BACK ANALYSES OF STONE CANYON DAM FOR THE 1994 NORTHRIDGE EARTHQUAKE

E. Dawson¹, P. Somerville², A. Skarlatoudis³, N. Nesarajah⁴, J. Hu⁵ and F. Tatone⁶

ABSTRACT

Stone Canyon Dam is a 225-ft high zoned earthfill dam located in Los Angeles, California. Downstream portions of the dam rest on up to 75 feet of potentially liquefiable alluvium. As part of an extensive field investigation and site characterization study, back analyses of the dam’s performance during the 1994 Northridge Earthquake were performed. At the time of the earthquake, strong motion instruments located at the dam crest and left abutment did not trigger. However, fortuitously, ground motions were recorded by a temporary station located on a ridgetop near the right abutment. Following the earthquake, to measure topographic effects during aftershocks, weak motion instruments were installed at the ridgetop and at the base of the ridge. Analysis of the aftershock recordings by these instruments suggests that the ridgetop significantly amplified ground motions with respect to the base of the ridge for shorter periods. This paper describes adaptation of the recorded ridgetop motion for use as input to the dam foundation, for use in various numerical back analyses performed to check that the shear strength and elastic properties derived from the site characterization effort are consistent with the dam’s performance during the Northridge Earthquake.

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Stone Canyon Dam is a 225-ft high zoned earthfill dam located in Los Angeles, California. Downstream portions of the dam rest on up to 75 feet of potentially liquefiable alluvium. As part of an extensive field investigation and site characterization study, back analyses of the dam’s performance during the 1994 Northridge Earthquake were performed. At the time of the earthquake, strong motion instruments located at the dam crest and left abutment did not trigger. However, fortuitously, ground motions were recorded by a temporary station located on a ridgetop near the right abutment. Following the earthquake, to measure topographic effects during aftershocks, weak motion instruments were installed at the ridgetop and at the base of the ridge. Analysis of the aftershock recordings by these instruments suggests that the ridgetop significantly amplified ground motions with respect to the base of the ridge for shorter periods. This paper describes adaptation of the recorded ridgetop motion for use as input to the dam foundation, for use in various numerical back analyses performed to check that the shear strength and elastic properties derived from the site characterization effort are consistent with the dam’s performance during the Northridge Earthquake.

Introduction

Stone Canyon Dam is a 225-foot high, zoned earthfill dam located on the south slope of the Santa Monica Mountains overlooking the Bel Air section of Los Angeles. The dam is owned and operated by the Los Angeles Department of Water and Power (LADWP). The current dam was constructed in the 1950s upstream of an earlier dam at the site (the 1924 Embankment), with much of this earlier dam left in place (Fig. 1). Downstream portions of the new dam, along with the original 1924 Embankment, rest on alluvium, up to 75 feet thick at the maximum section. Potential liquefaction of this alluvium is the primary cause of concern for the seismic stability of the dam, as the Maximum Credible Earthquake (MCE) scenario for the dam site is a magnitude (Mw) 7.0 event with peak ground acceleration (PGA) of 0.87 g [1].

Previous investigations of the alluvium in 1977 and 2001 were hampered by the significant fraction of gravel sized particles, complicating the interpretation of SPT blow counts and leading to poor CPT penetration. In 2014, with new technology becoming available for characterizing

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potentially liquefiable gravelly soils, LADWP undertook an additional, more extensive field investigation, including Instrumented Becker Penetration Tests (iBPTs), mud-rotary borings with SPT measurements, CPT soundings, and sonic core sampling and suspension logging for shear- and compression-wave velocities [2].

As part of this site characterization study, back analyses of the dam’s performance during the 1994 Northridge Earthquake were performed, using ground motions recorded by a temporary station located on a ridge crest near the right abutment. This paper describes adaptation of the recorded ridge crest motions for use as input to the dam foundation, and various numerical back analyses performed to verify that the shear strength and elastic properties derived from the site characterization effort were consistent with the dam’s performance during the Northridge Earthquake.

Figure 1. Stone Canyon Dam cross section.

**Dam Performance During Northridge Earthquake**

To date, the most significant seismic loading experienced by Stone Canyon Dam was from the 1994 Northridge Earthquake (Mw 6.7). During the earthquake, strong-motion instruments located at the dam crest and left abutment did not trigger. However, acceleration was fortuitously recorded by a temporary strong-motion station located along a ridgetop near the right abutment. Peak accelerations in the upstream-downstream direction and in the cross-canyon direction were 0.26 g and 0.38 g, respectively (Fig. 2). Additional ground motion data from the main shock were provided by seismoscopes at the dam crest and left abutment (Fig. 3), [3].

The crest monument survey after the Northridge Earthquake showed a maximum earthquake-induced settlement of 0.07 feet at the center of the dam crest, with 0.09 feet of horizontal movement downstream. The only signs of distress reported at the dam site were hairline extensions of existing cracks at the dam crest [4]. No evidence of liquefaction was reported.

**Topographic Amplification at the Ridgetop Station**

Since the temporary strong motion station was located on a ridgetop, there was concern that the recorded motions may have been affected by topographic amplification. To address this, following the earthquake a weak motion instrument was co-located with the strong motion station on the ridgetop, and another weak motion station was installed at the base of the ridge.
Figure 2. Northridge Earthquake horizontal motions recorded at the temporary ridgetop station.

Figure 3. Seismoscope traces of instruments located at left abutment and dam crest.

Aftershock recordings by these instruments were then compared in order to estimate topographic amplification.

Fourier amplitude spectra at the ridgetop and the ridge-base station, for a recorded $M_w$ 2.0 aftershock were smoothed using the Konno & Ohmachi [5] method (Fig. 4). The ratios of the smoothed spectra were then applied as filters to adjust the recorded main shock motions for topographic effects. Since the amplitude ratios of the two aftershock components were different,
different filters were applied to the two main-shock components. In Fig. 5, response spectra of these directly adjusted motions are compared with recorded motions. While the directly adjusted cross-canyon component looks reasonable, deamplification of the critical upstream-downstream component appears too extreme, especially for low periods.

![Figure 4. Fourier amplitude spectra for Mw 2.0 aftershock: Ridgetop and Ridge base.](image)

![Figure 5. Response spectra for recorded and modified ridgetop motions.](image)

Based on these results, it was concluded that applying Fourier amplitude ratios measured from a single aftershock might not be the best method for filtering out ridge effects. Instead, engineering judgment, combined with findings from previous research would provide more reasonable results.

One recent work on this topic was performed by Pagliaroli et al. [6] who reviewed several topographic amplification studies with the aim of developing recommendations for Eurocode 8. They present data for both peak ground acceleration and broad band amplification. In their work,
Topographic amplification is expressed as a function of the ratio H/L of horizontal distance to vertical distance between the measurement points. For the Stone Canyon ridgetop, this H/L ratio is 110ft/170ft = 0.65, which corresponds to an amplification factor of approximately 1.8.

Taking into account both the Pagliaroli et al. [6] data and the measured amplification ratios, a simplified, judgment-based spectral amplification function was developed. As shown in Fig. 6, for periods greater than 1 second, it is assumed there is no topographic amplification, and for periods less than 0.3 seconds, the amplification factor is a constant 1.8. These two segments of constant amplification are then connected with a linear segment which roughly approximates the measured amplification function.

Figure 6. Measured topographic spectral amplification, along with interpreted topographic amplification function used as filter.

This interpreted Fourier amplitude ratio function was then applied as a filter to adjust both components of the recorded Northridge main shock motions for topographic effects. In Fig 5, response spectra of the adjusted motions are compared to those of the recorded motions, and the ones obtained by direct filtering. For the cross-canyon direction, the adjusted motion is not very different from that produced by direct filtering. However, for the upstream-downstream direction the adjusted motion is significantly higher.

**Site Characterization Study**

A cross section of Stone Canyon Dam was shown in Fig. 1. In general the embankment materials are dense to very dense, silty sands with gravel. The new dam (1956 Embankment) was designed as a zoned embankment with relatively impervious fill placed on the upstream side and more pervious fill on the downstream side. The alluvium underlying the downstream portions of the dam is a complex, interlayered mix of silty and clayey sands with gravel and sandy clays.
Estimated material properties for the various dam materials, and the underlying alluvium, developed during a recent site characterization study [2], are summarized in Table 1. Shear modulus values are best estimates, while shear strengths are generally 33rd percentiles. An exception is the undrained strength of the 1956 embankment, impervious material which is a lower-bound curve fit to the CPT data:

\[
\frac{\sigma'_v}{\sigma'_v} \leq 8 \text{ atm} : \frac{s_u}{\sigma'_v} = 0.35 \left( \frac{8 \text{ atm}}{\sigma'_v} \right)^{0.35}, \quad \frac{\sigma'_v}{\sigma'_v} > 8 \text{ atm} : \frac{s_u}{\sigma'_v} = 0.35
\]  

Table 1. Summary of soil properties developed during site characterization study.

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit Weight (lb/ft³) (unsat./sat.)</th>
<th>Elastic Stiffness K₂,max</th>
<th>Friction Angle (degrees)</th>
<th>Cohesion (psf)</th>
<th>Undrained Strength Sᵤ/σ′ᵥ</th>
</tr>
</thead>
<tbody>
<tr>
<td>1956 Embankment, Pervious</td>
<td>130/135</td>
<td>200</td>
<td>36</td>
<td>50</td>
<td>-</td>
</tr>
<tr>
<td>1956 Embankment, Semi-Pervious</td>
<td>130/135</td>
<td>175</td>
<td>35</td>
<td>50</td>
<td>-</td>
</tr>
<tr>
<td>1956 Embankment, Impervious</td>
<td>130/135</td>
<td>160 (Vₛ ≤ 1,800 ft/sec)</td>
<td>-</td>
<td>-</td>
<td>See Eq. 1</td>
</tr>
<tr>
<td>1924 Embankment</td>
<td>125/130</td>
<td>150</td>
<td>35</td>
<td>50</td>
<td>-</td>
</tr>
<tr>
<td>Hydraulic Fill</td>
<td>125/130</td>
<td>77</td>
<td>-</td>
<td>-</td>
<td>0.23</td>
</tr>
<tr>
<td>Alluvium Beneath Embankment</td>
<td>125/130</td>
<td>77</td>
<td>33</td>
<td>-</td>
<td>0.23</td>
</tr>
<tr>
<td>Alluvium at Toe</td>
<td>120/125</td>
<td>77</td>
<td>37</td>
<td>50</td>
<td>-</td>
</tr>
<tr>
<td>Bedrock (Santa Monica Slate)</td>
<td>142</td>
<td>Vₛ = 3,350 ft/sec</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

The small strain shear modulus, Gₘₐₓ, is given as a function of mean effective stress σₗ (in psf) using the traditional K₂,max constant:

\[
Gₘₐₓ = 1000K₂,max \sqrt{\sigma'_m}
\]  

**Numerical Back Analyses**

With the modified ridgetop record applied to the base of the dam, various numerical back analyses were performed to verify that the elastic stiffness and shear strength developed during the recent site characterization effort are consistent with the dam’s performance during the Northridge Earthquake. Back analyses included (1-D) SHAKE equivalent linear analyses, as well as 2-D and 3-D equivalent linear and non-linear dynamic analyses.
Only the results of the 3-D analyses are reported here. These were performed with the computer code FLAC\textsuperscript{3D}, Version 5.0 [7] an explicit finite difference program for geotechnical engineering and rock mechanics computations. The 3-D numerical mesh used for the back analyses is shown in Fig. 7.

Dynamic shaking was simulated by applying acceleration-time histories to the base of the model through a compliant base. This non-reflective boundary absorbs downward-propagating shear and compression (p-) waves, eliminating non-physical reflections off the base of the numerical mesh. In order to minimize boundary disturbances, the four vertical sides of the mesh were equipped with free-field boundaries simulating level ground conditions.

For the nonlinear deformation analyses, soils were modeled with a linear-elastic/perfectly plastic model with a Mohr-Coulomb yield condition. The equivalent linear procedure was implemented by performing a series of linear elastic analyses, with the shear modulus and damping for each element updated individually according to the shear strain of that element, tracked during the previous iteration. Iterations were continued until strain-compatible properties were obtained. For the equivalent linear-analyses, all embankment and alluvium materials were assigned the Sand Average G/G\textsubscript{max} relation of Seed and Idriss [8], and Sand Lower Bound damping relation of Seed and Idriss [8].

![Figure 7. Cutaway view of 3-D numerical mesh used for equivalent linear and nonlinear back](image)

**Comparison of Computed Motions to Crest and Left Abutment Seismoscopes**

The seismoscope traces from the instruments at the dam center crest and at the left abutment provide a means of checking the reasonableness of the estimated Northridge input motions, as
well as the computed dam response. Seismoscopes are essentially damped pendulums. The Stone Canyon Dam seismoscopes had a natural period of 0.75 seconds with damping of 9.6% [3].

Back-calculating acceleration-time histories from seismoscope traces, as performed by Scott [9], is difficult and requires high quality seismoscope traces. However, forward analysis, computing a synthetic seismograph trace for a given input acceleration, is straightforward. Figure 8 shows synthetic seismograph traces for the modified ridgetop motion, and the dam center crest motion computed from the equivalent linear analyses. These can be compared to the recorded seismograph traces shown in Figure 3. It is assumed here that the motion at the left abutment is comparable to that of modified ridgetop motion, at least at the 0.75 second period of the seismoscope.

Figure 8. Synthetic seismograph traces for modified ridgetop motion and computed dam crest motion from 3-D equivalent linear analysis.

While not identical, the synthetic seismograph trace for the modified ridgetop motion and the recorded trace at the left abutment have several distinct features in common, including: two large narrow lobes oriented just west of north; a smaller, broader lobe in the north-northwest quadrant; a large narrow lobe oriented just east of south; and a smaller lobe oriented just south of east. This provides further confidence that the modified ridgetop motion is an appropriate input motion for back analyses of the dam.

Comparison of seismoscope traces for the dam crest is more difficult, as the recorded motion appears to have gone off the scale at several points and the trace lines are less distinct. However, a rough estimate of the amplification occurring at the dam crest can be made by measuring the extent of the traces along a line in the upstream-downstream direction. As shown in Figure 3, this distance is about 0.6 inches for the left abutment seismoscope, and is about 1.3 inches for the crest seismoscope, suggesting an amplification ratio of 2.2. For the computed synthetic seismoscope traces, this ratio is about 2.1, suggesting that the 3-D equivalent linear analysis is
producing approximately the same amount of amplification at the dam crest – at least at the 0.75 second period of the seismoscopes.

**Shaking-Induced Settlements and Displacements along Dam Crest**

In Fig. 9, measured displacements along the dam crest are compared with those obtained from the 3-D elasto-plastic analysis, using the soil properties listed in Table 1. While computed vertical displacements (settlements) are in good agreement with measured values, horizontal displacements are smaller; and, toward the right abutment are in the upstream direction, rather than downstream, as measured. The fact that this computed upstream deformation occurs within the impervious 1956 embankment material, suggests that the shear strength assigned to this material might be too low. This is consistent with the use of a lower bound shear strength for the impervious 1956 Embankment material.

![Comparison of computed and measured settlement along dam crest.](image)

**Conclusions**

As part of an extensive site characterization study, back analyses of the performance of Stone Canyon Dam during the 1994 Northridge Earthquake were performed, using ground motions recorded on a ridgetop near the right abutment. The recorded ridgetop motions were adapted for use as input to the dam foundation by filtering out estimated topographic amplification. The results from 3-D equivalent-linear and nonlinear elasto-plastic analyses verified that shear strength and elastic properties derived from the site characterization effort are consistent with the
dam’s performance during the Northridge Earthquake. The information presented also provides a useful case history of the seismic performance of a well compacted earthfill dam subjected to moderate shaking.

References

1. URS, Seismic Hazard Analysis and Ground Motion Time History Selection and Development, Stone Canyon Dam, Report prepared for Los Angeles Department of Water and Power, July 18, 2013.