

**VALIDATION OF FINITE ELEMENT ANALYSIS TECHNIQUES USING LOMA
PRIETA RECORDINGS AT LOWER CRYSTAL SPRINGS DAM**

Vojislav Cvijanovic, Mark G. Schultz, Ingrid J. Dittmar and William A. Fraser

California Department of Water Resources
Division of Safety of Dams

Abstract

Finite element modeling provides insight into the complex three-dimensional response of hydraulic structures and can heavily influence dam safety decisions. As the state of practice continues to evolve, confidence in results can only be gained once analysis methods have been validated and evaluated using direct measurements of known behavior. This paper focuses on comparing physical recordings of dynamic response to model calculations. The findings presented are intended to demonstrate how finite element analysis methods can capture wave propagation, site response, and structural response when material properties are well defined. The comparisons shown will demonstrate the capability of current modeling techniques to re-create a known earthquake and simulate dynamic response of a dam subject to seismic loading.

Introduction

Lower Crystal Springs Dam is a concrete gravity arch located on San Mateo Creek, approximately three miles west of the City of San Mateo. The structure was originally completed in 1888, and later raised by 30-ft in 1890. The dam was constructed using interlocking blocks and does not contain any contraction joints. It has a height of 149-ft with a crest length of 600-ft, a crest width of 40-ft, and a base width of 176-ft.

In 1906, the dam was subjected to significant seismic loading following rupture of the nearby San Andreas Fault, yet no damage was observed. The dam was also subjected to seismic loading during the 1989 Loma Prieta Earthquake. During this event, a series of instruments located throughout the site measured response at the dam toe, dam crest, left abutment, and a location approximately 550-ft downstream.

The Loma Prieta measurements were used to evaluate the analytical tools and finite element analysis methods typically used in making dam safety decisions. This study focuses on determining whether the current state of practice can accurately re-create the 1989 Loma Prieta Earthquake. Results will focus on comparing calculated and measured responses to assess whether the current state of practice can accurately capture wave propagation through a foundation, wave propagation through a structure, and dam-foundation-reservoir interaction.

Review of Site Geology

Supporting Geologic Studies

For the purpose of this study, a shear wave velocity profile for sandstone beneath the free field strong motion accelerometer (located 550-ft downstream of the dam) was needed. CSMIP records indicated that a Vs30 of 713 m/s was assigned to this instrument based on an estimated Vs30 value typical for Franciscan Sandstone. Although direct measurement of the Vs30 and rock weathering profile beneath the instrument would be ideal, the funding of those measurements was outside the scope of this study. Therefore, an approach which relies on statewide Vs30 databases and a study of the specific geology was utilized.

General Geology and Geomorphology

Lower Crystal Springs Dam is founded on Franciscan Formation, which consists of inclusions or blocks of varying size and rock type within a matrix of sheared shale known as the *mélange*. The right abutment, left abutment, and much of the terrain in the canyon downstream of the dam consist of a very large greywacke sandstone block. The greywacke sandstone is typically medium to coarse grained, well graded, and often contains interbedded shale and siltstone. The sheared *mélange* is predominately soft, light to dark gray sheared shale with rock fragments. The dam is located approximately 1,200-ft from the peninsula section of the San Andreas Fault Zone.

Pampeyan (1994) recognizes several Holocene aged units downstream of the dam, including younger alluvium (Qya) and slope wash/ravine fill/colluvium (Qsr). Pampeyan (1994) maps a widespread area of colluvium extending along the base of the right side of the canyon of San Mateo Creek. The colluvium consists of well graded, unconsolidated to moderately consolidated deposits of sand, silt, clay, and rock fragments accumulated by slow downslope movement of weathered rock debris and soil.

The source material for the colluvium is the underlying weathered greywacke sandstone. Rotational slides along the base of the right side of the canyon were observed, indicating thicker accumulations of colluvium. The rotated bases of trees and arcuate scarps along the south slope of the downstream channel provided further evidence of rotational slides developed in the thick colluvium deposits. The limits of the colluvium were confirmed by hummocky terrain clearly visible on the LiDAR image, compared to smooth surface in terrain underlain by in place sandstone. Overlaying the limits of the colluvium as mapped by Pampeyan (1994) and the limits of the hummocky terrain on the LiDAR showed remarkable agreement.

Geology Underlying Strong Motion Instrument

The metal box which housed the now abandoned strong motion instrument located approximately 47-ft (at a bearing of 263°) from the southwest corner of Highway I-280 Bridge Bent 3 (Figure 1). Although this instrument is shown in an area of colluvium on the regionally scaled Pampeyan (1994) map, a moderately weathered massive sandstone outcrop can be observed adjacent to the strong motion instrument. It appears that the immediate area surrounding the strong motion instrument has undergone significant man made modifications associated with the construction of the bridge in the 1960's.

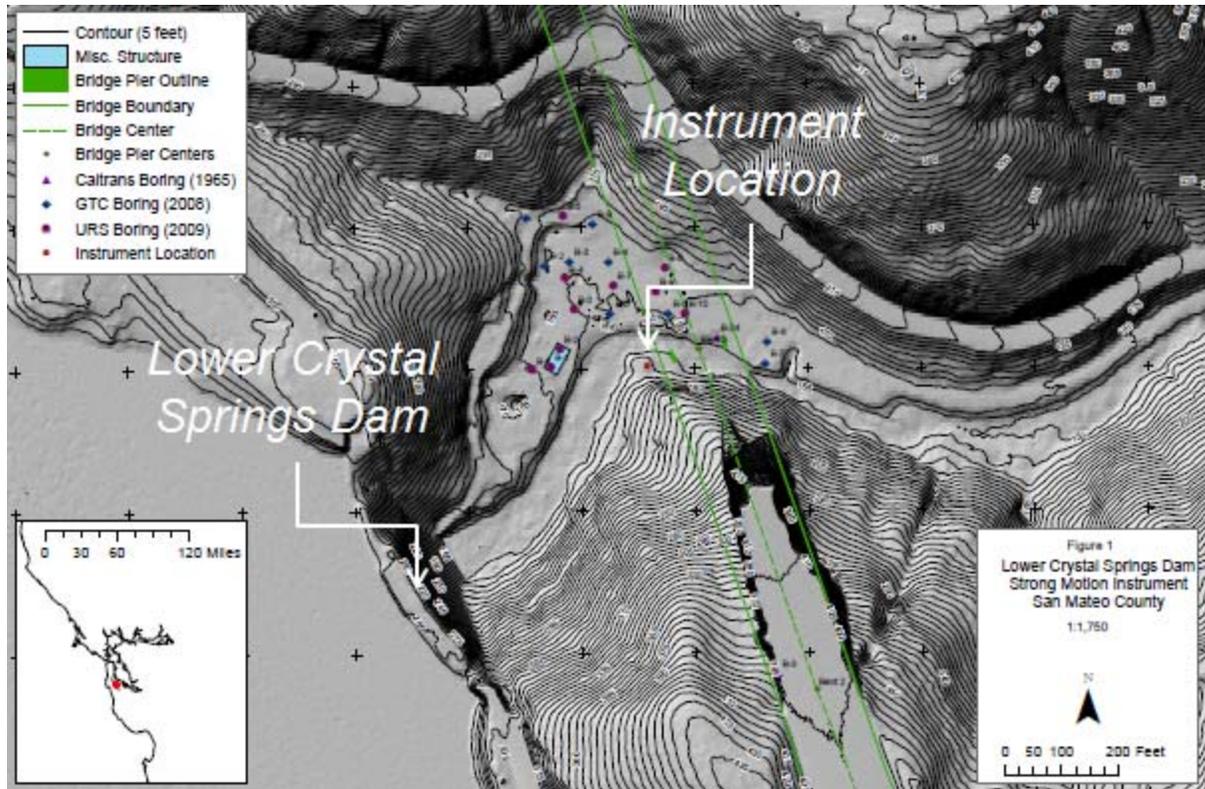


Figure 1. Lower Crystal Springs Dam Site Geology

The excavation was likely made to access a bent construction site, although the remaining evidence is subtle after 50 years. However, comparisons of the pre-bridge and current topography confirmed that approximately 15-ft of excavation has occurred at the instrument site. It is reasonable to assume that the colluvium and highly weathered and fractured sandstone in the immediate area of the instrument was removed for the construction of the bridge bent. When the strong motion instrument was installed in 1978, the site was likely chosen because of the convenient presence of exposed sandstone bedrock in the floor and cutslope of this then fresher excavation. Hand dug shallow subsurface exploration revealed that the instrument is in fact underlain by approximately 6-in of imported gravel underlain by moderately weathered sandstone bedrock.

Shear Wave Velocity Profile Estimation

The development of a shear wave velocity profile estimate at the location of the instrument is based on the assumption that the site possesses a V_{s30} of 966 m/s, which is a median value for sites on Franciscan sandstone as determined by a statewide V_{s30} database (Wills and Silva, 1998). A modification was then applied to that estimate based on the knowledge that lower velocity materials (which are present at sites considered by Wills and Silva) were removed at this instrument site (Figure 2).

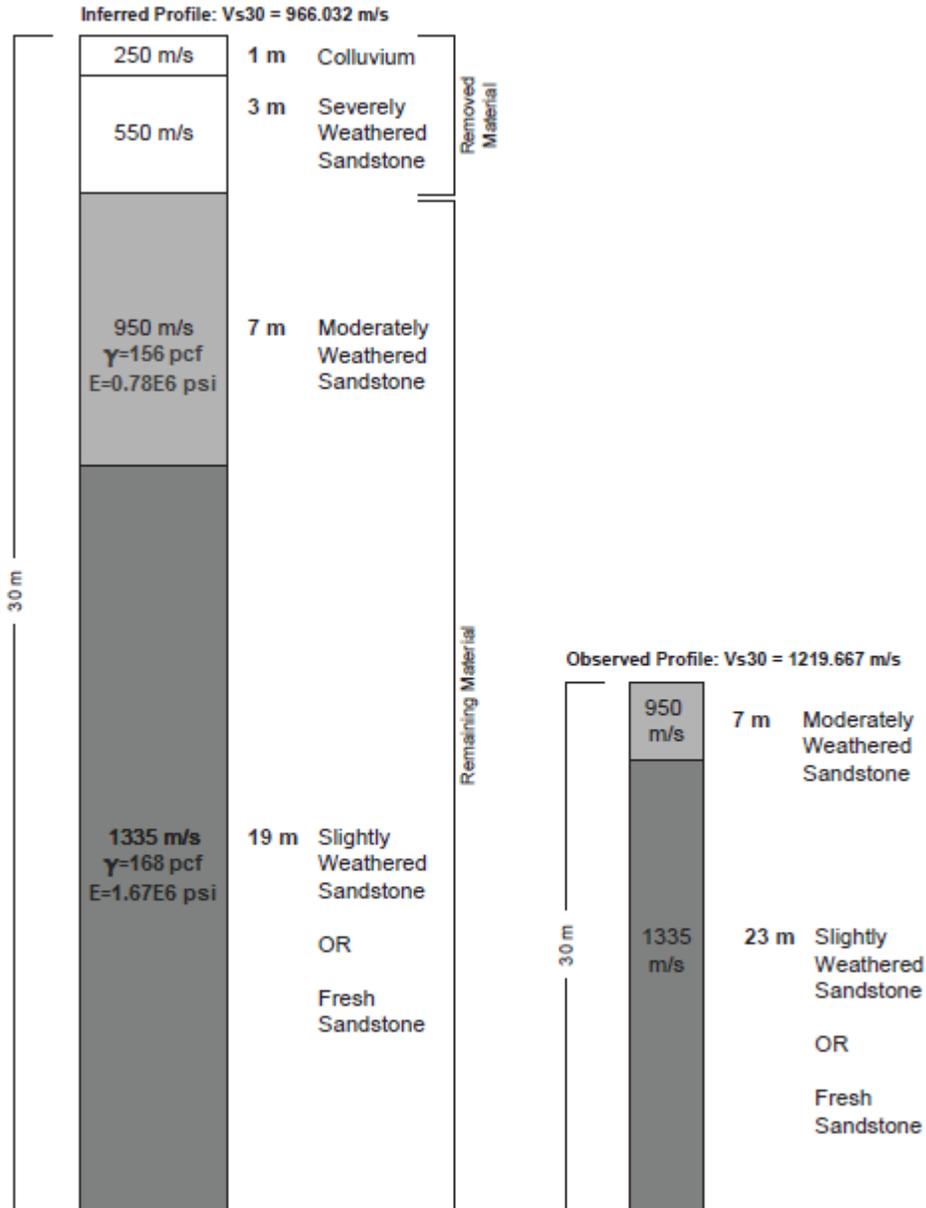


Figure 2. Shear Wave Velocity Profile of Geologic Units Underlying Downstream Instrument

To consider the local site conditions, the available boring logs (drilled during the bridge project and the nearby water distribution facilities) were reviewed to identify a rock weathering profile to a depth of 30 meters. The rock weathering profile contains four units which include surficial soils, severely weathered sandstone, moderately weathered sandstone, and slightly weathered to fresh sandstone.

Using the site weathering profile, the median V_{s30} of 966 m/s was distributed across the four units. This produced a “typical” 30 meter weathering and shear wave velocity profile consisting of 1 meter of surficial soil at 250 m/s, 3 meters of severely weathered sandstone at 550 m/s, 7 meters of moderately weathered sandstone at 950 m/s, and 19 meters of slightly weathered/fresh sandstone at 1335 m/s. Since the colluvium and severely weathered sandstone have been removed by the bridge construction exposing moderately weathered rock immediately below the instrument, it is assumed that the downstream instrument site is underlain by the remaining 7 meters of moderately weathered sandstone underlain by slightly weathered to fresh rock (Figure 2).

FEM Model of Lower Crystal Springs Dam

The finite element analysis of Lower Crystal Springs Dam is based on an LS-DYNA model of the dam, foundation, and reservoir. The model consists entirely of elastic eight node solid and fluid elements. Due to adequate contact at the dam-foundation interface, the dam and foundation were modeled as fully merged (coincident nodes along the interface) and therefore, the dam and two foundation regions consist of one continuously uninterrupted mesh. The reservoir however, was modeled as an independent part with sliding contact interfaces applied along the dam-reservoir and foundation-reservoir boundaries. Note that the reservoir model was created to be consistent with the reservoir elevation at the time of the Loma Prieta Earthquake (which corresponds to 95-ft of head).

The dam concrete was modeled using an elastic material with modulus of 3,600 ksi (consistent with 4,000 psi concrete). A nonlinear constitutive material model is often used in typical seismic evaluations. However, one was not considered in this case given the low amplitude of the recorded motion as the concrete does not reach a nonlinear range.

The foundation consists of a two layer model downstream of the dam, which is consistent with the geologic profile previously discussed (slightly weathered sandstone beneath seven meters of moderately weathered sandstone). The material properties assigned to the foundation layers are consistent with estimations presented in Figure 2. Foundation material immediately beneath the dam however, only considers slightly weathered sandstone (Figure 4), as moderately weathered sandstone beneath the dam was removed during construction.

Ground Motion Propagation through a Mass Foundation

Application of ground motion should simulate the upward propagation of waves through a mass foundation. However, because frequency content of the upward propagating wave changes throughout the propagation process, a procedure which produces a free field match requires two independent analyses.

The initial analysis is based on direct application of the target motion at depth. The computed free field accelerations can subsequently be extracted and transformed into the frequency domain ($F_{Computed\ Free\ Field}$). The frequency function of the input is then scaled by a transfer function, which is represented by the frequency function of the target divided by the frequency function of the free field measurements (Eqn. 1).

$$F_{New\ Base\ Input} = \frac{F_{Target}}{F_{Computed\ Free\ Field}} \times F_{Initial\ Base\ Input} \quad (1)$$

The result is a frequency function for a new input ($F_{New\ Base\ Input}$) which can be transformed back into the time domain and applied at depth. Application of the modified record results in a free field calculation which matches the target (Figure 3).

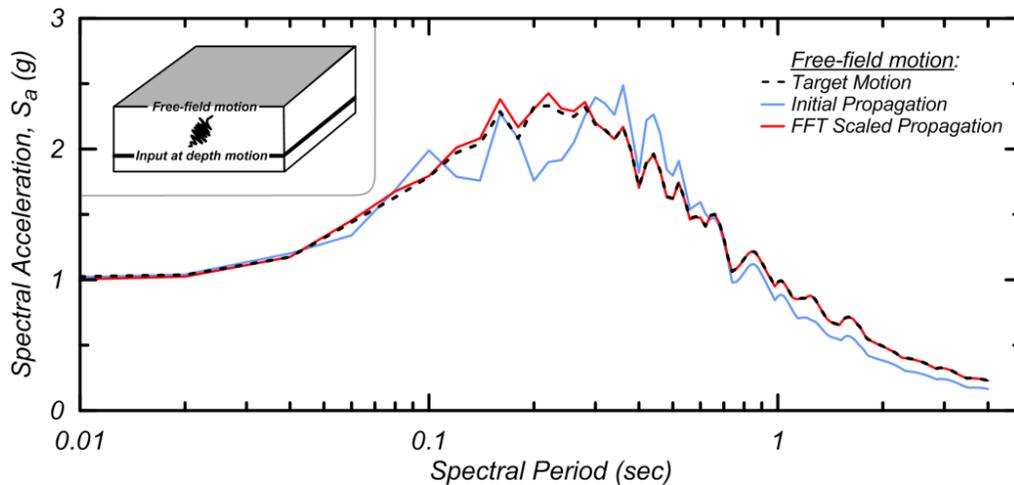


Figure 3. Example of Frequency Function Scaling.

Comparison of Loma Prieta Recordings to Analysis Calculations

The analyses completed are based on application of toe recordings at the dam toe. This decision is attributed to well-defined foundation properties beneath the structure, as moderately weathered material was removed prior to construction. Therefore, foundation uncertainties beneath the dam are limited whereas characterizing foundation material beneath the free field instrument was found to be much more of a challenge given the great deal of uncertainty.

To simulate upward propagation of waves, ground motions could not be directly applied at the toe. Therefore, analyses require developing an at-depth input which could propagate through the foundation and produce toe accelerations that match toe recordings (Figure 5). This procedure is based on the two phase ground motion application technique described above.

The initial analysis is based on the direct application of toe measurements at depth. Motions calculated at the dam toe were then used to scale the frequency function of the initial base input by a transfer function. The transformed motion, when applied at depth, then produces toe accelerations which match measured toe recordings. Once the model has demonstrated that the toe calculations coincide with the recorded toe measurements, calculated accelerations at other locations can be compared to physical measurements (Figure 6).

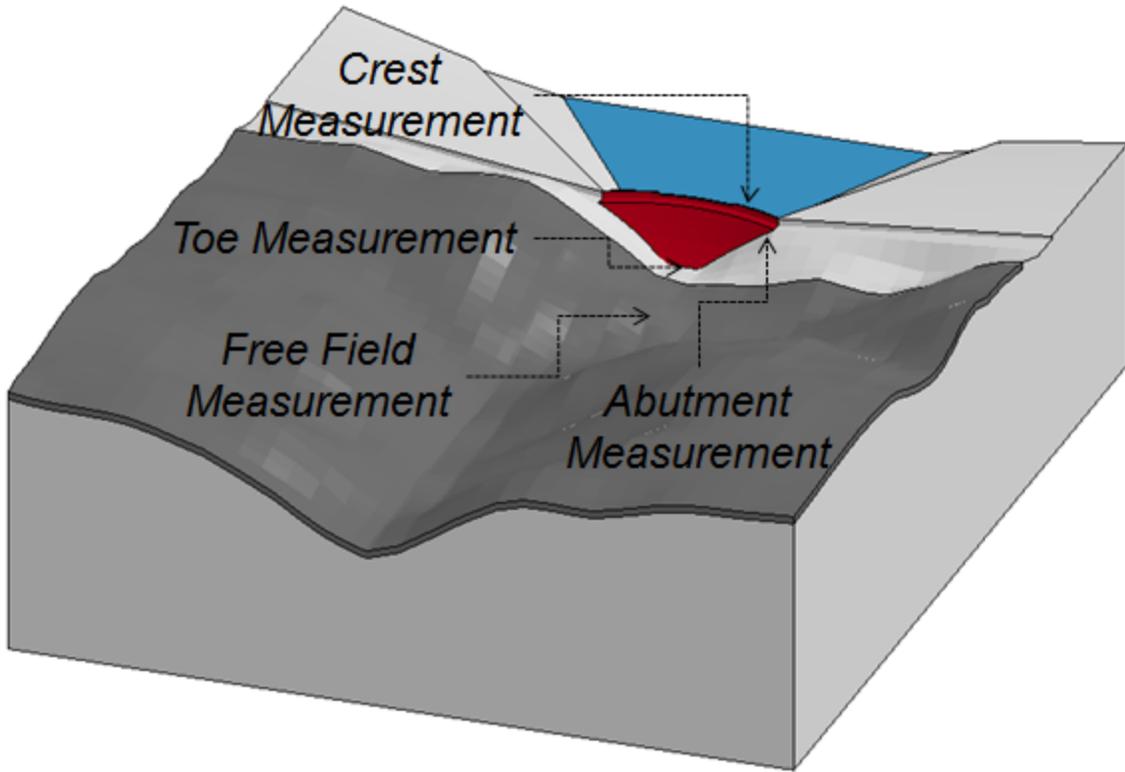


Figure 4. LS-DYNA Model of Lower Crystal Springs Dam.

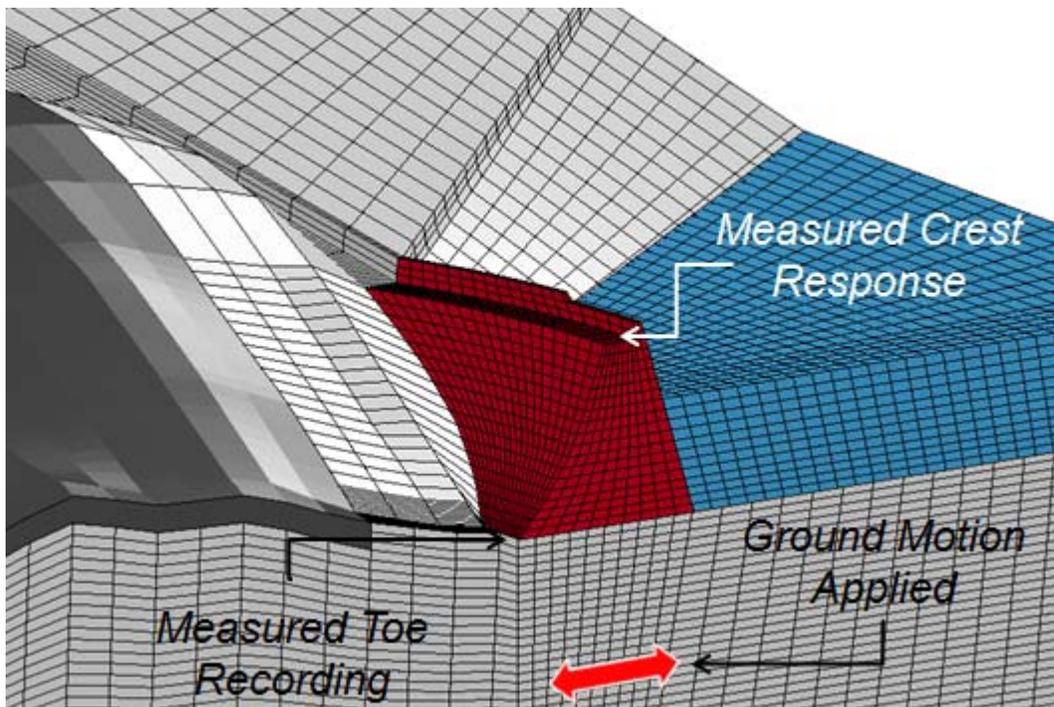


Figure 5. Ground Motion Propagation.

The analysis was found to produce positive results, which demonstrates the model's ability to re-create a recorded earthquake event. A comparison of response at the dam crest produced very favorable results, which implies that the model can accurately capture structural response. This is partially attributed to the well-defined foundation properties beneath structure, as moderately weathered material underlying the dam was removed prior to construction. Therefore, foundation uncertainties are limited and the one layer foundation profile (slightly weathered sandstone) accurately characterizes foundation properties beneath the dam.

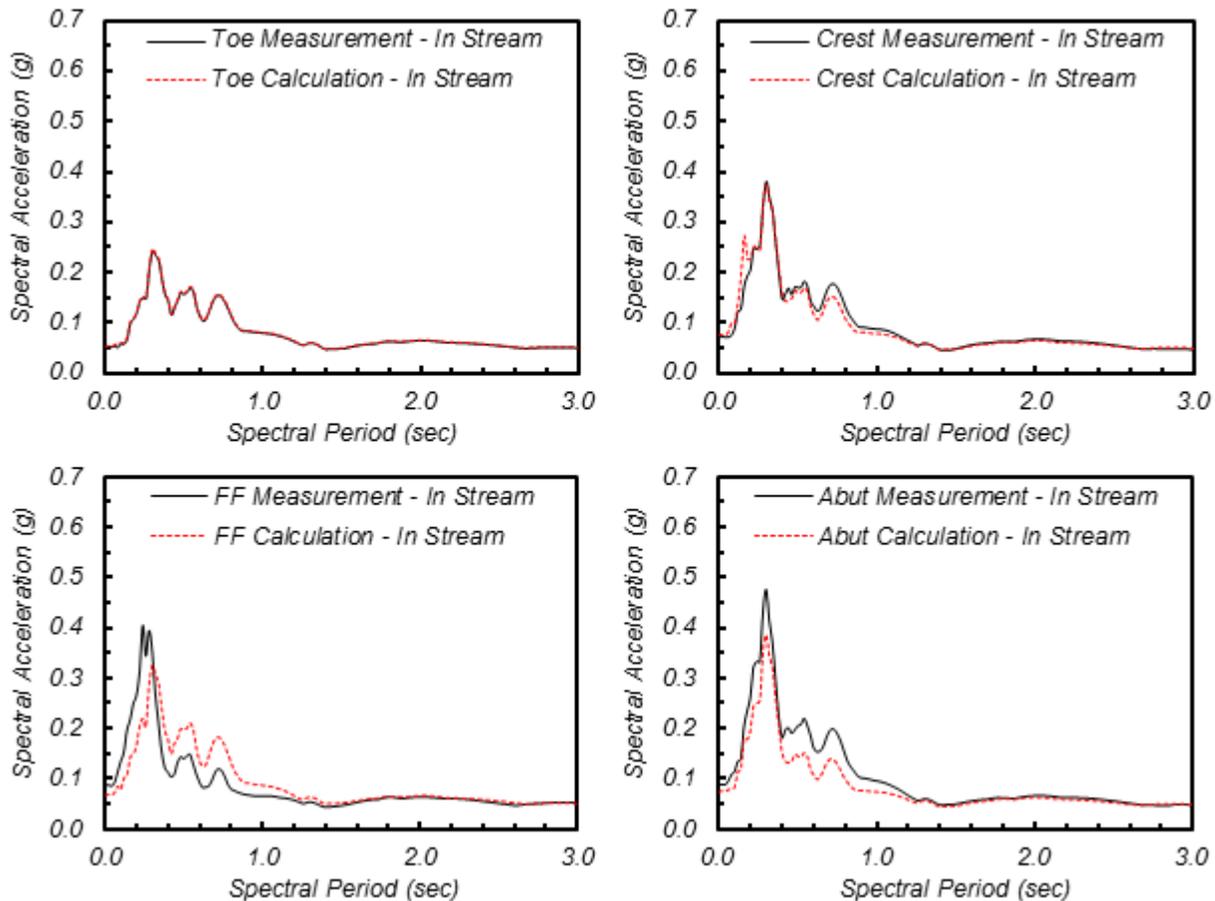


Figure 6. Comparison of Measured and Calculated Response.

Characterizing foundation material beneath the free field instrument however, was found to be much more of a challenge. This profile is based on a best estimate due to the lack of physically measured data. Therefore, the two layer foundation model beneath the free field instrument contains a great deal of uncertainty as the depth and properties of the moderately weathered sandstone are not known. This uncertainty leads to results which do not produce the same level of accuracy.

Because there are no actual shear wave measurements at the site, best estimates of moderately weathered sandstone parameters were given a range of acceptable values. This provides flexibility in evaluating how varying these parameters (within the acceptable range) affects free field response.

In Case 1, the shear wave velocity of the moderately weathered sandstone was reduced to 520 m/s, which is a lower bound stiffness estimate. As expected, amplification in free field response was calculated and results indicate a more accurate match.

In Case 2, the lower bound stiffness was combined with an upper bound moderately weathered layer thickness of 11 meters. This further amplifies response and begins to produce favorable results. Ultimately, results imply that the assumed profile may not be representative of what is present beneath the instrument. Therefore, additional exploration in this region could be helpful in better characterizing the sub-surface conditions and may lead to more accurate free field predictions.

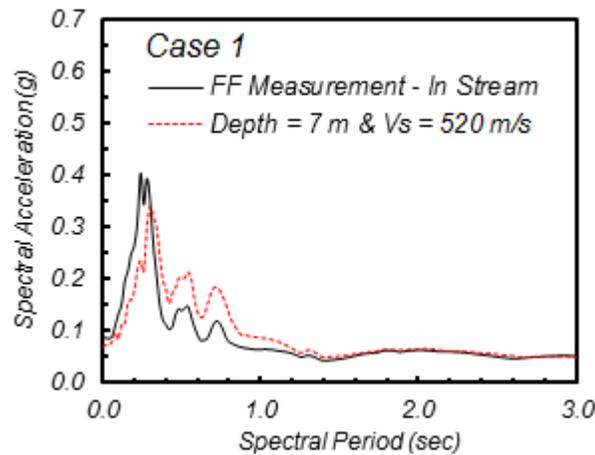


Figure 7. Sensitivity to Moderately Weathered Sandstone Stiffness.

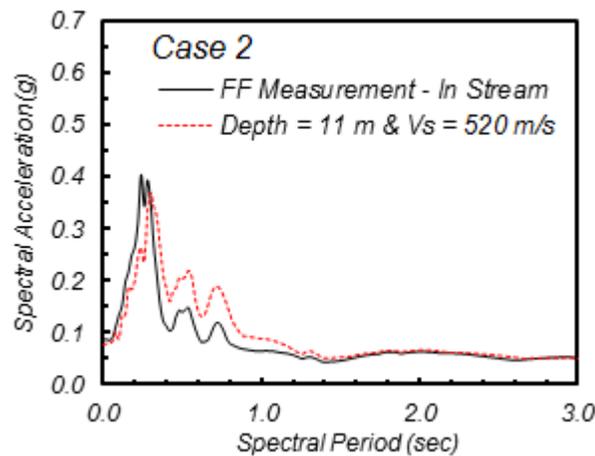


Figure 8. Sensitivity to Moderately Weathered Sandstone Depth.

Another sensitivity study was completed to evaluate how foundation damping affects response. In the previously discussed examples, five percent Rayleigh damping was applied to the structure, foundation, and reservoir. Given the favorable results produced at the dam crest, it is reasonable to conclude that applying five percent damping to the dam and reservoir is appropriate. Applying five percent damping to the foundation however, may not be suitable. Therefore, an analysis was completed to determine how sensitive free field response is to foundation damping.

The model used considers the best estimate shear wave velocity profile presented in Figure 2 and model parameters are consistent with those used in the initial analysis. The exception however, is that Rayleigh damping assigned to the foundation parts has been reduced to two percent (1-10 Hz). Note that significant changes in results were not identified (Figure 9).

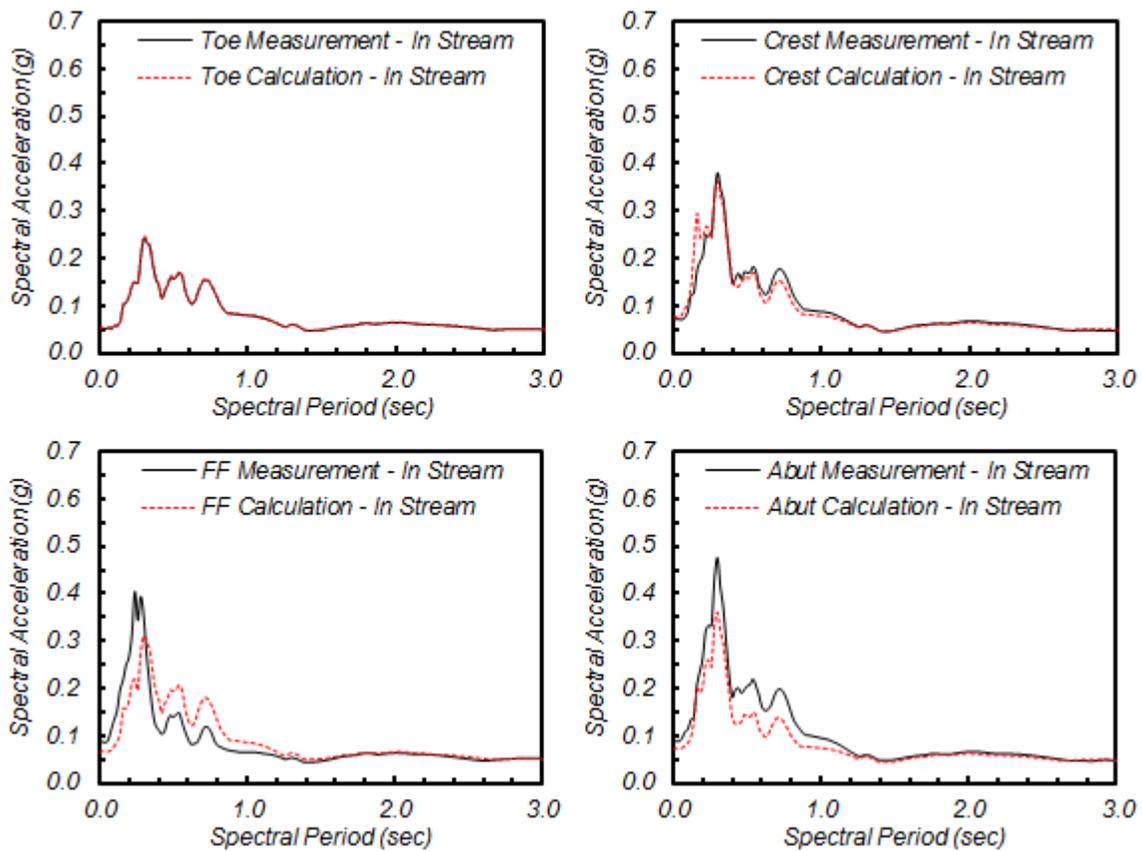


Figure 9. Sensitivity to Reduced Foundation Damping.

A final analysis was completed to evaluate how the model responds to the application of a two-component (in stream and cross canyon) toe record (vertical toe record is not available). For this analysis, five percent damping was applied to all parts and the best estimate shear wave velocity profile (presented in Figure 2) was considered. As shown in Figure 10, the comparison of response at the dam crest still produced favorable results, which implies an accurate representation of structural response.

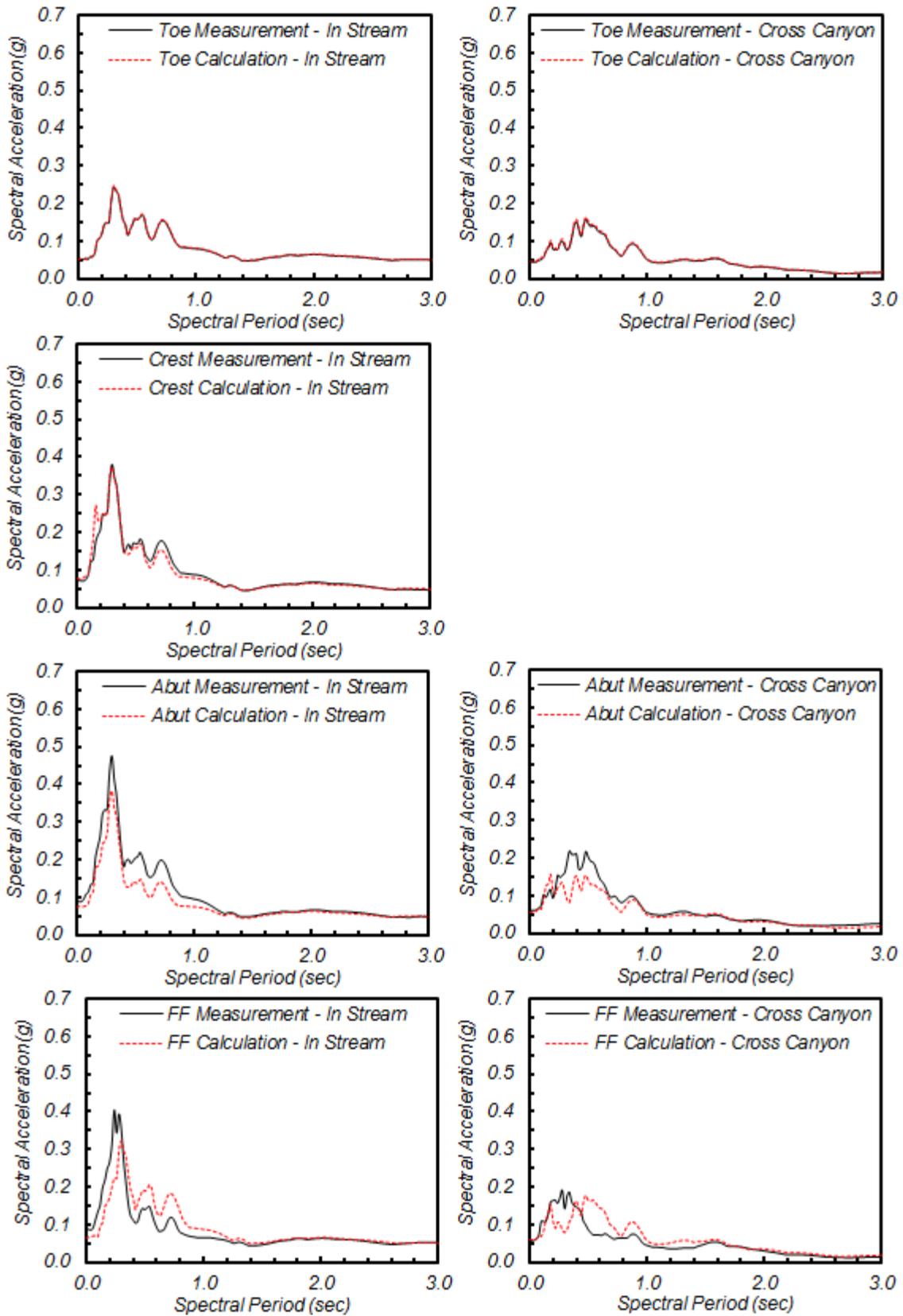


Figure 10. Two Component Ground Motion Application.

Conclusion

Findings presented in this paper are intended to demonstrate how finite element analysis methods can re-create a known earthquake when material properties are well defined. This idea is most clearly demonstrated at the dam crest, where the calculated response is in agreement with values measured during the earthquake. This comparison indicates that the model can accurately capture wave propagation through a foundation, wave propagation through a structure, structural response, and dam-foundation-reservoir interaction given well-defined properties for foundation materials underlying the structure.

In regions where material properties are not as well defined, characterizing foundation material is based on a best estimate with a certain degree of uncertainty. This uncertainty can lead to results which do not produce the same level of accuracy, as demonstrated by the free field comparison. Further exploration of material underlying the free field instrument could lead to more accurate results as demonstrated by this study through the variation of unknown parameters within a reasonable range.

Future efforts to further advance the state of practice will focus on case studies which consider stronger motion measurements. These studies will evaluate whether analytical tools can accurately predict non-linear structural response and behavior.

Acknowledgements

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