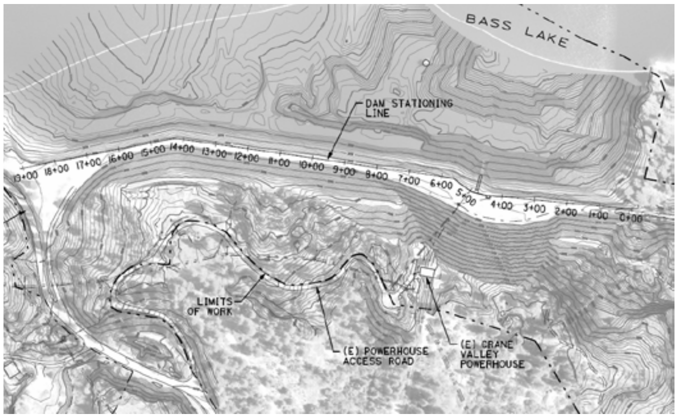

Fig. 4. Crane Valley Dam, Plan View.

The main dam is located across the deepest part of the valley, over the original Willow Creek channel (approximately Stations 2+00 to 7+50 on Fig. 4). The main dam is composed of hydraulic fill (including the original 1901 dam embankment) on the upstream side of a reinforced concrete core wall. The downstream side of the core wall is supported by dumped rockfill. Laterally, the downstream rockfill is separated from the full hydraulic fill sections of the embankment by hand-placed rock walls (now buried under rockfill), which mark the east and west limits of the main dam. The east dam is located between the main dam and the eastern side of the valley (approximately Stations 0+00 to 2+00 on Fig. 4). This section of the dam is composed of hydraulic fill on both sides of the concrete core wall.

The transition section is located between the main dam and the crest of the rocky knoll between the Willow Creek channel and the tributary valley to the west (approximately Stations 7+50 to 11+00 on Fig. 4). This section of the dam is composed of hydraulic fill on both sides of the concrete core wall, which is unreinforced west of the main dam. The west dam crosses the tributary valley (approximately Stations 11+00 to 18+80 on Fig. 4) and is also composed of hydraulic fill on both sides of the unreinforced concrete core wall.

Post-1911 Modifications and Improvements

Over a period of seven decades following construction of the enlarged dam embankment, additional rockfill materials were placed on the upstream and downstream slopes of the dam. Between 1914 and 1929, and in response to downstream movement of the core wall, several stages of rockfill were progressively added to the downstream slope of the main dam. The quarry-run rockfill material was generally placed by side-dumping from the downstream edge of the crest.

In 1970 and 1980, rockfill buttresses were constructed on the lower slopes of the west dam, the transition section, and the east dam. These buttresses, intended to reinforce the dam embankment against potential seismic deformations, were constructed on the downstream side of the dam in 1970 and on the upstream side of the dam in 1980.

In its current configuration, Crane Valley Dam is about 1,880 feet long, with crest elevations that range between approximately 3,380 and 3,382 feet. The main dam has a maximum height of about 145 feet over the deepest part of the valley. The maximum height of the west dam is about 55 feet. A paved roadway is located over the entire length of the dam crest, which ranges from about 20 to 100 feet in width. As shown on Fig. 1, the spillway structure, located about 500 feet east of the dam, has two radial gates and fourteen flashboard bays. Water released from the spillway structure flows down a 900-foot-long channel and into the North Fork of Willow Creek downstream of the dam.

SEISMIC STABILITY EVALUATION

In 2005 and 2006, AMEC performed a series of analyses to evaluate the seismic stability and performance of Crane Valley Dam. The evaluation included the development of design earthquake ground motions, implementation of field investigation and laboratory testing programs, and selection of static and dynamic material properties for use in the stability analyses (Makdisi et al., 2011). The data collected and analyses performed for the 2005 and 2006 seismic stability evaluation established the basis for subsequent analyses to support the seismic design of the retrofitted dam embankment.

The primary concern for performance of the dam during the postulated earthquake ground motions was the potential for liquefaction of the hydraulic fill material in the embankment and subsequent deformation and instability of the dam slopes. To address this concern, the 2005 and 2006 seismic stability evaluation utilized modern, state-of-practice procedures and updated earthquake ground motions for the dam site. Using dynamic finite-element methods to analyze representative cross sections of the dam, AMEC estimated earthquake-induced accelerations and stresses within the dam embankment. The cyclic resistance of the embankment material was estimated using standard penetration test results from recent and previous field investigations.

AMEC evaluated the seismic stability of the dam under loading from earthquake ground shaking caused by a local (random) event represented by a magnitude 6¼ earthquake at a distance of about 15 km from the site, and a distant event represented by a magnitude 8 earthquake on the San Andreas fault, at a distance of about 180 km from the site. Peak ground accelerations for the local and distant events were estimated to be about 0.2 and 0.07 g, respectively (Makdisi et al., 2011).

The results of the 2005 and 2006 seismic stability analyses indicated that the slopes of hydraulic fill sections of the dam would likely liquefy and become unstable during (or after) the earthquake shaking postulated for the site. Based on these results, it was concluded that the hydraulic fill embankments would experience large deformations that would likely result in an uncontrolled release of the reservoir water at the normal maximum operating level. This finding applied to the upstream and downstream slopes of modeled sections through the east dam, the transition section, and the west dam. Accordingly, AMEC recommended that retrofit measures be considered for improving the stability of the upstream and downstream slopes of these sections.

The results of the 2005 and 2006 analyses also indicated that portions of the upstream main dam embankment likely would liquefy and become unstable during and after earthquake shaking, resulting in as much as 5 to 10 feet of potential slumping of the embankment slope on the upstream side of the dam's concrete core wall. However, analyses performed by PG&E (PG&E, 2008) indicated that the core wall would remain in place because of the stabilizing effect of reservoir water pressure against the wall. This condition, together with the significant width of the dam crest and the downstream rockfill shell, was judged to provide a stable section that would prevent an uncontrolled release of reservoir water. Based on these analyses, it was concluded that remediation of the upstream main dam embankment section was not required.

The Board of Consultants (BOC) appointed by PG&E to review the project, the DSOD, and the Federal Energy Regulatory Commission (FERC) concurred with the findings of the 2005 and 2006 seismic stability evaluation. However, the DSOD requested, as a prudent measure, that PG&E consider alternatives to retrofit the upper portion of the main dam core wall to mitigate the potential for toppling due to slumping of the liquefied hydraulic fill slope upstream of the wall. The DSOD also requested that the crest of the existing dam be raised to provide a minimum freeboard of 10 feet above the normal maximum water level in the reservoir.

In response to the findings of the seismic stability evaluation, PG&E implemented a temporary 10-foot restriction on the normal maximum allowable water level in Bass Lake. Instead of a maximum water surface elevation of 3,376.76 feet, which is accomplished using radial gates and flashboards across the spillway channel, PG&E established a restricted maximum water surface elevation of 3,366.76 feet (i.e., about 0.7 foot above the elevation of the spillway crest) until completion of seismic retrofit measures. This restricted level reduces the capacity of the reservoir by about 25 percent (i.e., about 10,000 acre-feet) and maintains a minimum of 14 feet of freeboard between the water surface and existing dam crest.

RETROFIT OBJECTIVES AND DESIGN CRITERIA

The objective of the Crane Valley Dam Seismic Retrofit Project is to strengthen the existing dam embankment such that it can withstand, with acceptable deformations, the shaking generated by the postulated earthquakes, thereby preventing an uncontrolled release of the reservoir water and protecting public safety. To accomplish this objective, the following design elements and criteria were specifically developed for the Project:

1. Increase the height of dam crest by about 5 to 7 feet (i.e., to a finished elevation of about 3,387 ft) to provide a minimum 10 feet of freeboard above the normal maximum water surface elevation in the reservoir.
2. Construct filter and drain zones to mitigate the potential for internal erosion (piping) of hydraulic fill materials in the dam embankment.
3. For the design ground motions developed for local and distant earthquakes as part of the 2006 seismic stability evaluation, maintain a minimum calculated post-earthquake factor of safety (FS) of 1.25, and a maximum of 2 feet of estimated seismically-induced permanent, deformation for the upstream and downstream slopes of the east dam, the transition section, and the west dam.
4. Retrofit the main dam core wall to accommodate as much as 10 feet of slumping of the upstream embankment slope.
5. Upgrade existing monitoring instruments and systems (i.e., weirs, piezometers, and settlement monuments) for measuring the performance of the dam.

DESIGN CONSIDERATIONS AND CONSTRAINTS

In addition to the criteria described above, remedial design alternatives were subject to a variety of physical, operational, and environmental constraints. These constraints affected the selection of potential retrofit approaches and required the inclusion of several key elements within the Project's design.

Physical Constraints

Concerns for site access and protection of existing facilities were key considerations in evaluating potential retrofit approaches. As shown in Fig. 1, the site is heavily forested and includes areas of steep and rocky terrain. Because of difficult access and environmental concerns, some areas were not accessible to subsurface exploration equipment. Flexible design approaches were necessary to accommodate the limited availability of subsurface information and the potential for discrepancies between design assumptions and the foundation conditions encountered during construction.

Transportation of construction equipment and materials to and from the site also was a factor in the evaluation and selection of potential design alternatives. The dam is located in a remote area and is accessed by a narrow and winding two-lane county road that also serves local residents and recreational users. Because of concerns for public safety and transportation-related costs and environmental impacts, PG&E favored retrofit approaches that utilized on-site resources and limited the amount of waste material requiring offsite disposal.

Potential impacts to existing facilities, including the dam's intake tower, penstock, outlet works, spillway, powerhouse, and associated control structures, also were considered during the development and evaluation of potential retrofit alternatives. For example, modifications to the penstock, powerhouse, and outlet structures located near the downstream toe of the main dam were not allowed as part of the Project. Measures to avoid and/or protect these critical facilities were incorporated into the selected retrofit approach.

<u>Reservoir Operations</u>

Bass Lake is a popular vacation destination and the local economy is heavily dependent on use of the lake for fishing, boating, and other recreational activities. The lake also serves as a municipal water supply for nearby communities. Reservoir operations are controlled by these factors, as well as power generation objectives, environmental requirements, and agreements with downstream water users. Discharges from the lake are limited by the capacity of the dam's low-level outlet, which affects PG&E's ability to manage lake levels below the spillway elevation during periods of high inflow.

Because of these operational constraints, draining the lake and/or constructing a cofferdam were not considered to be viable alternatives for constructing improvements to the dam. As a result, implementation of remedial measures would need to occur while the dam remained in service. To address these constraints, the selected retrofit approach had to accommodate the lake level schedule summarized in Table 1.

Table 1. Anticipated Lake Levels during Project Construction

Operating Period	Anticipated Lake Level
Late May through August	At or near restricted maximum elevation of 3,366.67 feet (14 ft. below crest, 75% of maximum pool)
September through October	Fall drawdown to target minimum elevation of 3,348 feet (33 ft. below crest, 40% of maximum pool)
November through mid-December	Between target minimum and absolute minimum elevation of 3,345 feet (36 ft. below crest, 35% of maximum pool)
Mid-December through late May	Winter/spring filling to restricted maximum elevation (actual schedule dependent on weather and runoff)

As indicated by the anticipated lake level schedule, retrofit measures located on the upstream side of the dam below an elevation of about 3,348 feet would require wet (i.e., underwater) construction.

<u>Environmental Considerations</u>

Crane Valley Dam is located in an environmentally sensitive area. On the upstream side of the dam, Bass Lake supports fish populations and other aquatic and terrestrial species. On the downstream side of the dam, seepage and discharges from the reservoir support wetland and riparian habitats near the base of the dam and along the banks of Willow Creek. To reduce potential impacts to these habitats, retrofit measures and approaches were developed to limit the Project footprint along the downstream toe of the dam.

<u>Schedule</u>

Because of concerns for public safety and the need to protect critical hydroelectric and flood management infrastructure from seismic hazards, PG&E established an aggressive schedule for design and construction of the Project. In developing and selecting retrofit alternatives, PG&E considered year-round (i.e., four-season) construction to be a necessary part of the effort to meet these schedule demands.

RETROFIT APPROACH

After developing and evaluating a variety of alternatives, PG&E selected a retrofit approach involving the construction of new rockfill buttresses on the upstream and downstream sides of the dam. This approach was supported by geologic mapping and the results of preliminary exploration and siting studies, which suggested that a suitable source of rockfill could be developed locally, as part of the Project and within about ½ mile of the dam. Using a local source of rockfill would reduce potential traffic impacts on local roads, as well as transportation-related Project costs. However, if necessary and/or economical, imported rock and soil products could be used to supplement the material from on-site sources.

To limit the footprint of the downstream buttresses and accommodate underwater placement of rockfill on the upstream side of the dam, "clean" rockfill (i.e., rockfill containing less than 5 percent sand-size and smaller particles, measured by weight) was selected for construction of the buttresses. The strength and density characteristics of clean, well-compacted rockfill allow the downstream buttresses to be smaller than would otherwise be required for "dirty" rockfill or soil materials. For upstream construction, clean and coarse rockfill materials are less likely to segregate and develop zones of potential weakness when placed through water.

Figures 5 through 8 show buttress configurations for the transition section (Section A-A′), main dam (Section B-B′), west dam (Section C-C′), and east dam (Section D-D′), respectively. Figure 9 shows a plan view of the proposed buttresses. On the upstream side of the main dam (Section B-B′, Fig. 6), a rockfill buttress was not required, as strengthening the existing core wall is considered sufficient to accommodate deformation of the embankment slope and achieve the stated design objective described above.

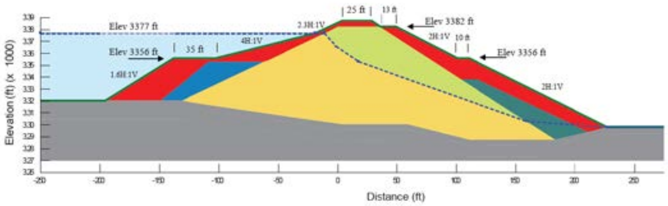

Fig. 5. Buttress Configuration at Transition Section (Cross Section A-A′).

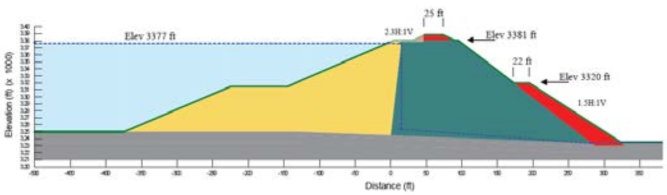

Fig. 6. Buttress Configuration at Main Dam (Cross Section B-B′).

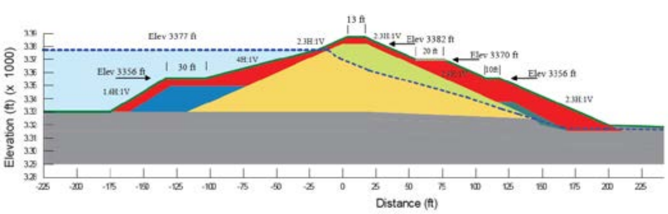

Fig. 7. Buttress Configuration at West Dam (Cross Section C-C′).

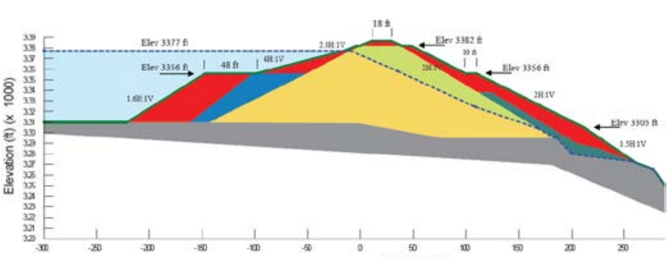

Fig. 8. Buttress Configuration at East Dam (Cross Section D-D′).

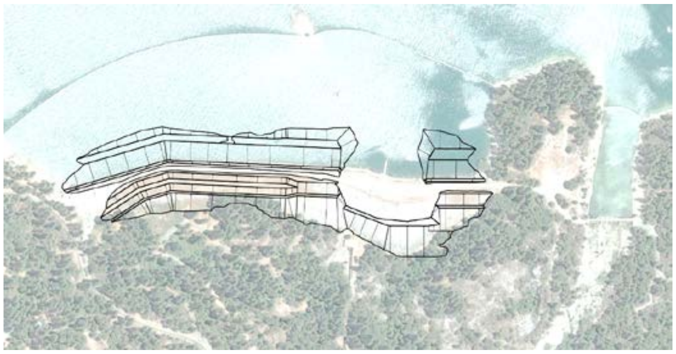

Fig. 9. Buttress Layout, Plan View.

As shown on Figs. 5 through 8, the selected retrofit approach includes placing earthfill and rockfill materials to raise the crest of the dam to a minimum elevation of 3,387 feet. Although not shown in Figs. 5 through 8, the selected retrofit approach also includes chimney and blanket filters and drains over the downstream hydraulic fill and excavated foundation surfaces beneath the east dam, transition section, and west dam buttresses. The filters and drains are connected to a subdrainage collection system that runs beneath all of the downstream buttresses. These drainage improvements are intended to control seepage within the retrofitted embankment and mitigate the potential for internal erosion (piping) of the hydraulic fill and buttress foundation materials.

STABILITY AND DEFORMATION ANALYSES

AMEC performed seismic stability and deformation analyses to evaluate the performance of the retrofitted dam embankment. In general, the procedure used to perform the seismic stability evaluation included the following steps:

1. Calculating the pre-earthquake static stresses within the embankment using static finite-element procedures.
2. Calculating earthquake-induced accelerations and shear stresses within the embankment and foundation using dynamic finite-element procedures.
3. Estimating the cyclic resistance of the embankment and foundation materials and evaluating the potential for liquefaction.
4. Establishing undrained shear strength parameters for embankment and foundation materials and undrained residual strength parameters for zones estimated to have liquefied during or after earthquake shaking.
5. Evaluating the post-liquefaction stability of the embankment using the undrained and residual strengths established in Step 4 above.
6. Estimating the magnitude of permanent deformation using results of the dynamic and stability analyses performed in Steps 2 through 5.

Seismic stability and deformation analyses were performed for four embankment sections, designated A-A′, B-B′, C-C', and D-D′, representing the retrofitted configurations of the transition section, main dam, west dam, and east dam embankments, respectively. A normal maximum reservoir elevation of 3,377 feet was assumed for the analyses.

Earthquake Ground Motions

AMEC evaluated the seismic stability of proposed modifications to Crane Valley Dam using the same scenario earthquakes that were used for the 2006 seismic stability evaluation (Makdisi et al., 2011). These scenario earthquakes were initially developed based on the results of a probabilistic seismic hazard analysis (PSHA) performed by PG&E to estimate ground motions with a return period of 1,500 years. However, based on recommendations from the BOC, PG&E also developed design ground motions based on a deterministic analysis of two controlling events similar to the scenario earthquakes identified during the PSHA.

For the deterministic analysis, the local controlling event was represented by a magnitude 6¼ earthquake at a distance of about 15 km from the site, with ground motions estimated at the median level; the distant controlling event was represented by a magnitude 8 earthquake at a distance of about 170 to 180 km, with ground motions estimated at the 84th percentile level (Idriss, 2005). The median-level ground motions were specified for the local event because of its relatively low recurrence rate. PGAs for the local and distant events were estimated to be about 0.2 g and 0.07 g, respectively.

After reviewing and comparing the characteristics of the probabilistic and deterministic ground motions, PG&E adopted the deterministic response spectra for design of the Project. For each of the deterministic scenario earthquakes, PG&E selected a recorded acceleration time history and modified it such that its response spectrum matched the target design spectrum. These time histories were used as input motions for dynamic response analyses of the retrofitted dam embankment.

Static Stress Analyses

To estimate the static, pre-earthquake distribution of stresses within the dam embankment, AMEC analyzed representative sections of the embankments using the two-dimensional, plane-strain, finite-element program FEADAM84 (Duncan et al., 1984). Static stress analyses were performed for cross sections A-A′, B-B′, and C-C′. Finite-element representations of the cross sections were developed with appropriate embankment zonation and a representative phreatic surface based on piezometer measurements provided by PG&E. As a conservative approach, the dam's concrete core wall was not modeled for the analysis. Seepage forces (if any) were considered negligible and also were not included in the analysis. Figure 9 shows the finite-element mesh developed for Section C-C′.

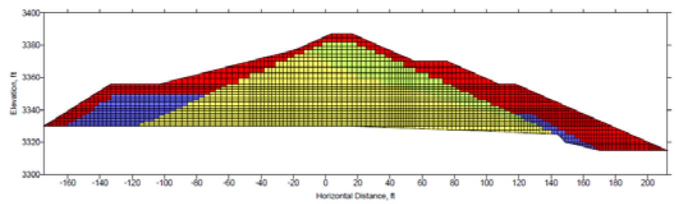

Fig. 9. Representative Finite-Element Mesh of Section C-C′ used for Static and Dynamic Stress Analyses.

The static analyses were performed by simulating construction of the embankment in layers. The nonlinear, stress-dependent stress-strain and volumetric strain properties of the embankment materials were approximated using a hyperbolic model developed by Duncan and Chang (1970) and modified by Duncan et al. (1980). Gravity loads were applied to simulate raising the embankment. For each layer, buoyant unit weights were used for the materials below and moist unit weights were used for the materials above the phreatic surface.

The effective normal stresses (σ_y and σ_x) calculated by FEADAM84 were used to compute the initial mean confining pressure for estimating the dynamic shear modulus at low strain, G_{max}. In the liquefaction assessment, the initial vertical stress (σ_y) was used, together with the dynamic induced peak shear stresses, to estimate the earthquake-induced stress ratio within the embankment. The initial vertical stress and the shear to vertical stress ratio (α) were used to make corrections (where appropriate) to the cyclic strength of the embankment soils to account for the effects of confining pressure (the K_σ effect) and sloping ground conditions (the K_α effect).

Dynamic Response Analyses

AMEC performed dynamic finite-element response analyses to assess the earthquake-induced stresses and accelerations within the retrofitted dam embankment and foundation. The finite element meshes used for the static stress analyses (e.g., Fig. 9 for Section C-C′) also were used for the dynamic response analyses. Input motions for the dynamic response analyses were applied in the transverse (upstream-downstream) direction as outcropping motions at the contact between the embankment and the underlying bedrock.

The program QUAD4M (Hudson et al., 1994) was used to compute the response of the embankment to the design earthquake ground motions. QUAD4M, an application for performing two-dimensional, dynamic, finite-element analyses, uses equivalent-linear, strain-dependent modulus and damping properties. The time domain analysis uses Rayleigh damping and allows variable damping for individual elements.

Non-linear, strain-dependent properties are estimated by the program using an iterative process.

Material properties required for performing dynamic response analyses using the equivalent-linear approach are the moist and saturated total unit weight (γ_{moist} and γ_{sat}, respectively), moist and saturated Poisson's ratio (ν_{moist} and ν_{sat}, respectively), dynamic shear modulus at low strain (G_{max}), and the relationships of the modulus reduction factor and damping ratio with shear strain. Table 2 lists the dynamic soil properties used in the finite-element analyses. Table 3 lists the relationships of the modulus reduction factor and damping ratio with shear strain used in the analyses.

Table 2. Material Properties for Dynamic Analyses

Material	γ_{moist} (pcf)	γ_{sat} (pcf)	G_{max}	ν_{moist}	ν_{sat}
Hydraulic Fill	115	120	41 (A-A') 47 (B-B') 36 (C-C') 41 (D-D')	0.33	0.47
Downstream Rockfill	135	--	90	0.33	N/A
Upstream Rockfill	--	140	90	N/A	0.47

Table 3. Dynamic Modulus and Damping Relationships

Material	Modulus Reduction Relationship	Damping Ratio Relationship
Hydraulic Fill	Darendeli (2001) 2 atm for A-A', B-B', and D-D' 1 atm for C-C'	EPRI (1993) 20'-50' for A-A', B-B', and D-D' 0'-20' for C-C'
Rockfill	Seed et al (1986) Upper bound for gravels	Seed et al (1986) Upper bound for gravels

Dynamic finite-element analysis of the embankment also requires the shear wave velocity of the bedrock underlying the embankment. The bedrock was modeled as an elastic half-space having an assumed shear wave velocity of 2,000 feet per second (fps), and a compression shear wave velocity of 3,464 fps (assuming a value of Poisson's ratio of 0.25).

The seismic response of the analyzed embankment sections was computed using the design input motions described above: M_w 6¼ with a PGA of 0.20g, and M_w 8.0 with a PGA of 0.07g. As an example, Fig. 10 shows the dynamic response results for peak horizontal acceleration for Section C-C′ and the M_w 6¼, 0.20g PGA input motion.

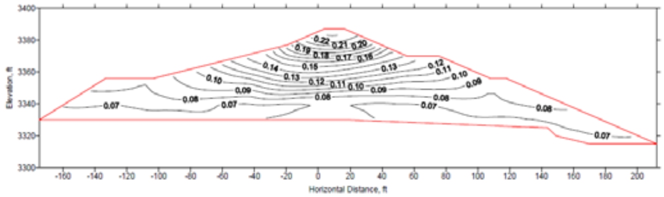

Fig. 10. Contours of Peak Horizontal Acceleration in g's, Section C-C′, Mw 8, 0.07g PGA Input Motion.

For the magnitude 6¼ outcrop rock motion (PGA of 0.20g), the computed maximum accelerations at the crest of the dam were about 0.50g for Sections A-A′ and D-D', 0.27g for Section B-B′, and 0.47g for Section C-C′. For the magnitude 8.0 outcrop rock motion (PGA of 0.07g), the computed maximum accelerations at the crest were about 0.23g for Sections A-A′, C-C', and D-D', and 0.19g for Section B-B'.

In addition to peak horizontal acceleration, the results of the dynamic response analyses provided earthquake-induced shear stresses that were compared with the cyclic strength to assess the potential for liquefaction of the embankment materials. The analyses also provided time histories of earthquake-induced accelerations ("seismic coefficient" time-histories) for potential sliding surfaces within the embankment slopes. These time histories were used to estimate earthquake-induced slope deformations.

Liquefaction Evaluation

AMEC evaluated the potential for liquefaction of hydraulic fill materials within the retrofitted dam embankment in accordance with the same methods and procedures established for the 2006 seismic stability evaluation (Makdisi et al., 2011). These methods and procedures were based on the approach of Seed and Idriss (1982) and Seed et al. (1985), as updated in Youd et al. (2001). Using this approach, liquefaction potential was evaluated by comparing the earthquake-induced cyclic shear stress ratio (CSR, obtained from the dynamic response analyses) with the cyclic strength (or cyclic resistance ratio, CRR) of the hydraulic embankment fill. The CRR is defined as the uniform cyclic stress ratio required to cause liquefaction for a given earthquake magnitude.

To develop CRRs for the liquefaction evaluation, AMEC used the magnitude-scaling factors (MSFs) and the cyclic resistance curve for clean sands developed by Seed and Idriss (1982) and updated in Youd et al. (2001). To account for the fines content of the hydraulic fill materials, AMEC used fines correction factors developed by Idriss and Boulanger (2008). To account for overburden stresses (i.e., effective vertical stresses other than 1.0 ton/ft^2) and initial static shear stresses (i.e., sloping ground conditions), AMEC used K_σ correction factors as recommended by Youd, et al. (2001) and K_α correction factors as recommended by Idriss and Boulanger (2008).

In situ standard penetration test (SPT) results were used to evaluate the cyclic strength of the hydraulic fill materials in the dam embankment, which generally range from silty sand to sandy silt. The SPT blowcount data were processed for the entire embankment and separately for each analysis section. Blowcounts were adjusted using clean sand correction factors for both liquefaction triggering and for the determination of residual strength (S_r). The selected mean blowcount values were then used to estimate the cyclic resistance of the embankment sections, which was further adjusted for MSF, $K\sigma$, and $K\alpha$ effects as described above.

To estimate the potential for liquefaction within the saturated zones of the embankment, the earthquake-induced CSR for each soil element was compared with that element's CRR. The factor of safety against triggering of liquefaction is defined as the CRR divided by the average earthquake-induced CSR. For the design earthquake, liquefaction is likely to be triggered in zones having a factor of safety equal to or less than 1.0. As an example, Fig. 11 shows contours of factor of safety against triggering of liquefaction for saturated hydraulic fill portions of the Section C-C′ embankment for the magnitude 8 event. The estimated factors of safety are less than 1.0 for the entire hydraulic fill portion of the upstream embankment, and approximately half of the saturated hydraulic fill portion of the downstream embankment.

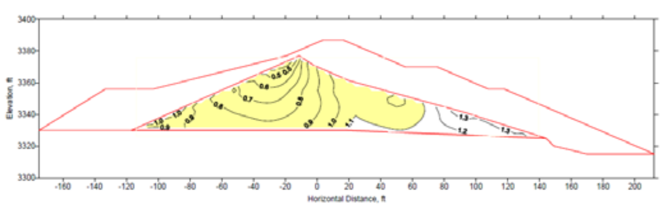

Fig. 11. Contours of Factor of Safety Against Triggering of Liquefaction, Section C-C′, Mw 8, 0.07g Input Motion.

Evaluation of Post-Earthquake Stability

AMEC performed slope stability analyses for conditions during or immediately following earthquake shaking for Sections A-A′, B-B′, C-C′ and D-D′ using the computer program SLOPE/W V7.14 (GEO-SLOPE, 2007) and Spencer's method for computing factors of safety. The effects of liquefaction induced by earthquake shaking were incorporated into the stability analyses by using undrained residual strength (S_r) values for zones within the embankment for which liquefaction is predicted. To account for the effects of excess pore pressure buildup, reduced strength values were assigned to zones that did not fully liquefy during the postulated earthquake shaking (Marcuson, et al., 1989).

The S_r values selected for use in the design stability analyses were based on published correlations with penetration resistance developed by Seed and Harder (1990). Because of uncertainties regarding the onset of liquefaction, and the potential for the development of residual strength at pore pressure ratios less than 1.0, undrained residual strengths were assigned to the hydraulic fill embankment materials having a computed factor of safety against liquefaction less than 1.1. As a lower limit, the residual strengths were constrained to be no greater than what would be predicted using the soil's effective friction angle. Table 4 summarizes the undrained residual strength parameters assigned to saturated hydraulic fill materials within the dam embankment.

Table 4. Undrained Residual Strength Parameters, Hydraulic Fill Materials

Material	Residual Strength (S_r, psf)	ϕ[1] (deg.)	Notes
Hydraulic Fill (FS $\leq$ 1.1)	440 [$(N_1)_{60\text{-}cs}$=13] 240 [$(N_1)_{60\text{-}cs}$=10] 240 [$(N_1)_{60\text{-}cs}$=10] 140 [$(N_1)_{60\text{-}cs}$=8]	--	B-B' A-A' D-D' C-C'
Hydraulic Fill (1.1 < FS $\leq$ 1.4)	0	25.3	Bi-linear, $\geq S_r$
Hydraulic Fill (1.4 < FS $\leq$ 2.0)	0	26.8	

[1] Reduced friction angle as a result of excess pore pressure buildup, defined by: $\tan\phi = (1\text{-}R_u)\tan\phi'$ [ϕ' = 34 degrees]

Drained strength parameters were assigned to hydraulic fill materials above the phreatic surface, as well as buttress rockfill and foundation materials. Table 5 summarizes the effective stress shear strength parameters developed for the post-earthquake stability analyses.

Table 5. Effective Stress Shear Strength Parameters, Buttress and Foundation Materials

Material	c' (psf)	ϕ' (deg.)
Hydraulic Fill, moist	0	34
Upstream Rockfill	0	45
Downstream Rockfill	0	45
Interface Layer (beneath upstream rockfill, Section D-D')	0	38
Residual Soil	0	35
Weathered Bedrock	0	40

After analyzing the subsurface conditions encountered in borings, CPTs, and test pits located in the west dam and transition section areas, AMEC concluded that foundation materials beneath the existing dam embankment and rockfill buttresses represented by Sections A-A' and C-C' likely consist of a layer of residual soil underlain by severely to moderately weathered bedrock. For the stability evaluations of these sections, the foundation profile included a 5-foot-thick layer of residual soil underlain by a 10-foot-thick layer of severely weathered bedrock. Relatively competent rock was assumed to underlie the severely weathered bedrock.

Borings performed near the east dam in the area represented by Section D-D′, did not encounter the residual soil layer that was observed in areas west of the main dam. However, to account for the possibility that lake bottom sediments beneath the existing upstream buttress were not fully displaced by the buttress rockfill, a silty "interface layer" was included beneath the buttress rockfill in Section D-D'. The layer, representing a mixture of rockfill, silt, and sand, was assigned a strength that is lower than the friction angle of the rockfill alone.

Presumably, foundation conditions similar to those modeled for Sections A-A', C-C', and D-D' would exist beneath the new upstream rockfill buttresses if the buttresses were placed without first removing the lake bottom sediments and residual soils. Stability analyses of the upstream slopes of the dam were performed making this conservative assumption. The increased strength at the rockfill/foundation interface that would result from the removal (i.e., dredging) of the lake bottom sediments and residual soils was ignored.

AMEC performed post-earthquake stability analyses of the retrofitted upstream and downstream slopes of Sections A-A′, C-C′, and D-D′ and the upstream slope of Section B-B' considering a maximum reservoir level at elevation 3,377 feet. AMEC also performed a long-term (static) stability analysis of the buttressed downstream slope of the main dam rockfill embankment (Section B-B'). These analyses were performed as part of an iterative process to develop buttress configurations meeting the established design criteria for post-earthquake factor of safety and maximum deformations. For the final design buttress configurations, all of the analyzed sections meet the established design criteria. Figure 12 shows the critical slip surfaces identified for the final design buttress configuration at Section C-C' under the distant magnitude 8 design earthquake scenario.

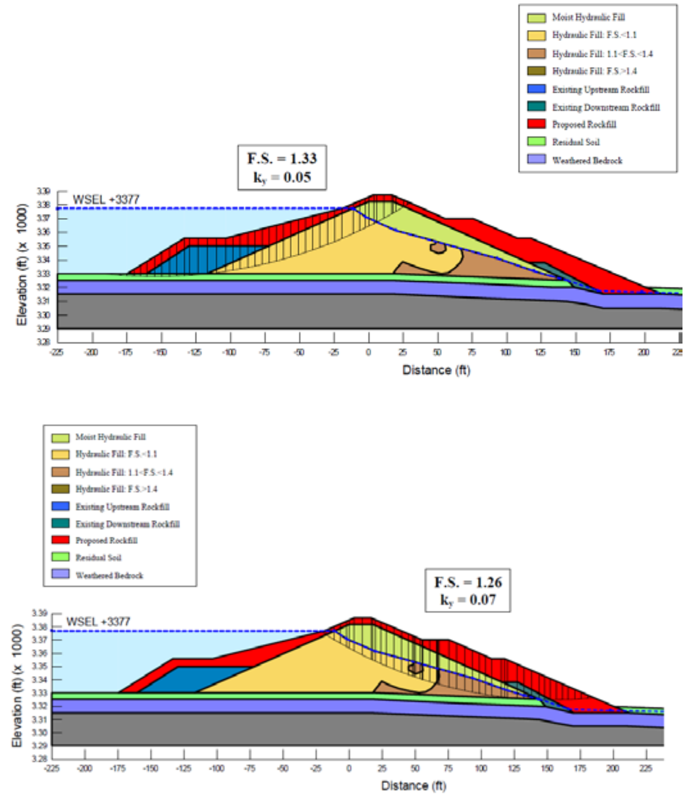

Fig. 12. Critical Slip Surfaces on Upstream (top) and Downstream (bottom) Slopes, Section C-C′, Mw 8, 0.07g Input Motion.

Earthquake-Induced Deformations.

When analyses of post-liquefaction stability indicate that the slopes of an embankment will remain stable, it is desirable to estimate the permanent earthquake-induced deformations. In the current state-of-practice procedures, permanent deformations are estimated using the yield acceleration concept proposed by Newmark (1965) and modified by Makdisi and Seed (1978). The procedure used to estimate permanent deformation is comprised of the following steps:

1. A yield acceleration, k_y, at which a potential sliding surface would develop a factor of safety of one, is determined using limit-equilibrium, pseudo-static slope stability methods.
2. The peak, or maximum, acceleration, k_{max}, induced within a potential sliding mass (peak value of a seismic coefficient time history) is estimated using a dynamic response analysis.
3. For a specified potential sliding mass, the seismic coefficient time history of that mass is compared with the yield acceleration, k_y. When the seismic coefficient exceeds the yield acceleration, downslope movement will occur along the direction of the assumed failure plane. The magnitude of the accumulated permanent

displacement is calculated by double-integrating the seismic coefficient time history above the yield acceleration.

AMEC used the procedures described above to estimate the permanent, earthquake-induced deformations of the retrofitted dam embankment slopes. Tables 6 and 7 summarize the complete evaluation of all four design sections subjected to the local and distant scenario earthquakes, respectively.

Table 6. Stability Analysis Results – Local Magnitude 6¼ (0.20g) Earthquake Scenario

Sec-tion	Slip Surface	FS	k_y (g)	k_y / k_{max}	Displace-ment (ft)
	u/s shallow	1.85	0.17	0.71	< 0.1
A-A’	u/s deep	2.05	0.17	> 1	Negligible
	d/s deep	1.44	0.17	>1	Negligible
B-B’	u/s shallow	1.26	0.06	0.35	0.6
	u/s shallow	1.32	0.07	0.35	0.5
C-C’	u/s deep	1.34	0.05	0.45	0.2
	d/s deep	1.90	0.26	> 1	Negligible
	u/s shallow	1.75	0.20	0.83	< 0.1
D-D’	u/s deep	2.24	0.19	> 1	Negligible
	d/s deep	1.40	0.16	> 1	Negligible

Table 6. Stability Analysis Results – Distant Magnitude 8 (0.07g) Earthquake Scenario

Sec-tion	Slip Surface	FS	k_y (g)	k_y / k_{max}	Displace-ment (ft)
	u/s shallow	1.69	0.11	0.92	< 0.1
A-A’	u/s deep	1.37	0.06	0.75	< 0.1
	d/s deep	1.37	0.14	>1	Negligible
B-B’	u/s shallow	0.92	n/a	n/a	Unstable
	u/s shallow	1.28	0.04	0.36	0.2
C-C’	u/s deep	1.33	0.05	0.63	< 0.1
	d/s deep	1.37	0.07	0.88	< 0.1
	u/s shallow	1.59	0.10	0.91	< 0.1
D-D’	u/s deep	1.25	0.04	0.50	< 0.1
	d/s deep	1.30	0.10	> 1	Negligible

Because the calculated factor of safety for the upstream embankment slope at Section B-B’ was less than 1.0 (indicating an unstable condition), pseudo-static methods were not used to estimate the permanent deformation of the slope for the distant earthquake scenario. For this case, AMEC estimated the deformation of the slope using the finite difference software FLAC (Itasca, 2005). Results of the slumping analysis, reported as part of the 2005 and 2006 seismic stability evaluation (Makdisi et al., 2011), indicated maximum deformations of about 6 to 7 feet within the upstream hydraulic fill embankment and about 5 feet of settlement at the crest of the slope as a result of earthquake-induced ground shaking and liquefaction.

<u>Sensitivity and Parametric Studies</u>

At the request of PG&E’s BOC, a sensitivity study was performed to evaluate the effect of other residual strength relationships developed by Idriss and Boulanger (2007) on the results of the stability analyses. These relationships are expressed in terms of the residual shear strength ratio, which is defined as the ratio of the undrained residual strength to the initial effective vertical stress prior to the earthquake. It should be noted that for relatively shallow embankments and low confining pressures, such as those analyzed for this project, the use of the strength ratio may not be applicable. However, the results of stability analyses performed using the Idriss and Boulanger (2007) relationships were compared with those obtained using the Seed and Harder (1990) residual strength values. In general, similar results were obtained from the two methodologies.

As part of a sensitivity analysis to evaluate the effect of selected time histories on the estimated seismic stability and earthquake-induced deformation of the embankments, PG&E developed one additional time history for each scenario earthquake. These time histories were selected to provide an average response of embankment deformation when compared to a range of estimated responses from a large number of time histories. AMEC analyzed Section C-C′ for the two additional ground motions provided by PG&E and found the results to be similar to those of the design analyses (AMEC, 2010).

In response to comments from the DSOD, AMEC analyzed Section C-C’ using a moist and saturated unit weight of about 125 pounds per cubic foot (pcf) for the buttress rockfill. This value is consistent with void ratios of about 0.7 for submerged rockfill on the upstream side of the dam and about 0.4 for compacted rockfill on the downstream side of the dam. Moist and saturated unit weights of 125 pcf are understood to represent a conservative lower bound for the buttress rockfill materials. Results of the unit weight sensitivity analyses suggest a 1 to 8 percent reduction in the calculated factor of safety and up to twice as much displacement than estimated under the design assumptions. However, the estimated displacements are still equal to or less than 1 foot and are within acceptable limits.

OTHER ANALYSES

In addition to the seismic slope stability analyses described above, AMEC performed static stability and seepage analyses to evaluate temporary construction conditions during the excavation of buttress foundation areas on the upstream and downstream sides of the dam. Results of these analyses indicated that the required excavations could be performed without adversely affecting the overall stability of the existing dam embankment; however, effective dewatering measures would likely be necessary to prevent seepage, soil migration, and localized instability of the downstream buttress excavation slopes.

AMEC also performed filter compatibility analyses to determine appropriate gradation limits for filter (Zone 3), drain (Zone 4), and light rockfill (Zone 2A) materials within the downstream buttresses. Gradation limits for the Zone 3 and Zone 4 materials were developed considering three conditions: first, the Zone 3 material should adequately filter the underlying hydraulic fill or weathered bedrock foundation materials; second, the Zone 4 material should adequately filter the Zone 3 material and provide the highest flow capacity possible; and third, the Zone 2A rockfill should adequately filter the Zone 4 material.

AMEC used two approaches for selecting filter-compatible materials. The United States Department of Agriculture's methodology (USDA, 1994) was used to develop compatible gradations for hydraulic fill, buttress foundation, Zone 3, and Zone 4 materials; and the DSOD's methodology (DSOD, 1986) was used to develop compatible gradations for Zone 4 and Zone 2A rockfill materials. Figure 13 shows example results from the Zone 3 filter analysis and recommended gradation limits, which correspond to the requirements for ASTM C33 Concrete Sand.

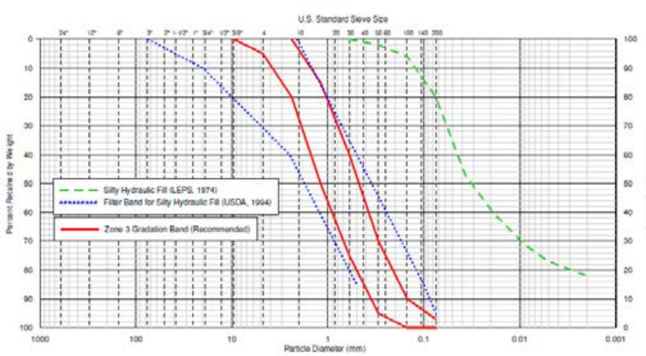

Fig. 13. Filter Analysis Results and Recommended Zone 3 Gradation Limits.

RETROFIT DESIGN

The final design for the Crane Valley Dam Seismic Retrofit Project includes the following major elements:

- Constructing an on-site quarry and processing plant to produce sand, gravel, and rockfill materials for the Project.
- Installing a temporary dewatering system (shallow ejector wells and deep pumping wells) to control groundwater and seepage during excavation of downstream west dam buttress foundation.
- Constructing a permanent concrete "bridge" structure to protect the existing penstock from construction and buttress loads.
- Removing the existing "1970s" rockfill buttresses to expose the downstream hydraulic fill slopes of the enlarged (1909-1911) dam embankment.
- Excavating approximately 35,000 cubic yards (CY) of existing rockfill and unsuitable buttress foundation material from areas near the downstream toe of the existing dam embankment.
- Placing approximately 15,000 CY of filter and 25,000 CY of drain material over the existing hydraulic fill slopes and prepared buttress foundation surfaces on the downstream side of the dam.
- Installing a subdrainage collection system along the downstream toe of the dam.
- Placing approximately 100,000 CY of rockfill to construct new buttresses on the downstream slopes of the dam.
- Dredging approximately 50,000 CY of sediment and unsuitable soil from buttress foundation areas on the upstream side of the dam.
- Placing approximately 125,000 cubic yards of rockfill over the existing upstream "1980s" buttresses and on the upper slopes of the dam embankment.
- Installing a concrete block and series of steel anchors beneath the crest of the main dam to strengthen portions of the existing concrete core wall.
- Placing earthfill and rockfill materials to raise the crest of the dam to a completed elevation of about 3,387 feet.
- Installing new dam monitoring instrumentation, including monitoring wells (piezometers), settlement monuments, and weirs.

These design elements, some of which are described in more detail below, meet the stated design criteria and allow the completed Project to accomplish PG&E's retrofit objectives for the existing dam embankment.

<u>West Dam Dewatering System</u>

To mitigate potential hazards related to slope instability and soil migration as a result of groundwater seepage, the Project specifications require groundwater levels to be maintained at least 3 feet below the bottom of all excavations. The potential for groundwater seepage is of particular concern in the downstream west dam area, where groundwater levels at or above the ground surface (i.e., artesian conditions) have been observed in monitoring wells, and year-round seepage flows into a wet area near the toe of the dam.

PG&E retained Mueser Rutledge Consulting Engineers to design a dewatering system for the west dam buttress excavation area. The design included installing 117 ejector wells around the perimeter of the excavation area and 19 ejector wells in two lines within the excavation area (MRCE, 2010). The design also included 5 deep wells extending into sound rock. Figure 14 shows a plan view of the west dam dewatering system layout.

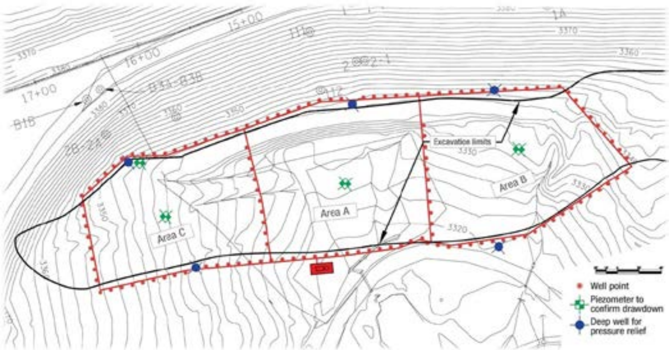

Fig. 14. West Dam Dewatering System Layout (MRCE, 2010).

Downstream Excavation

A stable, competent foundation is required for construction of the downstream rockfill buttresses. As revealed by the borings and test pits used to explore the Project site, bedrock materials underlying the downstream buttress foundations are generally overlain by relatively weak fill and residual soil.

In consultation with PG&E, PG&E's BOC, FERC, and DSOD, AMEC developed criteria for identifying suitable foundation materials beneath the downstream buttresses. In accordance with these criteria, suitable foundation material was defined as severely weathered bedrock or better, where severely weathered bedrock is described as having: "all rock except quartz discolored or stained, with clear and evident rock 'fabric,' but reduced in strength to 'soil' with only fragments of strong rock remaining."

AMEC used descriptions of the various earth materials encountered in exploratory borings and test pits to estimate the thickness of unsuitable foundation materials present near the downstream toe of the dam. The estimated thickness of unsuitable material was used to establish preliminary excavation limits shown on the Project Drawings. To avoid potential impacts to the existing dam embankment, the limits of upstream excavation were designated as the toe of hydraulic fill materials and the excavation slopes were limited to a maximum inclination of 2:1 (Horizontal: Vertical).

Figure 15 shows a portion of the downstream excavation plan for the west dam buttress area. As shown on the excavation plan, the Project design requires portions of the west dam buttress foundation area to be excavated in stages. This requirement, which stipulates that the area be excavated and backfilled in maximum 100-foot-wide sections, provides an additional measure of protection against hazards from potentially unstable temporary construction slopes in the west dam area.

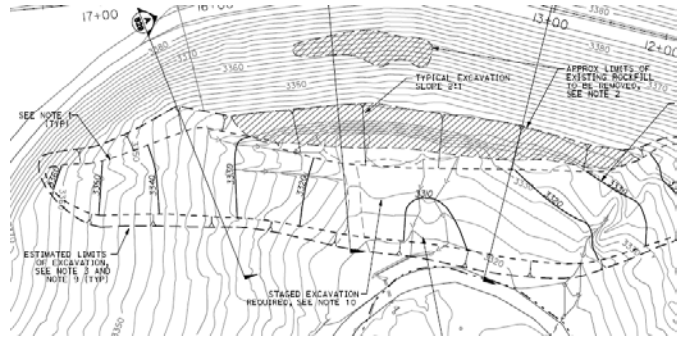

Fig. 15. Excavation Plan, West Dam Area.

As described in the Project Drawings and Specifications, the actual depths and limits of excavation within the downstream buttress footprint will be determined in the field during construction. This flexible excavation approach is intended to allow design assumptions to be field verified and to facilitate potential modifications to accommodate subsurface conditions encountered during construction. Visual inspection and final approval from AMEC, PG&E, DSOD, and FERC is required for all downstream buttress foundation surfaces.

Blanket/Chimney Filters and Drains

To mitigate the potential for internal erosion of the hydraulic fill embankment materials after earthquake shaking, the retrofit design includes chimney filters and drains on the downstream slopes of the dam between the rockfill buttresses and the existing hydraulic fill. The chimney filters and drains transition to blanket filters and drains at the base of the new rockfill buttress. Figure 16 shows a typical design cross section of the downstream buttress with associated filter (Zone 3), drain (Zone 4), and rockfill (Zones 2A and 2B) zones.

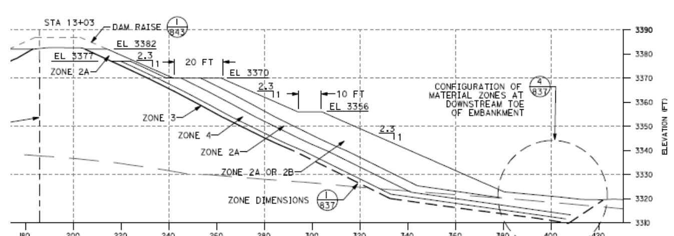

Fig. 16. Typical Downstream Buttress Section.

To install the filters and drains, the existing rockfill buttresses on the downstream slopes of the west dam, the transition section, and the east dam will be completely removed to expose the underlying hydraulic fill. Filter materials will be placed in direct contact with the hydraulic fill and will in turn be covered by drain materials.

Subdrainage Collection System

To control seepage through the dam embankment and foundation, the retrofit design includes a subdrainage collection system beneath the new downstream buttresses. The subdrainage collection system includes several reaches, which collect and convey seepage to a series of four manholes along the toe of the dam. From each manhole, the seepage flows will pass through a monitoring weir before discharging into existing drainages downstream of the dam. Discharge locations and elevations for the subdrainage system were selected to limit environmental impacts by maintaining existing drainage patterns downstream of the dam. Figure 17 shows a plan view of the east dam and main dam reaches of the subdrainage system and Fig. 18 shows a typical subdrain detail.

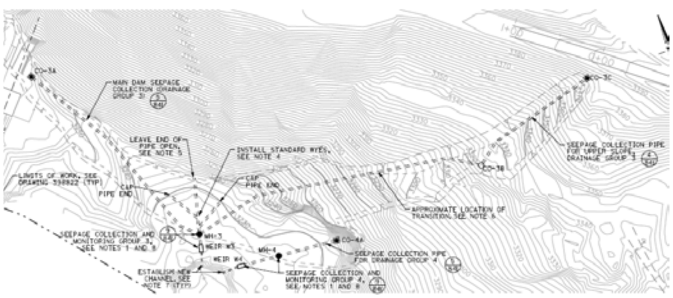

Fig. 17. Subdrain Plan, East Dam and Main Dam Reaches.

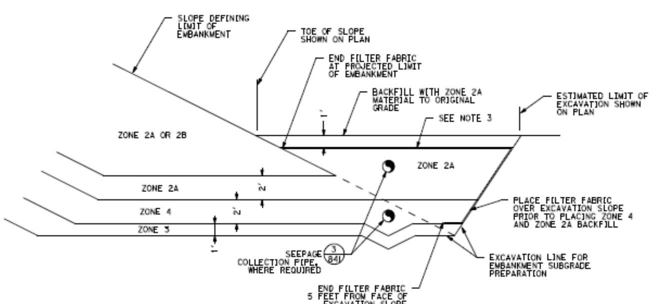

Fig. 18. Typical Subdrain Detail.

Downstream Buttresses

As described above, the downstream buttresses will be constructed of clean, compacted rockfill. The rockfill will consist of smaller (Zone 2A) material, with particle sizes ranging from about ¾-inch to 12 inches, and larger (Zone 2B) material, with particle sizes ranging from about 6 to 20 inches. Zone 2A rockfill is required in portions of the buttress adjacent to the chimney and blanket drains. Elsewhere, to facilitate efficient operation of the on-site quarry and accommodate potential variations in the available quantities of the different rockfill materials, the downstream buttress design includes options to use Zone 2A or Zone 2B rockfill.

Cross-sections showing the design configurations of the downstream transition section, main dam, west dam, and east dam buttresses are shown on Figs. 5 through 8 and a plan view of the downstream buttresses is shown on Fig. 9. Figure 16 shows a typical design section for filter, drain, and rockfill zones within the downstream buttresses.

Dredging

Dredging is required to expose suitable buttress foundation materials on the upstream side of the dam. Based on the results of an offshore exploration program that included eight rotary wash borings and 46 geotechnical probes, AMEC developed a dredging plan to remove lake bottom sediments, loose residual soils, and other unsuitable foundation materials from the upstream side of the dam. Figure 19 shows the dredging plan for the west dam area.

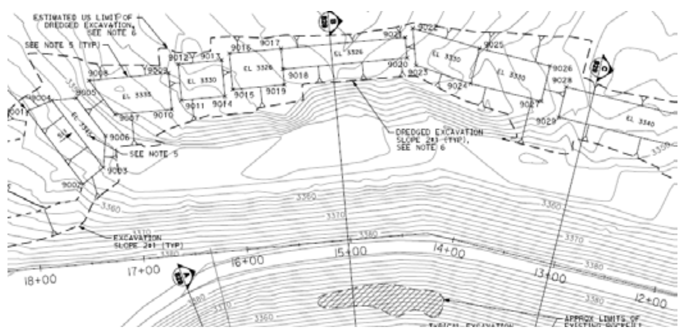

Fig. 19. Dredging Plan, West Dam.

As shown on Fig. 19, the dredging plan includes a series of dredging "panels." Within each panel, the buttress foundation excavation is required to extend to a specified target elevation unless "hard material" (i.e., weathered bedrock or boulders that cannot be excavated by conventional dredging methods) is encountered above the target elevation. The design dredging panels extend to depths between about 5 and 20 feet below the lake bottom. Based on anticipated lake levels during construction, it is expected that dredging will be performed in as little as 5 and as much as 70 feet of water.

To reduce the amount of time required for inspection, testing, and regulatory acceptance of the dredged foundation areas, AMEC implemented a record testing program to "pre-characterize" and secure regulatory acceptance of foundation materials based on the elevation of the dredged surface. The record testing program included offshore drilling, sampling, and testing activities to confirm that the proposed limits of dredging would expose suitable foundation materials. DSOD and FERC participated in the field exploration activities and reviewed and accepted the findings of the program. Subsequent acceptance of the upstream buttress foundation areas was based on the surveyed elevations and lateral limits of dredging. Post-dredging sampling and testing of the upstream buttress foundations was not required.

Upstream Buttresses

The upstream buttresses will be constructed of clean rockfill. To facilitate underwater placement, which will be necessary for portions of the buttresses below an elevation of about 3,350 feet, and to protect against wave action, the rockfill will consist of Zone 2B (i.e. 6- to 20-inch) material. Portions of the buttresses placed underwater also may use larger Zone 2C (i.e., 18- to 36-inch) rockfill. The portions of the upstream buttresses located above an elevation of 3,350 feet will be placed and compacted in the dry.

Sections showing the design configurations of the upstream transition section, main dam, west dam, and east dam buttresses are shown on Figs. 5 through 8, and a plan view of the upstream buttresses is shown on Fig. 9. Figure 20 shows a typical upstream buttress design section.

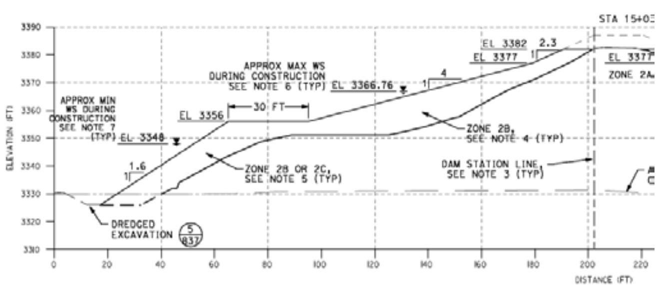

Fig. 20. Typical Upstream Buttress Section.

Main Dam Core Wall Improvements

Proposed improvements to the main dam core wall include excavating to expose the upper 10 feet of the wall. To reduce potential lateral loads in the event of slumping and loss of soil support on the upstream side of the wall, rockfill materials excavated from the downstream side of the wall will be replaced by lean concrete. To restrain the wall from toppling, horizontal reinforcing bars will be installed through the wall and embedded in the lean concrete. The reinforcing bars will be attached to steel anchor plates on the upstream face of the wall. Figure 21 shows a cross section of the proposed core wall improvements.

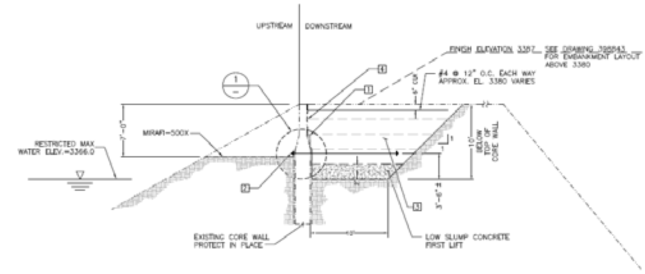

Fig. 21. Typical Section, Main Dam Core Wall Improvements.

Crest Raise

Once the upstream and downstream buttresses are complete, earth and rockfill materials will be placed to raise the crest of the dam to a completed elevation of about 3,387 feet. In accordance with the Project's design objectives, the retrofitted crest elevation will provide a minimum 10 feet of freeboard above the normal maximum reservoir water surface elevation. Figure 22 shows a typical design section for the crest raise.

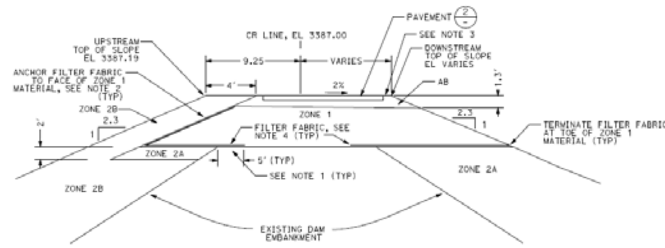

Fig. 22. Typical Section, Crest Raise.

Instrumentation

Constructing the downstream buttresses and crest raise will require the demolition of 20 existing piezometers, 4 existing weirs, and up to 24 existing settlement monuments that are currently used to monitor the dam. During construction, these instruments will be replaced by 16 new piezometers, 4 new weirs, and 27 new settlement monuments. The new piezometers will be installed at eight locations on the crest and downstream slopes of the dam. Two piezometers will be installed at each location; one to monitor water levels within the foundation, and one to monitor levels within the embankment. PG&E currently plans to equip the new piezometers and weirs with automatic data collection systems.

CONCLUSION

The results of seismic stability analyses performed in 2005 and 2006 showed that Crane Valley Dam would experience large deformations as a result of the postulated ground motions during design-level earthquake events. To improve the seismic stability and performance of the dam, PG&E

initiated the Crane Valley Dam Seismic Retrofit Project, which includes raising the dam crest, placing new rockfill buttresses on the upstream and downstream slopes of the dam, constructing internal drainage improvements, and reinforcing portions of the concrete core wall.

Project components were designed to meet seepage control and seismic stability criteria and to accommodate existing facilities, limited site access, seasonal reservoir operations, and environmentally sensitive areas within and adjacent to the Project site. Engineering analyses included static, seepage, and dynamic finite element analyses to evaluate the potential for liquefaction of hydraulic fill materials and post-earthquake stability of the retrofitted dam embankment.

Construction of the Project began in October 2010 and was completed in November 2012. Figure 23 shows an aerial view of the Project site in June 2012, after completion of the downstream buttresses and during dredging and rockfill placement operations on the upstream side of the dam.

Fig. 23. Project Construction, June 2012.

Completion of the Crane Valley Dam Seismic Retrofit Project represents a key milestone in PG&E's efforts to protect public safety and improve the seismic performance of its facilities. In meeting the Project's design objectives, the retrofitted dam embankment is expected to withstand the design earthquake ground motions and continue to provide energy, water storage, flood protection, and recreational benefits to local communities, PG&E customers, and visitors to Bass Lake.

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During the initial phases of Project planning and development, PG&E convened a project-specific board of consultants (BOC) to review and provide technical guidance related to analysis, design, and construction of the Project. Members of the BOC include Professors Alfred J. Hendron, Jr., I.M. Idriss, and Mete A. Sozen. In support of the analysis and design efforts described in this paper, the BOC advised on a number of key topics, including:

- Development and selection of appropriate retrofit design criteria;
- Establishment of design earthquake scenarios and ground motions;
- Analysis and interpretation of SPT blowcount data;
- Development of soil properties and input parameters for static and dynamic stability analyses;
- Selection of preferred design approaches;
- Development of essential Project components; and
- Conceptual design and analysis of structural improvements to the main dam core wall.

The BOC also reviewed and provided technical input for the Project Drawings and Specifications.

As integral members of the Project team, the BOC played a key role in identifying appropriate considerations for analysis and developing technically sound design approaches. The BOC's expertise and experience were particularly valuable in addressing regulatory and potential dam safety concerns related to the design, construction, and performance of the completed Project.

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