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MEMORANDUM

To: Program Manager, Dam Safety Office Technology Development Research
Attn: 84-44000 (LKrosley)

From: Hillery Venturini, Civil Engineer Hillery Venturini, P.E.
Waterways and Concrete Dams Group 1 (86-68110)

Subject: Finite Element Model Reduction Research & Validation Study

Attached for your use is the "Finite Element Model Size Reduction Research and Validation Study, DSO-2018-05" that has been prepared by the Technical Service Center for the Dam Safety Technology Development Program.

This transmittal memorandum finalizes the report phase of the dam safety technology development research study. The technical memorandum provides a comparison of results from reduced models and their larger, more extensive models, provides an insight to the parallel research efforts to validate and verify model responses, and introduces the initial efforts taken to investigate the efforts of boundary conditions, size, and ability to match ground motions.

If you have any questions, or if you would like hard copies of this report, please contact Ms. Hillery Venturini at 303-445-3281.

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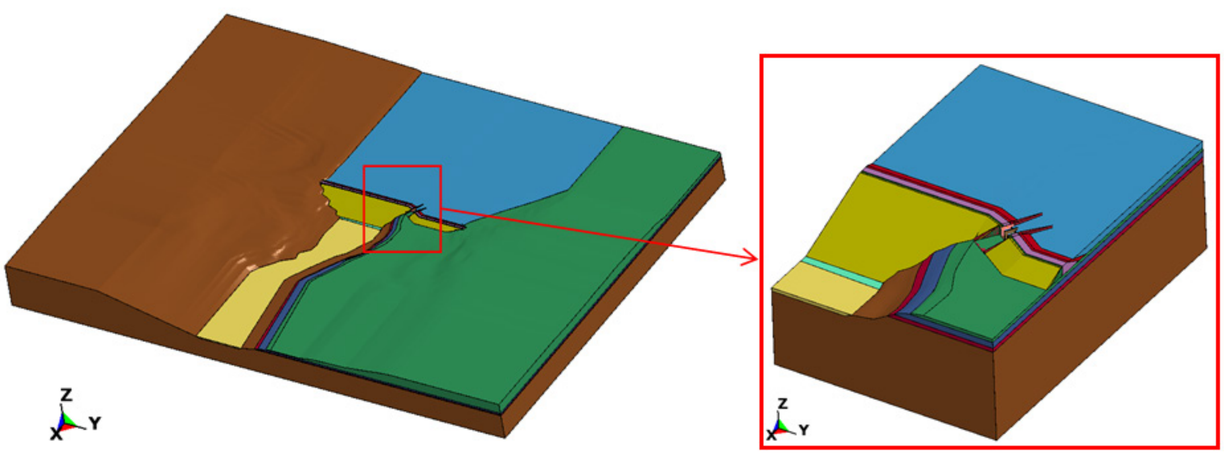
Managing Water in the West

Finite Element Model Size Reduction Research and Validation Study

DSO-2018-05

Dam Safety Technology Development Program



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April 2018

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14. ABSTRACT ***(Maximum 200 words)***
In the past years, the scope of finite element models (FEM) has grown significantly. FEMs can be used to analyze various types of dams, with processing time varying based on the complexity of the model. FEMs used to inform dam safety decisions now contain over two million elements, with the potential to increase depending upon project complexity. These models are generally used to assess the likelihoods of spillway-related seismic potential failure modes (PFM) and incorporate features such as the spillway, dam, reservoir, and surrounding landscape.
Dynamic analysis run times can take as long as one day per second of ground motion time history. This means that running a simulation of a 21-second-long earthquake can require as much as three weeks of computing time. If a dam should happen to be located near a subduction zone area, in which ground motions can last as long as 300 seconds, computations may require months to run. Such complex models are time-consuming in their initial mesh generation, and as run times protract, they can extend project schedules in the latter stages as well. In an effort to deliver the greatest value to every client, this research was undertaken to investigate and test smaller FEMs and asses their quality and accuracy. Such efforts could have the potential to reduce cost to the client.

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Dam Safety Technology Development Program

Waterways and Concrete Dams Group, 86-68110

DSO-2018-05

Finite Element Model Size Reduction Research and Validation Study

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Acronyms and Abbreviations

TM – Technical Memorandum

FEA – Finite Element Analysis

SPC – Single Point Constraint

CSCM – Continuous Surface Cap Model

K&C – Karagozian & Care Concrete Model

CRB – Consultant Review Board

EW – East-West

NW – Northwest

Vert – Vertical

FEM – Finite Element Model

Keywords

Finite element model, reduced finite element model, structural analysis.

Team Members, Partners, Reviewers

Table 1. Internal members, partners, reviewers.

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Background

In the past years, the scope of finite element models (FEM) has grown significantly. FEMs can be used to analyze various types of dams, with processing time varying based on the complexity of the model. FEMs used to inform dam safety decisions now contain over two million elements, with the potential to increase depending upon project complexity. These models are generally used to assess the likelihoods of spillway-related seismic potential failure modes (PFM) and incorporate features such as the spillway, dam, reservoir, and surrounding landscape.

Problem

Dynamic analysis run times can take as long as one day per second of ground motion time history. This means that running a simulation of a 21-second-long earthquake can require as much as three weeks of computing time. If a dam should happen to be located near a subduction zone area, in which ground motions can last as long as 300 seconds, computations may require months to run. Such complex models are time-consuming in their initial mesh generation, and as run times protract, they can extend project schedules in the latter stages as well.

In an effort to deliver the greatest value to every client, this research was undertaken to investigate and test smaller FEMs and asses their quality and accuracy. Such efforts could have the potential to reduce cost to the client.

Objective

The purpose of this technical memorandum (TM) is to document and compare the results from reduced models and their larger, more extensive models, provide an insight to the parallel research efforts to validate and verify model responses, and introduce initial efforts taken to investigate the efforts of boundary conditions, size and ability to match ground motions.

Methods

Two projects were selected for inclusion in this TM: Boca Dam and Monticello Dam. Boca Dam is a zoned earthfill embankment in California and the analysis documented herein is a soil-structure interaction problem focusing on behavior of the spillway crest structure walls under seismic loads. Monticello Dam is a concrete medium arch dam located in California. In 2015, strong motion instrumentation at the dam captured seismic activity as well as structural response. The response was assessed during the 2016 USSD Annual Conference and Exhibition through participation from various analysts from across the engineering industry [1].

While details regarding the analysis performed for Boca, as well as Monticello, will be provided in the report, the full analysis can be reference in the Nonlinear Dynamic Analyses of Boca Dam Spillway [3] and Nonlinear Dynamic Analysis of Monticello Dam Response [2]. Conclusions and recommendations from the additional sensitivity analyses performed following the 2016 USSD conference and continued efforts by Reclamation to match the field earthquake response at Monticello Dam will be discussed as well. Finally an overview of the conclusions and recommendations for additional research will summarize the current state on the issue.

Research Discussion

Finite Element Analyses

The mesh generator, TrueGrid, a, was used to create the 3-D finite element model of the dam, reservoir, and the foundation with surrounding topography for both Boca Dam and Monticello Dam [4]. Both models and their analysis results are described in further detail in the sections that follow.

Boca Dam

Boca Dam was completed in 1939 and is a zoned, rolled earthfill dam in California. The spillway crest structure is situated on a knoll between the main dam embankments and the dike, and the walls of the gated crest structure are directly adjacent to the embankments' impermeable core material. Since the spillway crest structure is in such a critical location, Reclamation has been assessing the likelihood that seismic failure of the spillway crest structure walls could lead to a failure of the dam embankment.

2017 Structural Dynamic Analysis

Since 2009, Reclamation has been working to better understand the risks associated with the potential seismic failure of the spillway crest structure walls at Boca Dam. This first evaluation used pseudo-static methods to compute seismic earth pressures and compared these loads to the spillway crest structure wall capacity in order to develop risk estimates. These methods are quite conservative and estimate high values for seismic earth pressures with unfavorable distribution of those loads.

As Reclamation moved forward with efforts to better understand and reduce the risks associated with a possible seismic failure of the spillway crest structure walls, an FEM was developed for use in risk analysis and evaluation of alternatives for dam safety modification of the spillway crest structure.

Model

Concrete Spillway Crest Structure

The suite of analyses focuses on the reinforced concrete spillway crest structure because this structure directly abuts the impermeable core material of the dam and its failure during a seismic event could result in an uncontrolled release of the reservoir. The spillway crest structure model consists of the ogee crest, gates, and wall panels supported by footings with three counterforts. The spillway crest structure is modeled with brick elements, the reinforcement being represented by truss elements, as seen in Figure 1 left and right, respectively.

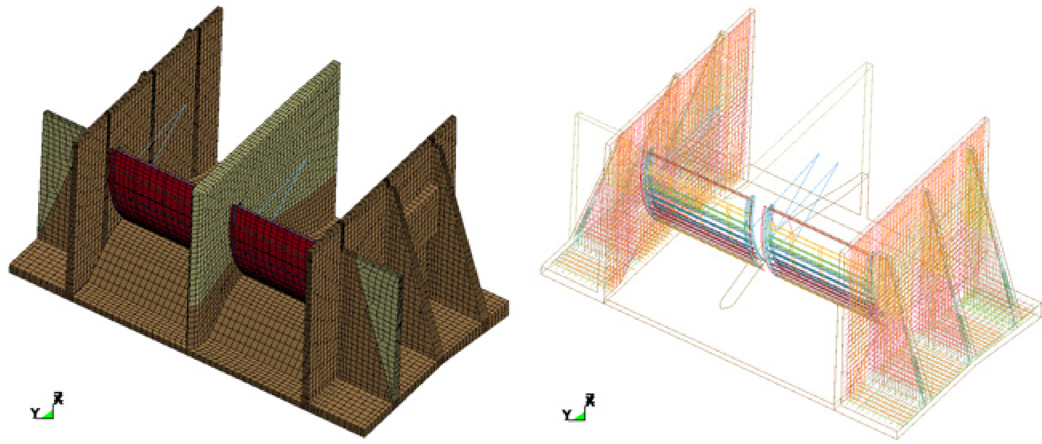

Figure 1. The LS-DYNA Model of the Spillway Crest Structure

Dam and Dike Embankments

The 3-D FEM of the dam and dike consits of four zoned materials, using soild elements with nonlinear soil properties. Material properties were assigned based on site invesitgation and laboratory testing. See Figure 2 for a representation of the dam and dike as viewed from the right abutment foundation.

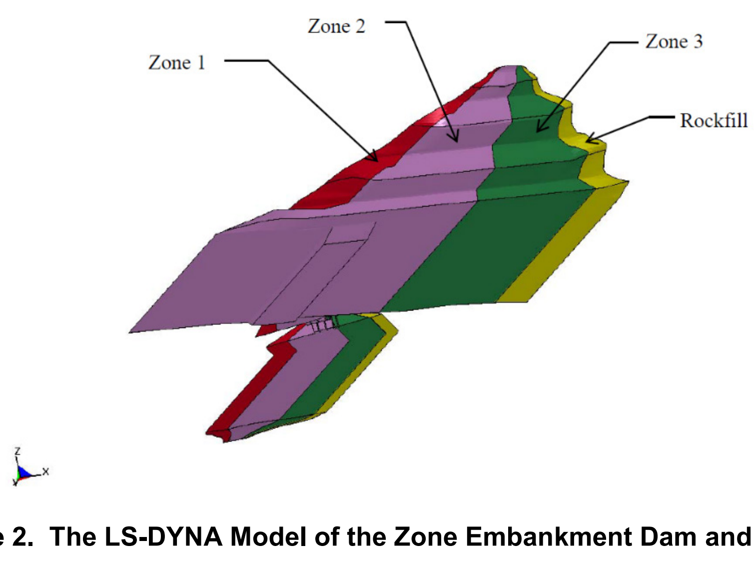

Figure 2. The LS-DYNA Model of the Zone Embankment Dam and Dike.

Foundation

The foundation at Boca is copmosed of several materials. The tuff and tuff breccia bedrock (Tt/Tv), shown as the brown material in Figure 3, is modeled by linear material solid elements. In addition, there are quaternary units including channel alluvium (Qal), Donner Outwash (Qdo), and two Tahoe Outwash units (Qto1 and Qto2). All of these units are modeled with solid elements and a nonlinear soil material. Between the two layers of quaternary Tahoe Outwash is a narrow subunit of Basal Sand. This subunit, being a 1.5- to 3-foot-thick layer, is modeled by a sliding contact interface using a coefficient of friction computed from the estimated friction angle of the Basal Sand. Additional information on foundation materials is availabel in TM No. BOC-86-68110-FD-2017-01 [3].

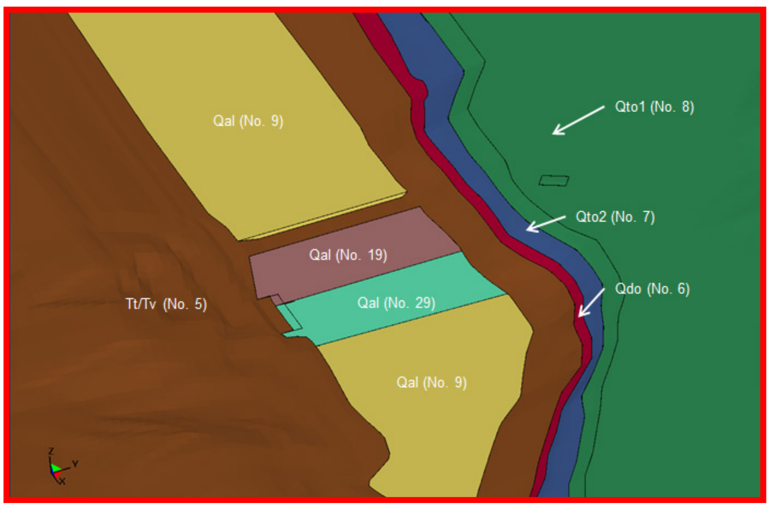

Figure 3. The LS-DYNA Foundation Model Parts.

Reservoir

The reservoir behind the dam and dike was modeled using solid elements with a fluid equation of state. The inlet channel reservoir adjoining the gates is also modeled with the same elements and fluid parameters. The reservoir is shown in blue in Figure 4.

Ranges of Finite Element Models Studied

There are three ranges of FEMs studied in the analyses. The large extensive (baseline) model shown in Figure 5(a), has a footprint of 490 acres. This model includes all foundation parts and deconvolved ground motions were applied 1 element up from the base of the foundation bedrock. The medium model used was not as extensive as the large model, but contained all of the needed foundation, embankment and reservoir parts. It will be referred to as the reduced complete (truncated) model, shown in Figure 5(b), and has a footprint of apporximately 15 acres. Deconvolved ground motions are applied in the same manner for the reduced complete model as for the large extensive model. The reduced partial (simplified) model, contained only two foundation layers and limited portions of the dam, dike and reservoir, as seen in Figure 5(c). The free field motions were directly applied at the base of the model (Qto2 layer), and this model has a footprint of approximately 3 acres.

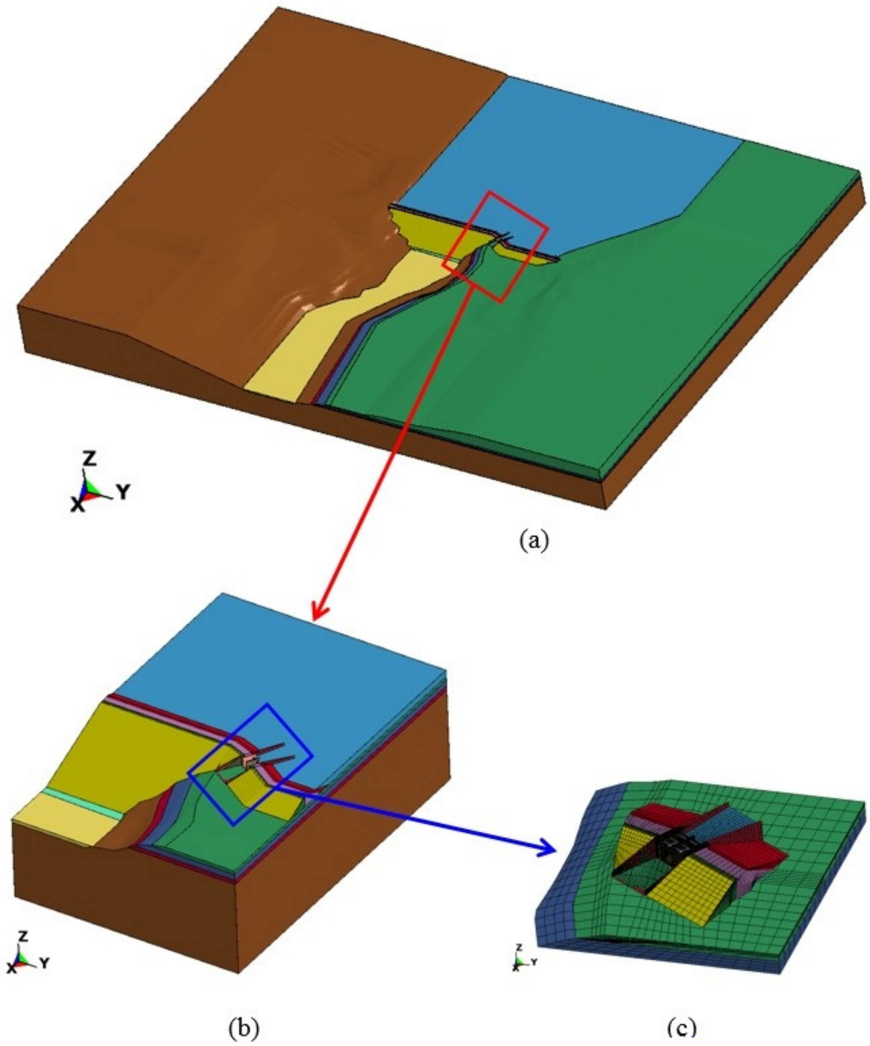

Figure 4. The LS-DYNA FEMs of Boca Dam (a) Large extensive model (Baseline), (b) Reduced complete model (Truncated) and, (c) Reduced partial model (Simplified).

Loads

During final designs the Boca FEMs were analyzed for the Reclamation series of local (KOZ, NIS and STU) seismic events with 10,000-, 20,000-, and 50,000-year return periods. The local events had slightly higher peak accelerations with shorter durations (up to 23 seconds) of strong shaking. For the purpose of this research, the NIS results were used in comparisons.

Seismic motions in LS-DYNA were applied as 3 deconvolved, orthogonal stress time histories at 0.01-second time step along a horizontal layer of element faces at a depth of about 240 feet below the Qdo in the Tt/Tv foundation, and allowed to propagate up the ground surface in both complete models. For this reason, it was important to make sure the computed motions at the rock surface near the spillway in the FEM matched the free-field motions supplied by the seismologists [6].

Foundation-only FEMs with the same mesh density as the complete ones, also known as a Flat Box model, were used for the deconvolving analyses. The deconvolved stress time histories were applied at depth and measured at the free surface, the top of the Tt/Tv, in Boca Dam model. The free surface is flat to match the fundamental assumptions made by the seismologists in developing the ground motions. The response was measured at a node near the spillway at the top of the Tt/Tv.

The simplified model (Figure 4c) does not include the bedrock (Tt/Tv) part. The parametric studies done with this FEM applied the free field motions themselves directly to the Qto2 foundation layer, shown in blue in Figure 5c.

Response Spectra

Well defined load inputs and reasonably accurate boundary conditions are critical to obtaining results that risk analysis teams and decision makers can apply with confidence. In seismic finite element analysis (FEA) ground motions are applied at a depth in the foundation, through which they propagate to excite the dam, reservoir, and spillway. Examining the response spectrum, a plot of the peak acceleration response for oscillators at varying natural frequencies, can help engineers gage the overall accuracy of: 1) the loading to the structure of interest, 2) the FEM's boundary conditions, and 3) the resulting structural response.

Figures 5 and 6 show the 1,000-year return period free field acceleration response spectra at five percent damping of the NIS earthquake for the baseline model and the truncated model, respectively. The free field records are shown with solid lines and can be compared to the computed response at the surface of the rock, shown with the dotted lines.

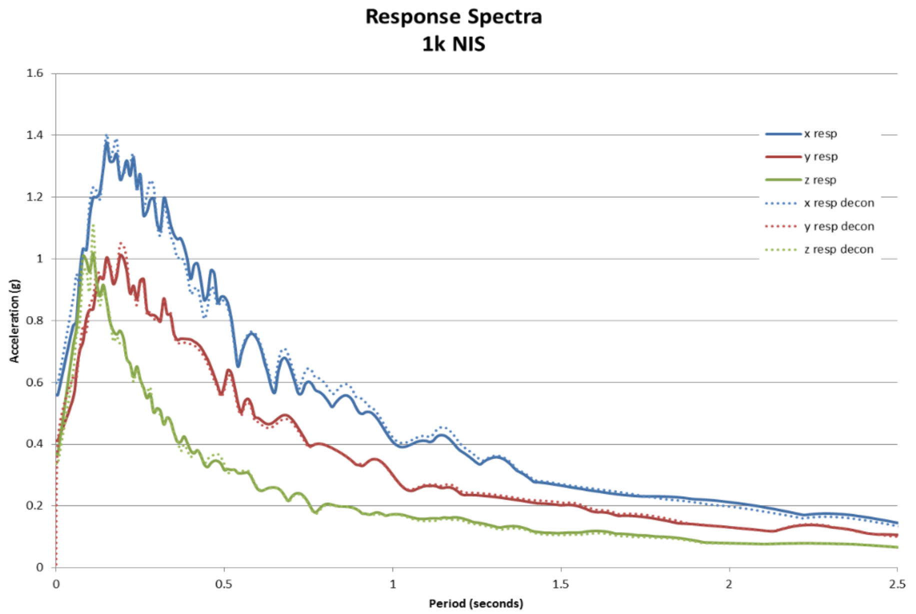

Figure 5. Response spectra for free field and deconvolved ground motions in the large extensive FEM.

Figure 5 shows the results for the baseline model and acceleration peaks of the deconvolved ground motions are plotting close to those of the free field motions for most periods of oscillation. For many of the shorter periods, the accelerations match very closely in all directions and they are reasonably close for the longer periods as well.

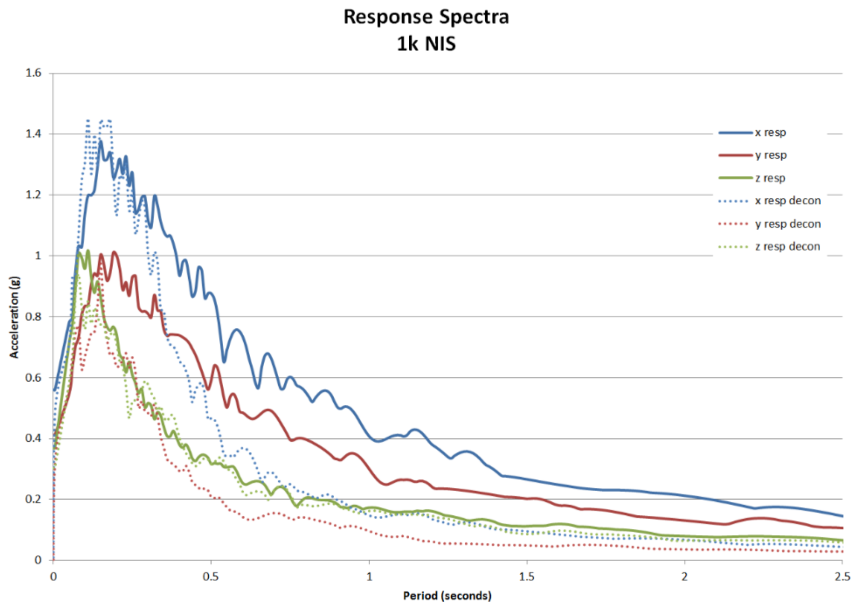

Figure 6. Response spectra for free field and deconvolved ground motions in the reduced complete FEM.

In Figure 6 the acceleration peaks of the deconvolved ground motions applied to the truncated FEM do not match those of the free field motions as well as those seen in Figure 5. The matching in the z-direction for the truncated model is better than in any other direction; however, it still does not agree with the free field as well as the baseline model does. The model is calibrated in the z-direction by way of establishing the element size based on the shear wave speed of the bedrock and the depth of application of the ground motions. Perhaps this is why the match is the z-direction is somewhat better. For periods of oscillation of less than 0.5s, the horizontal components of the ground motions achieve matches on a few occasions but are still not considered to be close. The longer period of the horizontal components are off by a considerable measure.

Results

In order to investigate the effect of reducing the FEM for Boca Dam, several parameters were compared. Comparison included the standard baseline model, truncated model and simplified model (Figure 4) limited to the 10,000-year return period. In addition, two separate material models were applied and are briefly described below.

Model 72R3: The Karagozian & Case concrete model, release III. This model is a three-invariant model and uses three shear failure surfaces, including damage and strain rate effects. The damage is displayed on a scale from zero to two; two indicating that the element completely cracked or crushed. This model has the ability to generate parameters based solely on the unconfined compressive strength of the concrete. For more details on this model, the reader is referred to the Users and Theoretical Manual [7].

CSCM: The continuous surface cap model. This model provides the user the option of inputting all the material properties, or requesting default material properties for normal strength concrete. The latter was employed for this analysis. Concrete damage is shown on a scale from zero (no damage) to 1.0 (concrete element is fully cracked). For more details on this model, the reader if referred to the user's manual for LS-DYNA material model 159 [8].

Parameters considered of interest for the comparison include: 1) the global extent of concrete damage and related stress time history plots, 2) state of the reinforcing steel, and 3) wall displacements (based on yield of steel). The following figures show a comparison among the baseline FEM, as well as the truncated and simplified models. A red line has been provided for each figure set, indicating the change in baseline results.

Figure 7 shows the time history plots for maximum principal stresses in five elements selected in the right spillway wall stem in the vicinity of the ogee crest. The elements selected are in the same location for each model type. Element A is located at the base of the wall, Element C is level with the ogee crest, and Element E is 5 feet above the ogee crest. Elements B and D and immediately below and above the ogee crest, respectively.

In the baseline model, elements in the walls near the top of the ogee crest appear to be subject to the highest stresses as seen in the time history plots in Figure 7a. At the right side of the spillway, elements C and E are reaching high maximum principal stresses just before failing relatively early. Both elements have failed before t = 9s.

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In the simplified FEM using the K&C concrete model, elements in the walls near the top of the ogee crest appear to fare better than those in baseline for the full model, as seen in the stress time history plots in Figure7b. At the right side of the spillway, element D reaches higher maximum principal stresses but all seem to ride out the shaking and provide some resistance to loading. The simplified model's spillway crest structure foundation is allowed to move much less than that of the full model, due to the boundary conditions applied for the simplified model simulations. In the full model, the Qto1 layer is restrained on left by a single point constraint (SPC) boundary condition and on the right by the loading from the dam embankment. In the simplified model, the Qto1 is more tightly restrained by SPCs on all sides. This allows the Qto1 layer, and by extension the spillway crest structure, to slide much less along the basal sand contact. As such, the spillway may not be moving into the backfill quite as hard as for the large model case.

For the simplified FEM using the CSCM concrete model, elements in the walls near the top of the ogee crest appear behave somewhat differently those in the simplified case with K&C concrete, as seen in the time history plots in Figures 7c. At the right side of the spillway, elements do not reach such high values of maximum principal stress as those in the simulation using the K&C concrete model.

The principal stresses for the truncated model using CSCM concrete show that high stresses are experienced earlier in the simulation and then drop significantly, leveling off to less than 100 lb/in^2 by time t = 10s. Figure 7d shows that element C, just above the ogee crest, is one of the more highly stressed elements. For the truncated model using K&C concrete, maximum principal stresses are shown in Figure 7e, and this plot indicates that element C, just above the ogee crest, is one of the more highly stressed elements. However, with the K&C material the trend of stress is different than that of the CSMC concrete. In both instances, the stresses are near 200 lb/in^2 near the start of the shacking. However, the stresses in the K&C simulation continue to increase and level-off at approximately 600 lb/in^2 at the end of the simulation.

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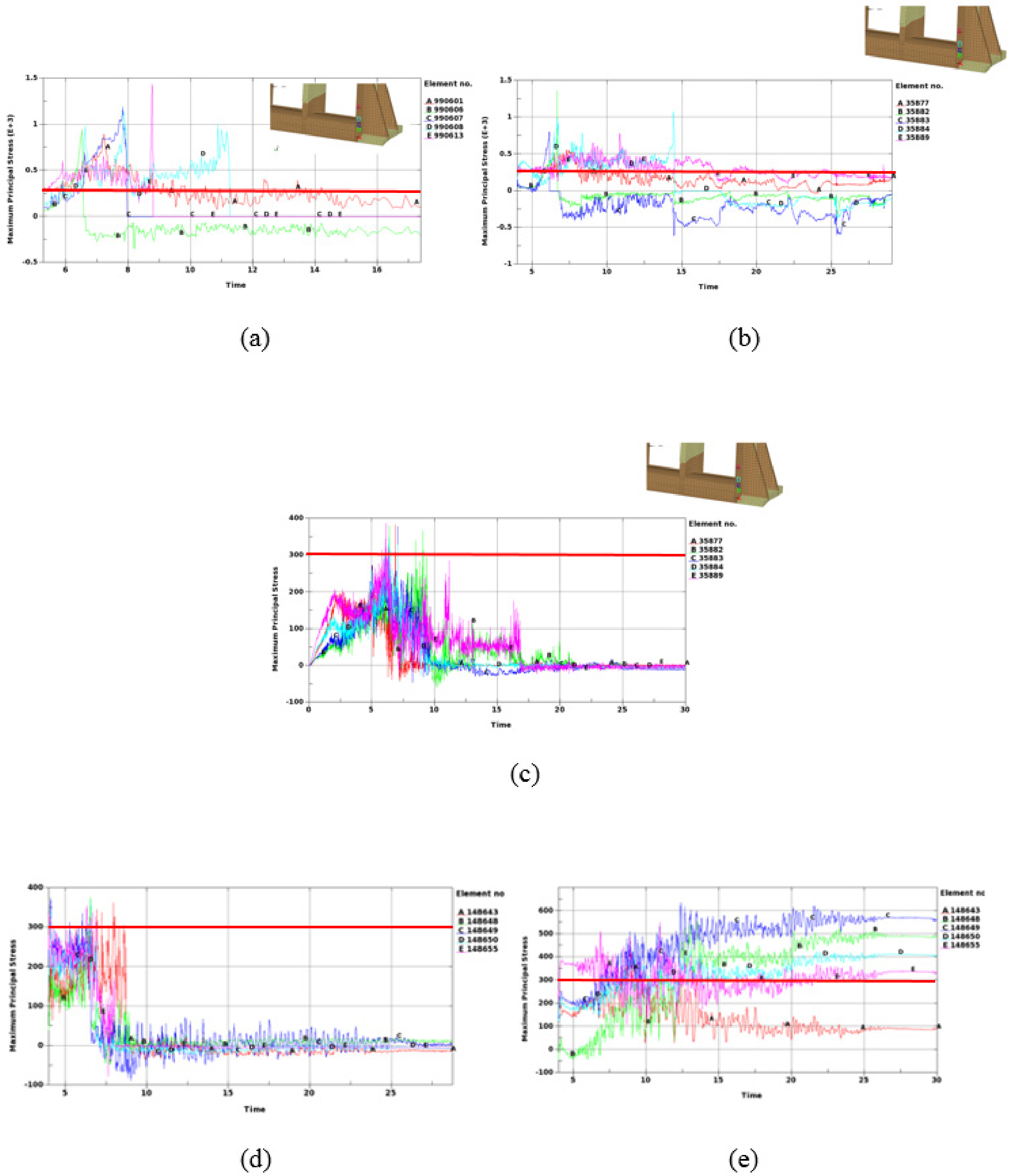
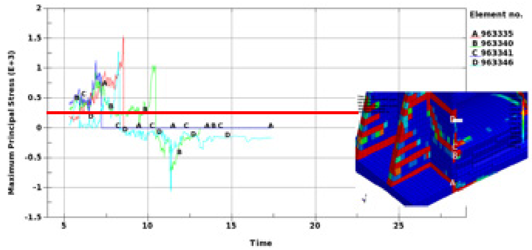


Figure 7. 10,000-year right wall maximum principal stresses (lb/in^2): (a) Baseline Model, (b) Simplified Model, (c) Simplified Model with CSCM, (d) Truncated Model with CSCM, and (e) Truncated Model with K&C. (Comparison made at principal stress = 300 lb/in^2)

Figure 8 shows the time history plots for maximum principal stresses in four elements selected in the left spillway wall stem in the vicinity of the ogee crest. The elements selected are in the same location for each model type. Element A is located at the base of the wall, Element B is level with the ogee crest, Element C is just above the ogee crest, and Element D is 5 feet above the ogee crest.

In the baseline model with the K&C concrete, elements in the walls near the top of the ogee crest appear to be subject to the highest stresses as seen in the time history plot in Figure 8a. At the left of the spillway, Elements C, just above the ogee crest, and A, at the bottom of the wall, both reach high maximum principal stresses before failing. These elements have both failed by time t = 8.0s.



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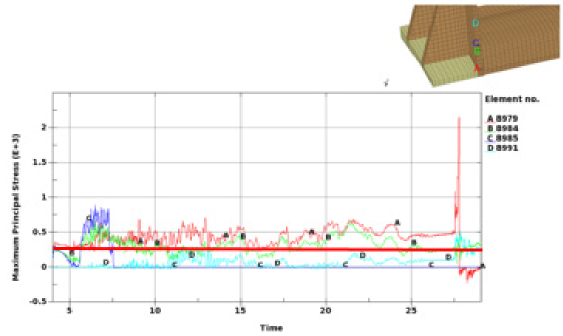

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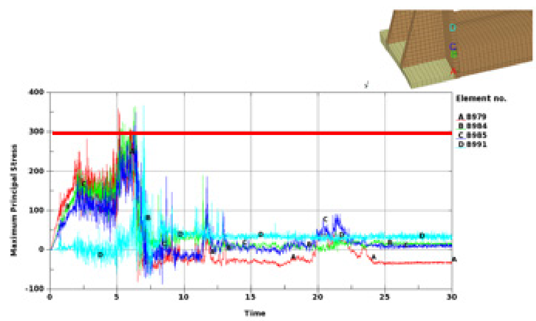

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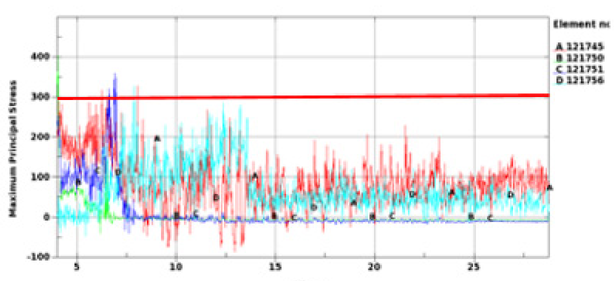

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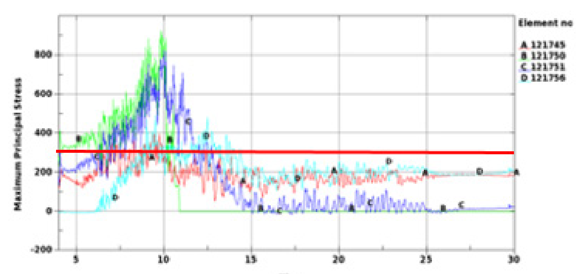


(e)

Figure 8. 10,000-year left wall maximum principal stresses (lb/in^2): (a) Baseline Model, (b) is Simplified Model, (c) Simplified Model with CSCM, (d) Truncated Model with CSCM, and (e) Truncated Model with K&C. (Comparison made at principal stress = 300 lb/in^2)

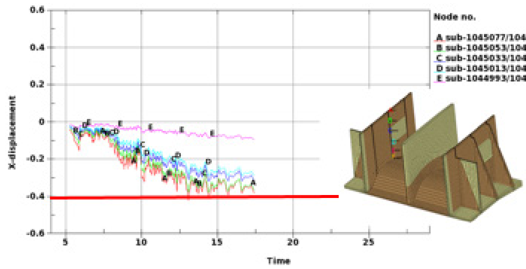
The time history for the simplified model with K&C concrete shows that element C, just above the ogee crest, reaches high maximum principal stresses before failing early in the earthquake, as seen in Figure 8b. When modeled with the CSCM material, elements in the walls near the top of the ogee crest appear behave somewhat differently, as seen in the time history plots in Figure 8c. Principal stresses, for the most part are less than 300 lb/in^2 between times t = 5s and t = 10s; generally much lower than those when the K&C concrete model is used.

Examining the maximum principal stress time history plot in Figure 8d for the truncated FEM using CSCM concrete, we see that elements abutting the ogee (B) or just above (C) experience some of the stresses, peaking near 300 lb/in^2, between times t =5s and t = 10s. However, element A just above the wall footing is more highly stressed earlier in the simulation. When this FEM used the K&C concrete, maximum principal stress values peaked at about 800 lb/in^2 near time t = 10s. Figure 8e indicates that the elements near the ogee crest (B and C) were the most highly stressed.

Figures 9 and 10 show time history results for upstream/downstream and cross-valley relative displacements, respectively. Relative displacements of the left spillway wall are found by subtracting the displacement at node F, near the ogee crest, from nodes at various heights on the wall. Figure 9a shows the Upstream/Downstream displacement time histories in units of inches and Figure 10a shows the Cross-Valley displacement time histories for the baseline model using K&C concrete. From these plots we can see that by the end of the simulation, the top of the wall has moved into the spillway channel by about 0.6 inches due to the shearing and bending of the wall along the ogee crest.

Relative displacements of the simplified modeling using K&C concrete are plotted in Figures 9b and 10b. From these plots we can see that by the end of the simulation, the top of the wall has moved into the spillway channel by about 0.7 inches due to the shearing and bending of the wall along the ogee crest. The simplified results using the CSCM model are shown in Figures 9c and 10c. The x-direction displacement are more like the simplified model with K&C concrete than are the y-displacements. The results is that the deflections of the top of the wall into the channel for this case is slightly less, at about 0.6 inches. In general, the simplified model is giving slightly higher relative displacements than the full model, but the two are reasonably close.

Relative horizontal displacements for the truncated modeling using CSCM concrete are plotted in Figures 9d and 10d. These plots indicate that the resultant displacement at the top of the wall is approximately 0.75 inches near time t = 13s; at which point they begin to settle to a final value of 0.6 inches. This pattern differs slightly from that shown in Figures 9e and 10e: the horizontal relative displacement time histories for the truncated model with the K&C concrete. Here the upstream/downstream displacements follow a similar trend, but the cross-valley displacements peak just before time t = 10s. The resultant displacements at the end of the simulation are approximately 0.7 inches. Again, this is very close to the results obtained from other FEMs using either concrete material model.



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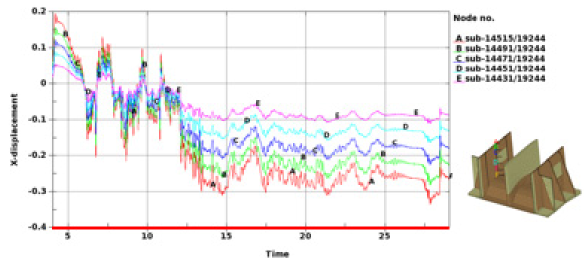

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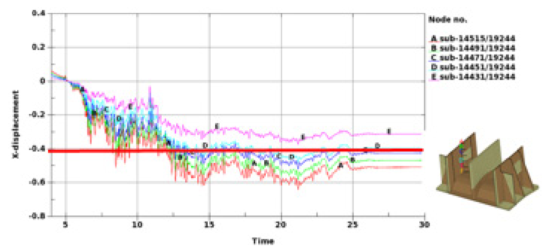

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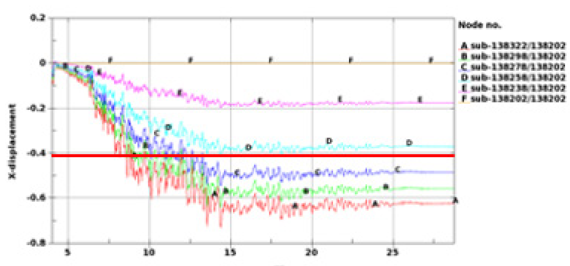

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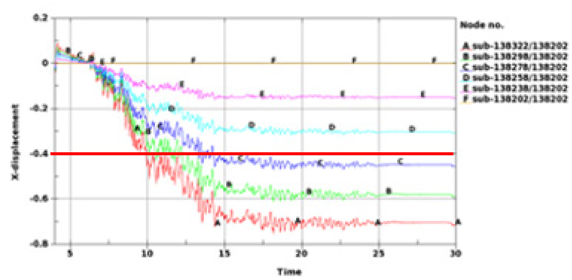
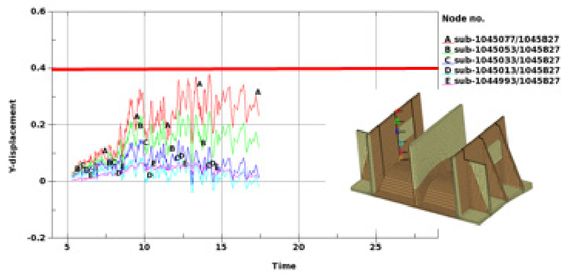


(e)

Figure 9. 10,000-year upstream/downstream relative displacements left wall (in.): (a) Baseline Model, (b) is Simplified Model, (c) Simplified Model with CSCM, (d) Truncated Model with CSCM, and (e) Truncated Model with K&C. (Comparison made at displacement = 0.4 feet)



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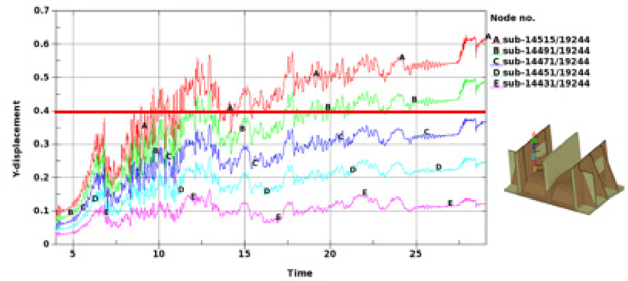

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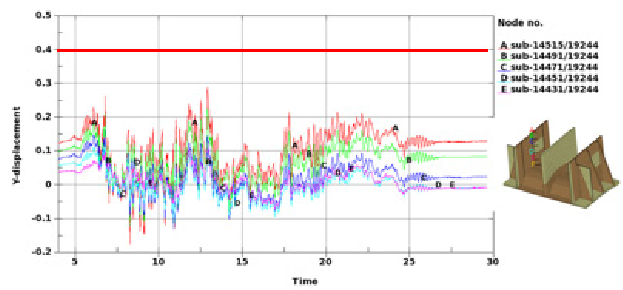

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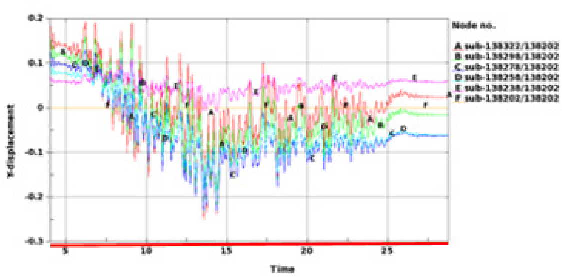

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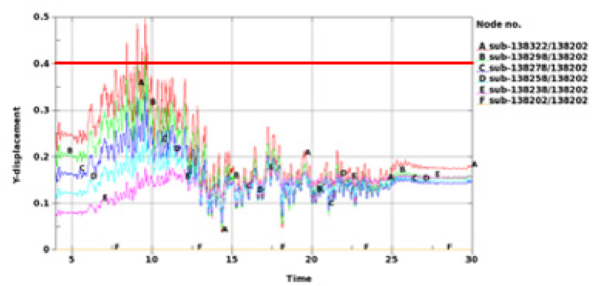

(e)

Figure 10. 10,000-year cross-valley relative displacements left wall (in.): (a) Baseline Model, (b) is Simplified Model, (c) Simplified Model with CSCM, (d) Truncated Model with CSCM, and (e) Truncated Model with K&C. (Comparison made at displacement = 0.4 feet)

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Figure 11 shows the time history of the axial forces in the reinforcing steel in the downstream, left counterfort. The elements chosen are part of the moment-resisting reinforcement, and have a yield strength of 40,000 lb/in^2 [9]. The bars in this model represent the as-built condition by splitting the area of reinforcement shown in the design drawings among the two beam elements in the model. The bars in the model at the bottoms of the counterforts have cross-sectional areas of 2.5in^2; obtained by splitting the total 5in^2, which is composed of five 1-inch square bars in the structure. The yield force in these bars, as modeled, is 100,000 lb. However, of the five bars, only two are continuous for the entire height of the counterfort. Midway up the counterfort,

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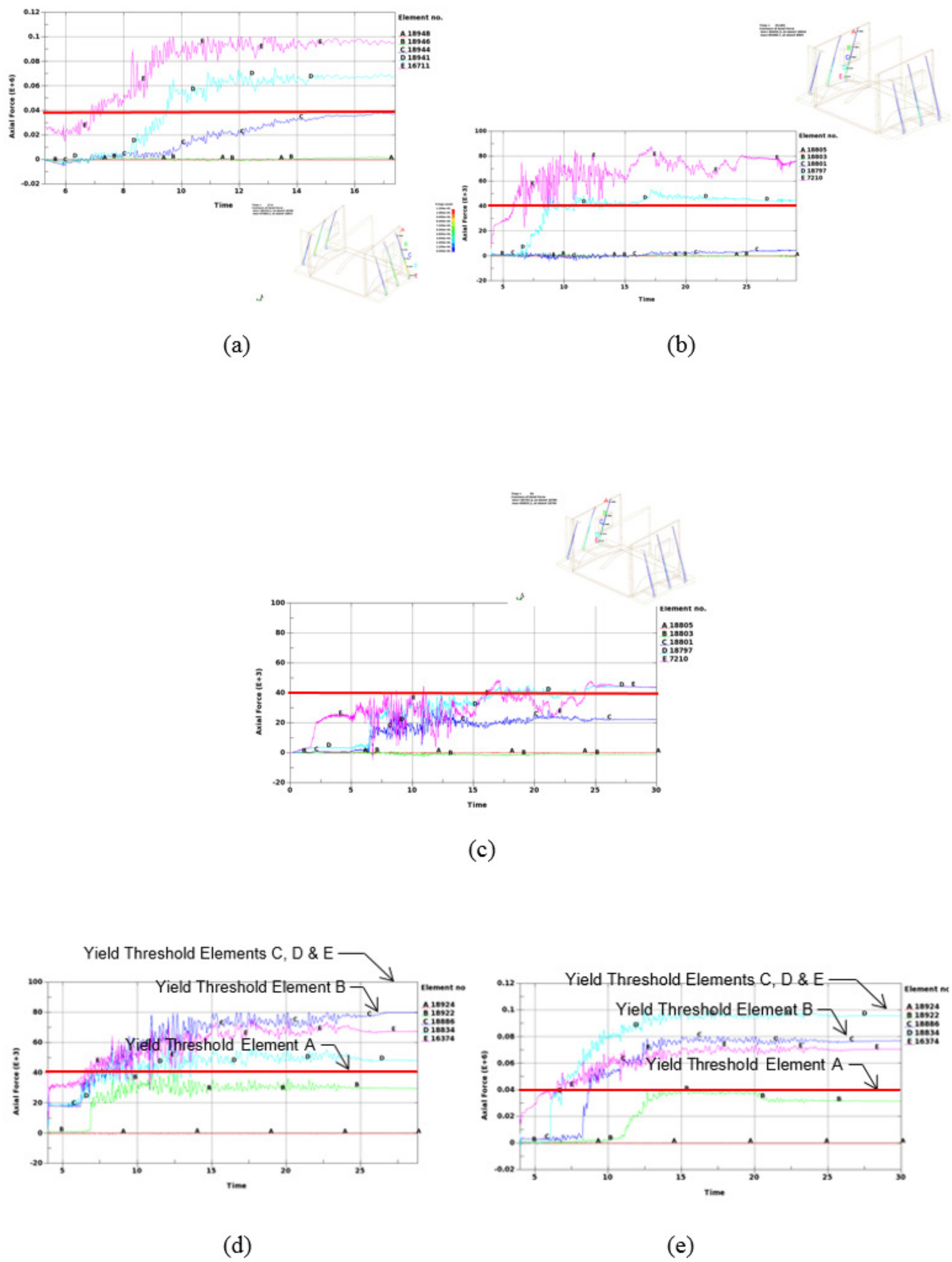

Figure 11. 10,000-year axial force time histories in right, downstream counterfort moment steel (lb.): (a) Baseline Model, (b) is Simplified Model, (c) Simplified Model with CSCM, (d) Truncated Model with CSCM, and (e) Truncated Model with K&C. (Comparison made at yield strength = 40,000 lb/in²)

There is a total of four 1-in^2 bars providing moment resistance. These are modeled by two sets of beam elements having a cross-sectional area of 2in^2, and thus they have a yield force of 80,000 lb. Finally, in upper portion of the wall, there are two bars modeled by two sets of beams elements have a cross-sectional area of 1in^2, and a corresponding yield force of 40,000 lb.

In Figure 11, the beam elements are labeled from A to E where Element A is near the top of the counterfort and element E is near its base. The results baseline model using K&C concrete is shown in Figure 11a. Here we see that element E is experiencing the most force, nearing its yield strength at time t = 8s and oscillating near that value for the remainder of the earthquake. Element D follows a similar pattern, but the rest of the selected elements are comfortably below yield.

Figures 11b and 11c show the axial force time history for the same elements in the simplified model using K&C concrete and CSMC concrete, respectively. In the lower portion of the wall, Elements E and D are experience greater loads for the K&C modeling case. The values for the K&C case are more in keeping with the pattern seen in the baseline simulation; however, the values for the CSCM model with the simplified FEM are significantly lower.

Figure 11d and 11e show the axial force time histories for the truncated FEM using the CSMC and the K&C concrete materials, respectively. These plots are generally more like those resulting from the baseline model, than the simplified model, indicating a more desirable performance. However, there are still some inconsistencies. Element E performs similarly for each of the concrete materials, but is experiencing only 80% of the force seen in the baseline model.

Figure 12 shows the damage contour plots for the three model types. Generally, the models show damage accumulating in the same areas of the spillway. The counterforts show cracking in the bottom one third and walls crack in the vicinity of the ogee crest. The FEMs that applied the K&C concrete material were the ones most likely to exhibit numerical instabilities. The risk team concluded that the damage that occurred near the upstream counterfort in the left spillway wall, in Figure 12a, was of a spurious nature and adjusted risk estimates accordingly. Similarly, in the truncated model with K&C concrete, Figure 12c, the heavy damage on the right side of the spillway was traced back to numerical instability.

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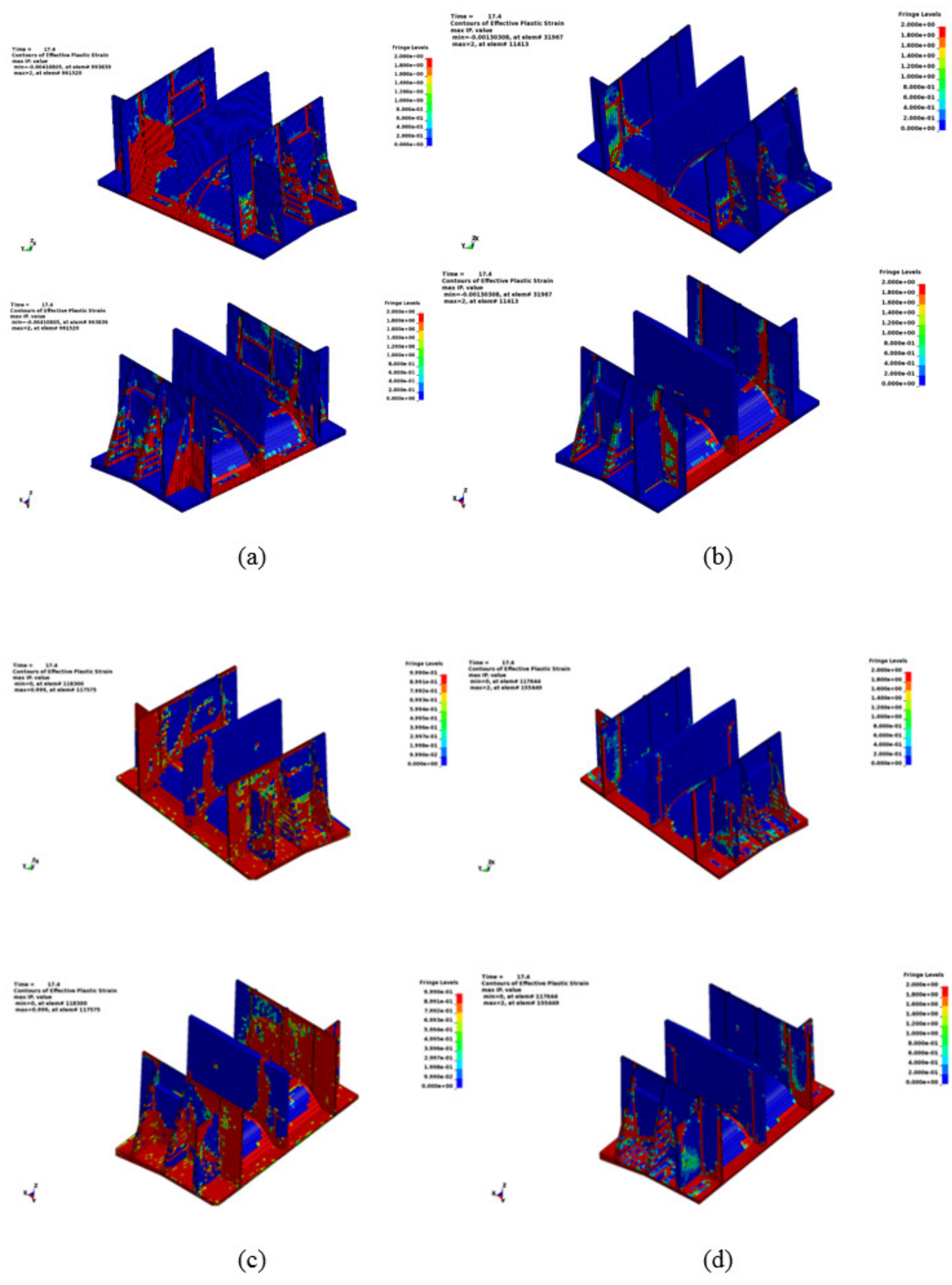

Figure 12. 10,000-year concrete damage contours from right (top) and left (bottom) (t=17.4s): (a) Baseline Model, (b) Simplified Model, (c) Truncated with CSCM, and (d) Truncated with K&C. (Comparison made at time step = 17.4 sec)

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Summary and Conclusions

For the purpose of the finite element reduction research, three models sizes were selected to investigate the effect on result compared to the typical full model scenario, as recommended by an independent consultant review board (CRB) assessment of the Boca Dam analysis (Figure 4). The two models used for comparison efforts for this results were the simplified and truncated, with the baseline model size.

Based on the parametric study with reduced model sizes, the simplified model was a very useful tool for evaluating impacts to model results from changing input parameters. One of the most meaningful changes was modeling the wall footings with a nonlinear material type, as compared to a linear elastic material type. In general, when the material in the wall footings is changed from linear to nonlinear, stress redistribution occurs and results in lower stress values in the wall elements. In the case of Boca Dam, this change resulted in less stress being transmitted to the walls and reinforcement when the footings were allowed to accumulate damage [3].

Additional observation from the Boca Dam Final Design analyses noted that the simplified model, in general, gave less conservative results as compared to the baseline model. As a result, the simplified model could be used to conduct sensitivity studies, but should not be used in inform estimates in a team risk analysis. In addition, it appeared from the reduction studies that global kinetic energy plots indicated that global kinetic energy resulting from the earthquake becomes trapped within the smaller models. Therefore, their results become less trustworthy.

Monticello Dam

Monticello Dam is located on Putah Creek in northern California, approximately 10 miles west of Winters, California. The dam is a concrete arch dam with a structural height of 304 feet, a crest length of 1,023 feet and a crest elevation of 456.0 feet. The crest of the dam is 12 feet wide, and contains 4-foot-high parapet walls.

2016 Structural Dynamic Analysis

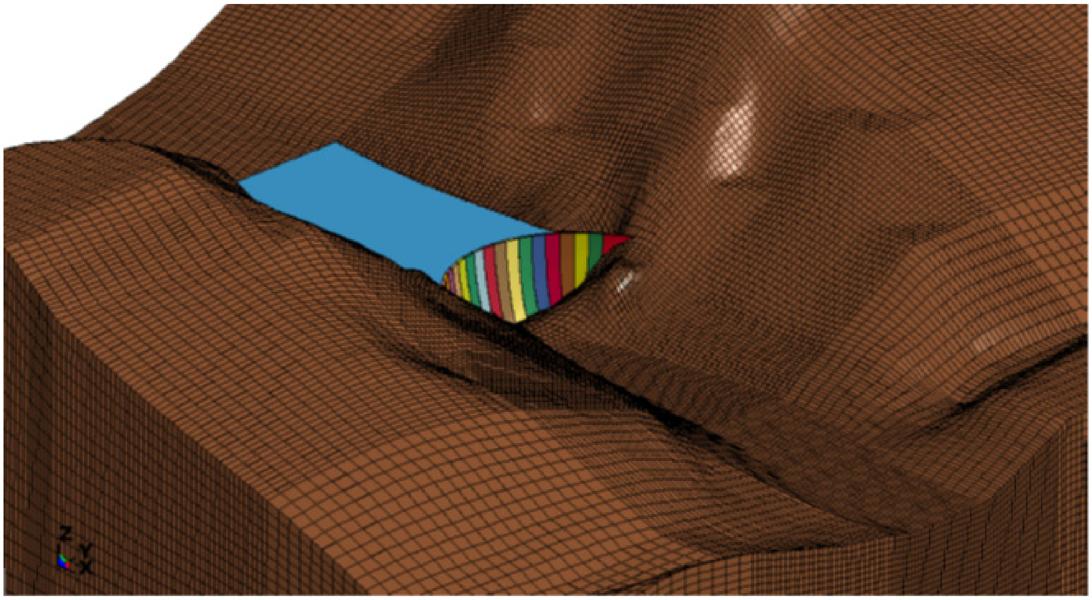
Based on the available evidence, the bedrock units beneath the dam are fresh and tight. Joints in the sandstone do not continue into the siltstone beds. The joints are closed at shallow depths (from less than 10 feet to 50 feet) are free of coatings and infilling. Bedding is homoclinal, with steep downstream dip and strike directly transverse to the valley. There are no continuous flat-lying joints mapped in the right abutment, and flat-lying joints mapped in the left abutment cannot creat blocks which daylight on the abutment slope. Shear zones are few and of short extent, and are mainly oriented sub-parallel to bedding and may contain fractured calcite or clay goud=ge up to 4- to 6-inches thick.

In 2015, Monticello Dam in California, experienced an M 4.1 earthquake with the epicenter approximately 16 km away from the dam. There was instrumentation on both the foundation and the dam crest that recorded the event. In 2016, the Earthquakes Committee of USSD held a workshop to examine the predictions of multiple organizations from around the world, using various numerical prediction codes and the same input motion. Each analyst was given a packet of information which contained drawings of the dam, material testing results, and the free field rock time history. The analysts were each responsible for building a model that they judged to be sufficient to predict the response of the dam from the provided information. All analysts submitted their predicted response one month prior to the workshop for compilation. Each analysis type, assumptions, and results were presented and the actual response was unveiled in the comparison. The group then discussed the various approaches and results of all submittals and identified areas where further improvement in analysis methods could be made.

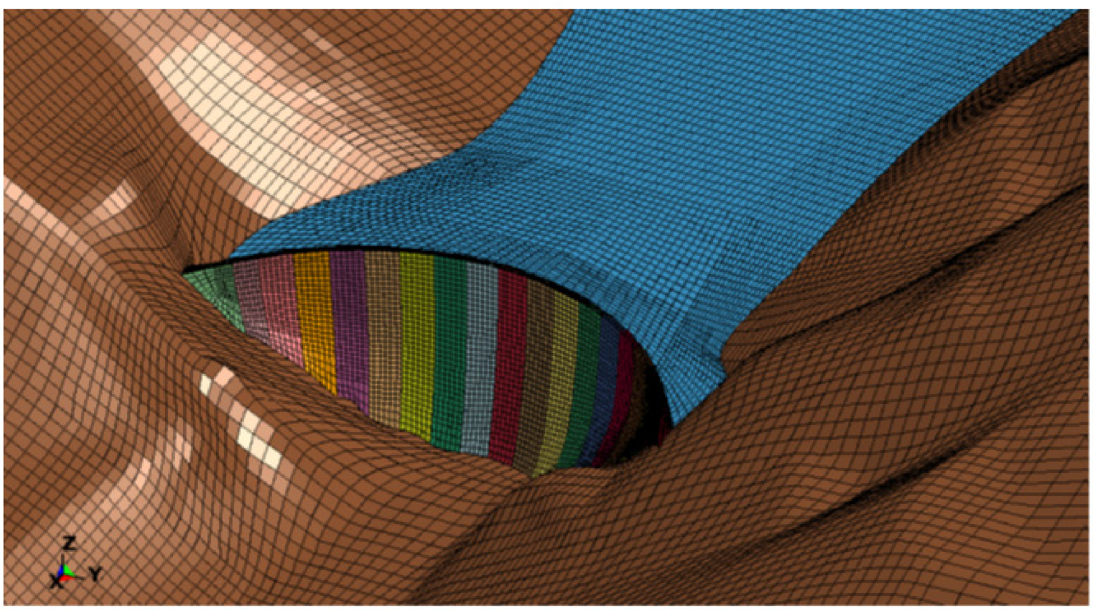
USBR Model

Concrete Dam

The concrete dam consisted of all 17 monoliths and two abutments. These monoliths had contraction joints between them that were represented with contact surfaces. The shear keys were not modeled, since it was expected that the dam would remain in compression during the seismic events.



(a)



(b)

Figure 13. General View of the 3-D Model

Foundation

The main bedrock units in the foundation footprint consist of thick beds of sandstone and slightly lesser thickness of siltstone. The foundation rock is firm, competent, and essentially water tight. The beds strike nearly normal to the creek, approximately north-south, and dip steeply, 75 degrees east and downstream.

Orientation data on specific geologic features were developed from numerous construction photographs and geologic maps, and were used to define deterministic potential foundation wedges. Considerable efforts were made to perform a realistic assessment of joint continuity using geologic interpretation. It was assumed that the steeply-dipping bedding places could form a release plane, the relatively flat-lying joint sets could form a base place, and the stress relief joint sets could act as a side plane. The base and/or side planes act as potential slide planes, and the release plane limits the size of the wedge at the upstream end. Other downstream foundation blocks may be dependent upon various shear zones on both abutments. Considering the various features of the foundation and the potential impact, an elastic and monolithic model was deemed an appropriate method for modeling the foundation.

Loads

On May 22, 2015, Monticello Dam experienced a magnitude 4.1 earthquake with the epicenter located approximately 16 km away from the dam. There was instrumentation on both the foundation and the dam crest that recorded the event. Figure 14 shows an aerial view of the dam and the location of the two instruments that recorded the earthquake. Point MONF was identified as the instrument located on the foundation that best recorded the shaking. The quality of the records taken during the earthquake presented a unique opportunity for the structural analysis community to better understand how the assumptions made in analyses impact the prediction used in decision making.

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Figure 14. Aerial View of Monticello Dam and Seismic Instrumentation

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Response Spectra Plots

Figures 15 and 16 show the site specific and measured motion at Monticello Dam. The response spectra include 3 orthogonal directions. Directions included East-West (EW), North-South (NW), and Vertical (Vert). In comparing Figures 15 and 16, the response is considered very good. Of note, the fundamental frequency of the dam was estimated to be within the range of 3.13 Hz, or 0.32 seconds.

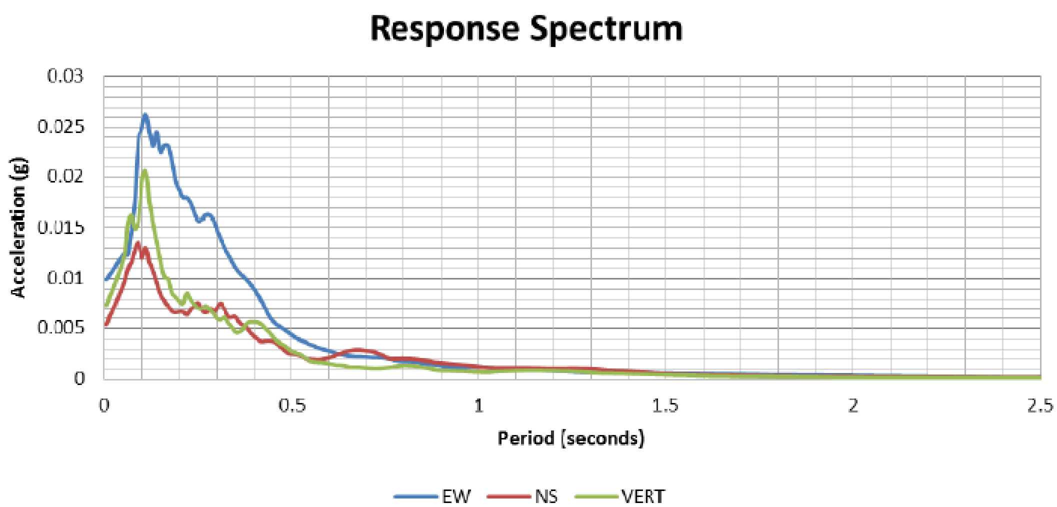


Figure 15. Three-Dimensional Response Spectra for Measured Motion at the Base of Monticello Dam

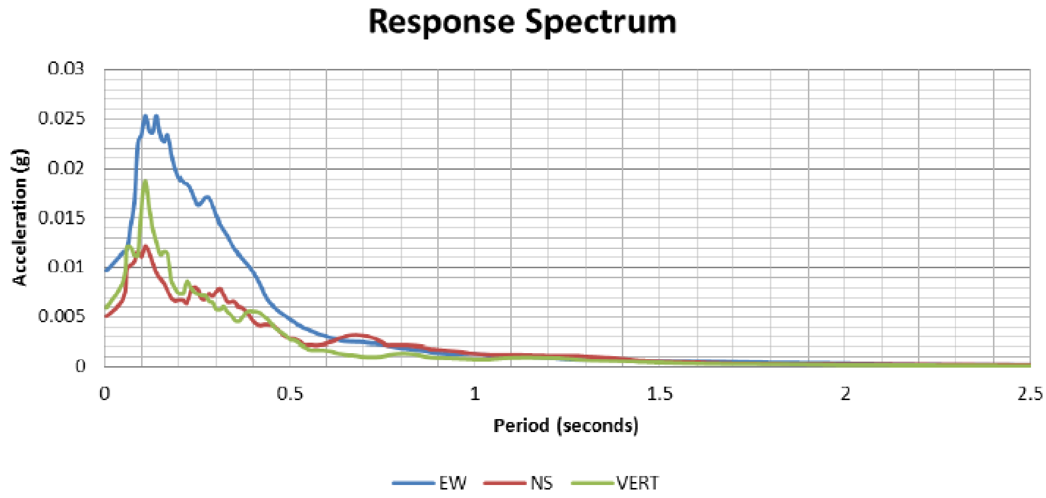


Figure 16. Three-Dimensional Response Spectra for Captured Motion at the Rock Base of Monticello Dam FEM

Results

Investigation with the Monticello Dam model had an objective to match the response spectra for the dam to the response spectra related to the measured motion, which occurred in 2015. In doing so, several parameters such as damping, foundation and dam stiffness, and nonlinear parameters allowing the concrete dam to crack were considered. Results presented are referenced from the nonlinear dynamic analysis of Monticello Dam [2] for the MONC location at the crest of the dam along monolith 11. Additional locations are discussed in the nonlinear dynamic analysis of Monticello Dam [2] for reference.

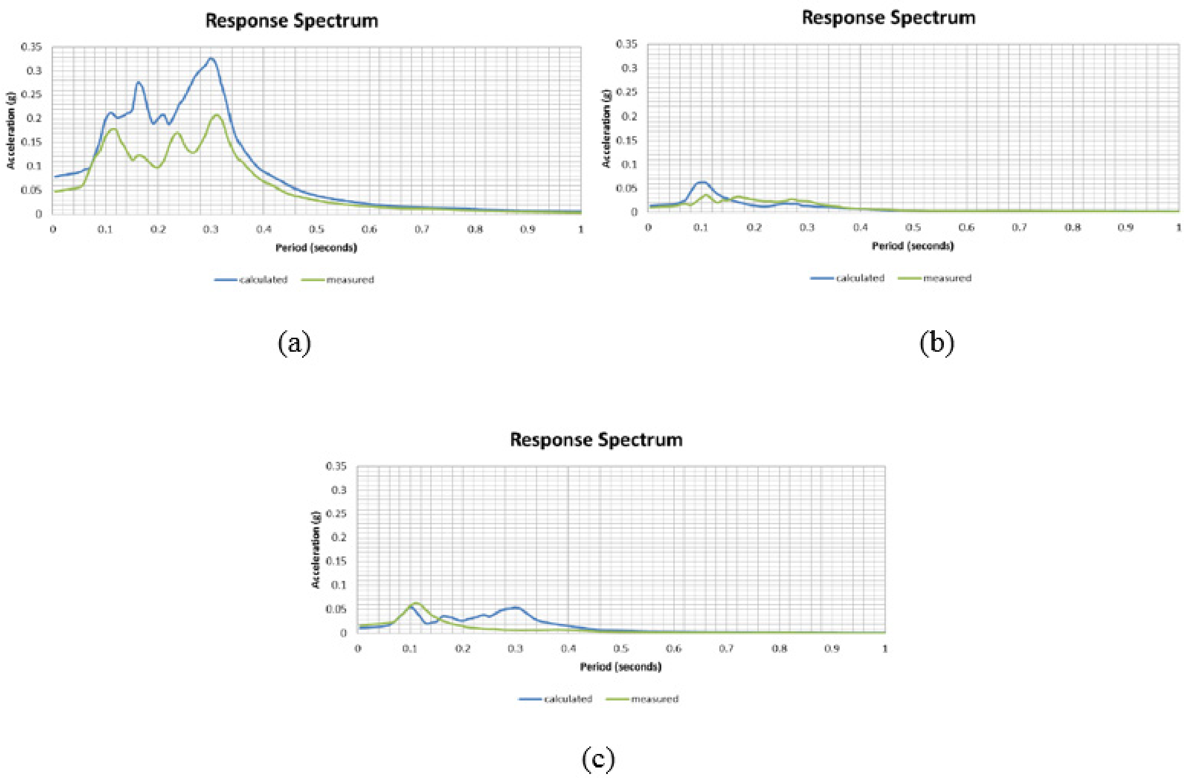
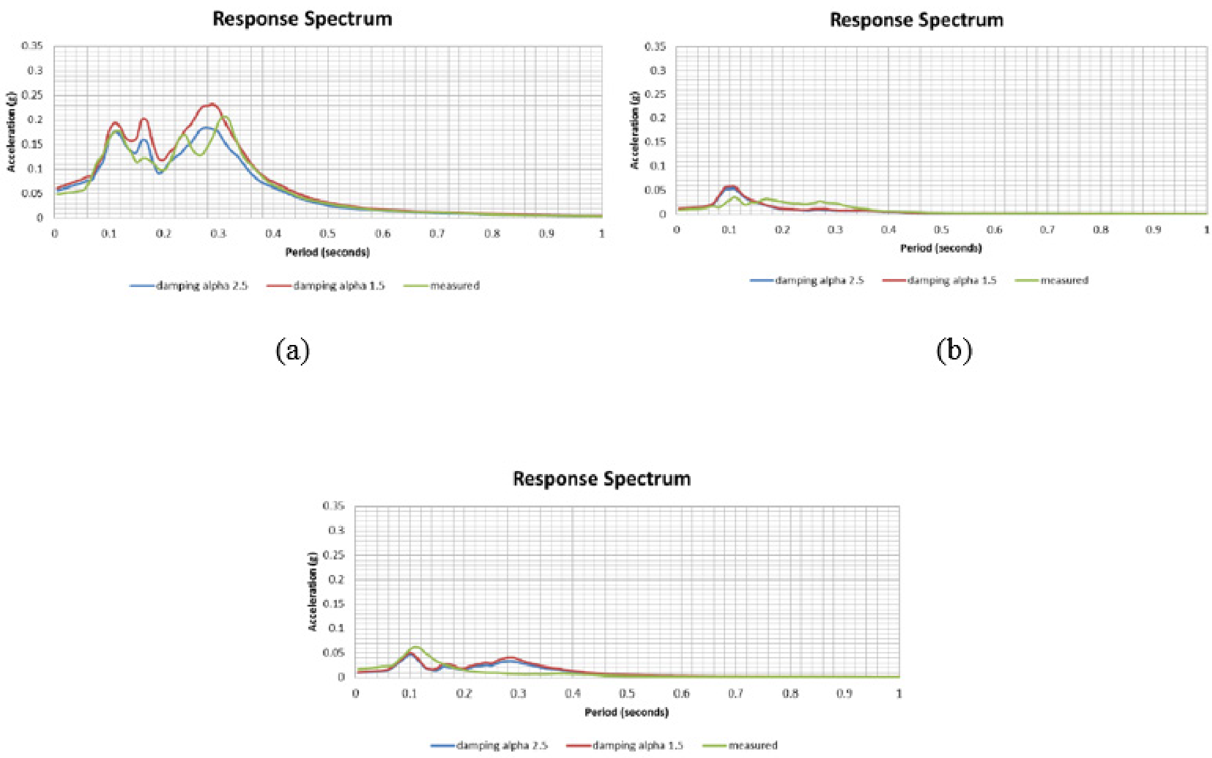


Figure 17. Measured Response Spectra at Dam Crest at Monolith 11: (a) EW, (b) NS, (c) Vert.

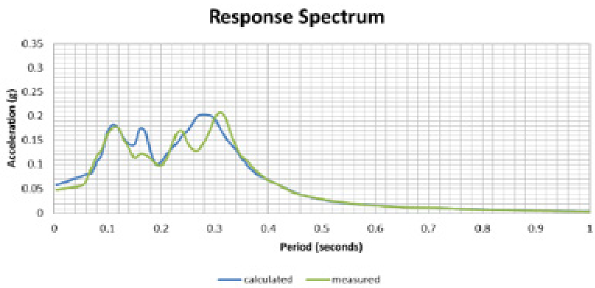
From these results, a general observation can be made that the measure motion in the FEM is higher, indicating more energy in the system, then the actual site motion. In the context of results anticipated from the FEM, one might observe higher, more conservative displacements. Typically, based on current Reclamation practice, a damping value of 5% is normally used. In order to establish a reasonable range of damping, 3 to 5% (alpha of 1.5 to 2.5) was selected for the purpose of the sensitivity study on Monticello Dam. Figure 21 illustrates the measured response for alpha range.



(c)

Figure 18. Measured Response Spectra at Dam Crest at Monolith 11 with Rayleigh Damping, alpha = 1.5 and 2.5: (a) EW, (b) NS, (c) Vert.

From the results, the response appears to have improved, with near matches in all 3 directions. An additional adjustment however made a final attempt to match the motions. In this case however, a Rayleigh damping alpha value of 2, and a stiffness value of 0.15 was included. Figure 22 illustrates the comparison.



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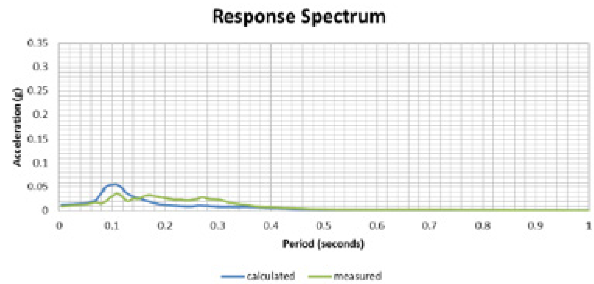

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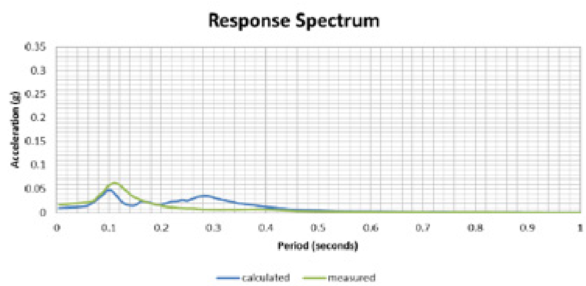
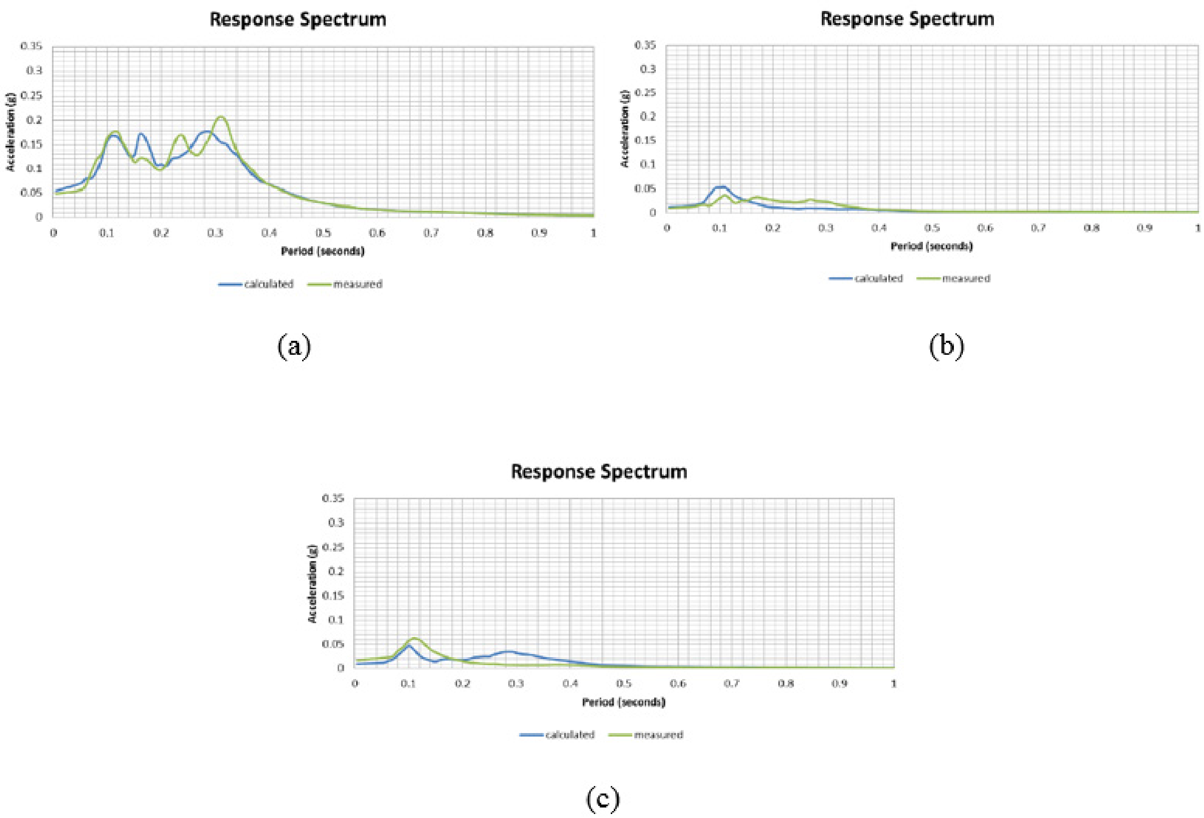


(c)

Figure 19. Measured Response Spectra at Dam Crest at Monolith 11 with Rayleigh Damping, alpha = 2.0, Stiffness Damping, beta = 0.15: (a) EW, (b) NS, (c) Vert.

From the results, focused on altering the damping input into the FEM, one can observe that while portions of the measured motions are similar, there are segments that are not matched. In an attempt to refine the FEM to match the measured motions, considerations was given to the potential for overestimated energy to be trapped in the model. In response, the material of the concrete dam was changed from a linear to non-linear model. The nonlinearities allow the dam to crack based on an estimated tensile level, representing the occurrence of cracking. Figure 23 illustrates how the FEM responded in the measured motion**.**



(a) (b)

(c)

Figure 20. Measured Response Spectra at Dam Crest at Monolith 11 with Rayleigh Damping, alpha = 1.5 and 2.5: (a) EW, (b) NS, (c) Vert.

Summary and Conclusions

According to the findings of the dynamic analysis on Monticello Dam and attempt to match the field instrumentation response during the 2015 earthquake, "the response at the dam crest was over predicted. In general, there was too much energy in the response spectrum and the displacements were overestimated. The damping used is low with respect to dams in general. Typically, 5 percent is assumed as appropriate in analysis. When the damping was increased, the energy in the response spectrum was much more reasonable with peak values about the same as measured. In general however, peaks did not occur at the same frequencies as measured. This would indicate that the model absorbs energy at different frequencies as compared with the actual structure. Various parametric studies were performed, including changing the foundation stiffness, the dam stiffness (allowing the dam to crack) and applying a variety of different Rayleigh damping values to see if a closer match can be obtained."

2016 USSD Conference Workshop

In 2016, a blind prediction workshop for seismic event at a concrete dam was held at the 2016 USSD Conference and Annual Exhibition. The workshop brought together predictors from multiple organizations and countries. They used various numerical prediction codes and were allowed to make assumptions considered reasonable to reach the best possible prediction of the motion at the crest of Monticello Dam (MONC). Analysts were given a packet of information which contained layout drawings of the arch dam, material testing results for both the foundation and the concrete am, the time history taken at the point MONF for all three directions of movement (upstream/downstream, cross-canyon, and vertical), shaker test data that was performed on Monticello Dam, temperature data, geophysics, and instrumentation/structural behavior histories.

The analysts were responsible for building a model that they judged to be sufficient to predict the response of the dam from the provided information. All analysts submitted their prediction response at point MONC, along with other data they felt would properly represent their prediction prior to the conference for compilation.

The analysis predictions were presented and compared to the actual motion using response spectra, displacements, velocities and accelerations at point MONC. The general assumptions made for each model were presented anonymously. The predictions were used to lead discussion on the assumptions made, how they may have impacted the predictions, confidence level in analyses and impacts on potential decision making, and where the group could identify areas that further improvements in analysis methods could be made.

Based on the predictions submitted for upstream to downstream and cross canyon displacements, there was noticeable variation compared to the actual motions recorded at MONC at the center, top of the dam.

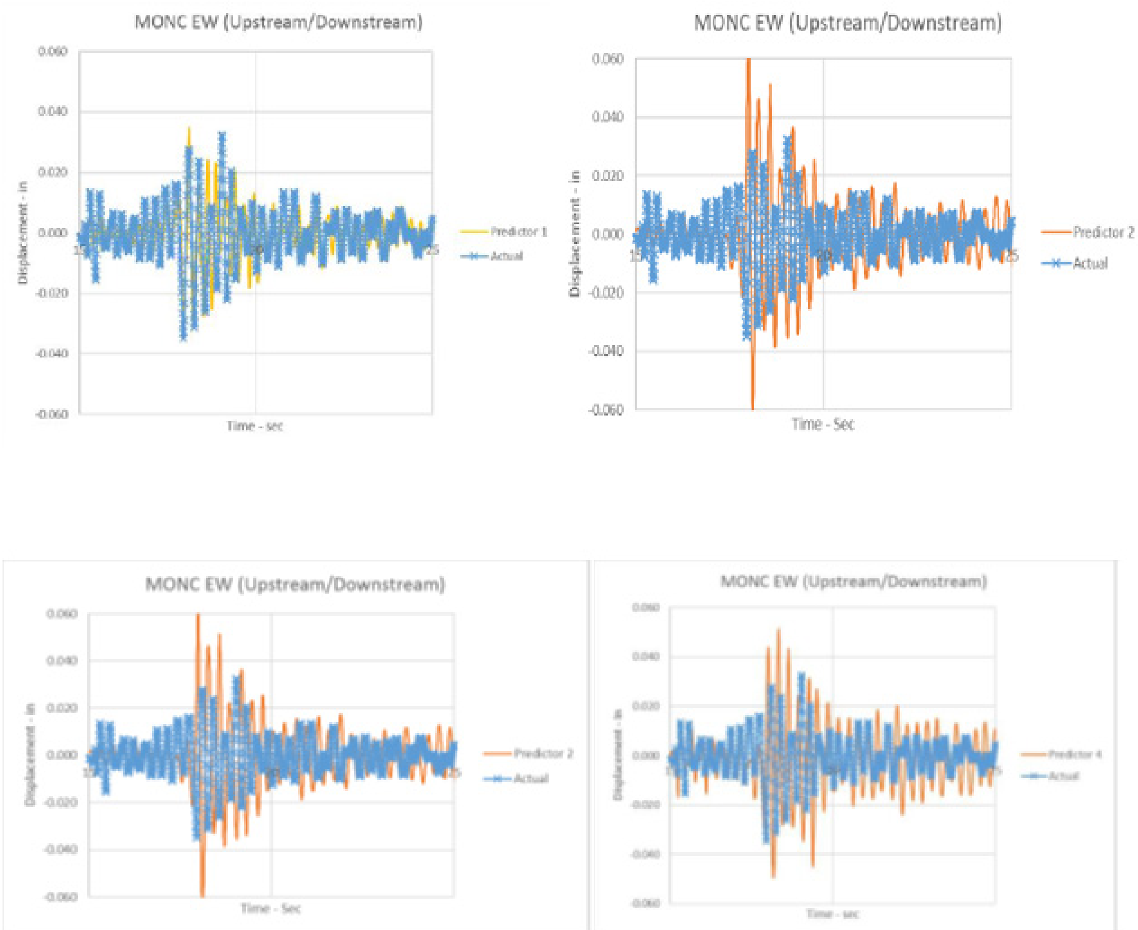


Figure 21. Upstream/Downstream for Participants Compared to Actual Ground Motion at MONC

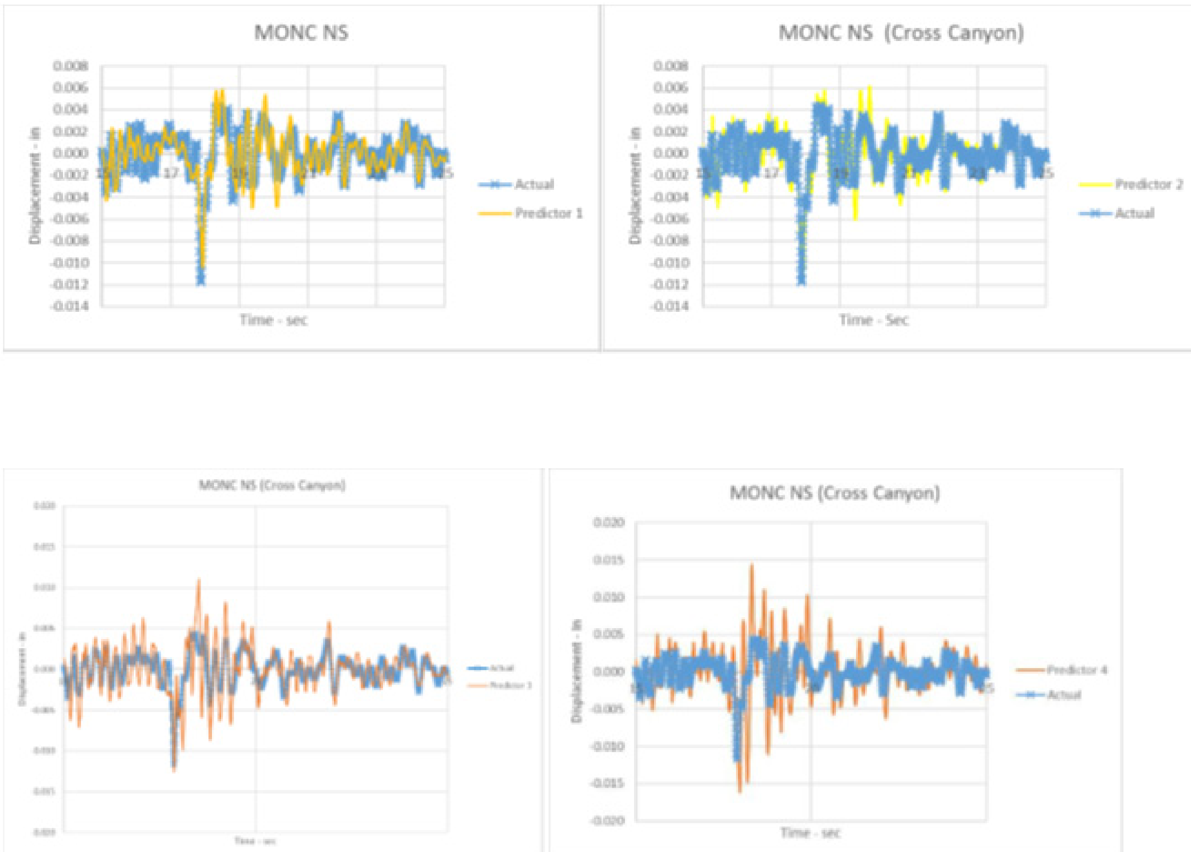


Figure 22. Cross-Canyon Displacements for Participants Compared to Actual Ground Motion at MONC

Differences in cross-canyon displacements (Predictor 3) were attributed to the use of added mass to model the reservoir loads on the dam. The simple method for modeling water loads can pull the arch dam in the cross-canyon direction, rather than sliding along the face of the dam, leading to an increase in displacement values.

Comparison of the response spectra resulted in similar variation in results. Figure 23 and Figure 24 below illustrate the variation in upstream/downstream and cross-canyon acceleration responses.

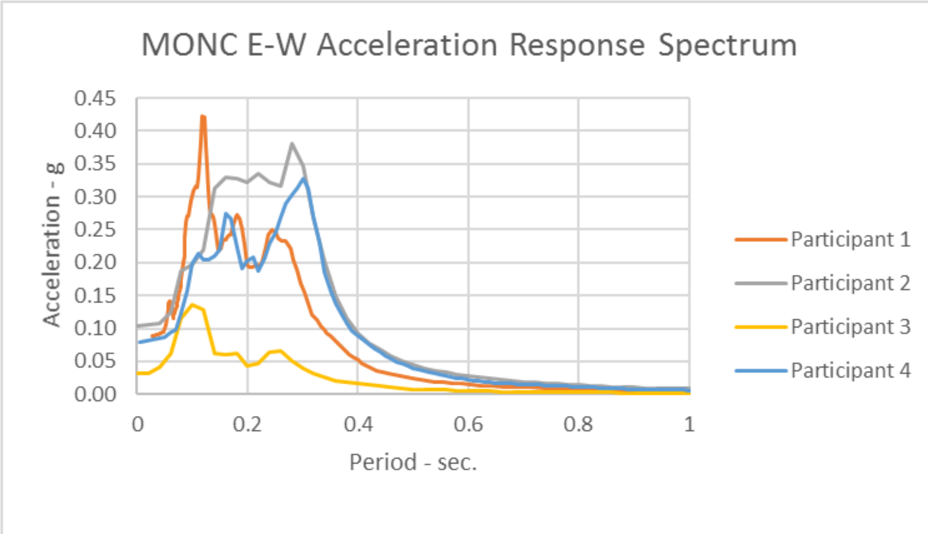


Figure 23. Upstream/downstream Response Spectra for all Participants

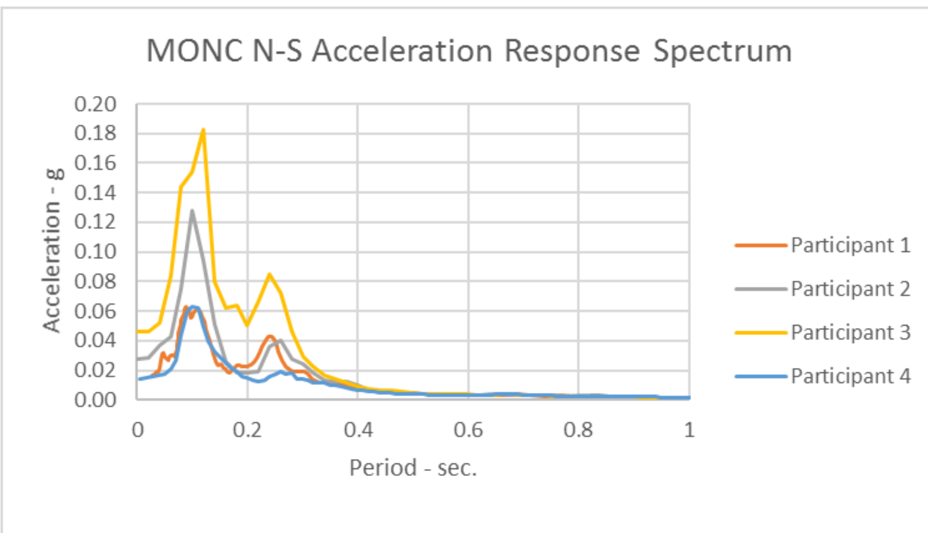


Figure 24. Cross-Canyon Response Spectra for all Participants

Overall, the blind prediction provided the opportunity for analysts to take all available information and use their own judgement to determine their preferred path forward and submit the best prediction possible at the crest of Monticello dam. Some assumptions let to a prediction of displacement more accurately, and different assumptions brought the analysis closer for the response spectrum. Uncertainty shown in the results from the blind study workshop posed the industry concern regarding whether decision makers for the dam industry can use these types of analyses will provide confidence in committing significant financial resources to the modification of these structures to ensure the safety.

Additional Studies

In response to the results of the 2016 USSD Conference blind study workshop, Reclamation initiated additional studies to further investigate why the attempt to match the response of Monticello Dam based on field seismic data.

Initial testing focused on ensuring the boundary condition on the LS-DYNA model used by Reclamation in the Monticello Dam study. In order to do so, the Monticello foundation was reduced to a flat box. With a provided wave length formulated from our Seismologists, a pulse was applied at depth in the foundation, with the purpose of measuring the pulse at the surface of flat box. Similarly, the pulse should be measurable at the boundaries at the flat box, if the boundary conditions were working properly.

The flat box was reduced to a 2-dimensional wide model, with a depth ratio of 25. Figure 25 illustrates the pulse load response between selected nodes within the flat box model, and the observation that the pulse load experiences minor reflections towards the end.

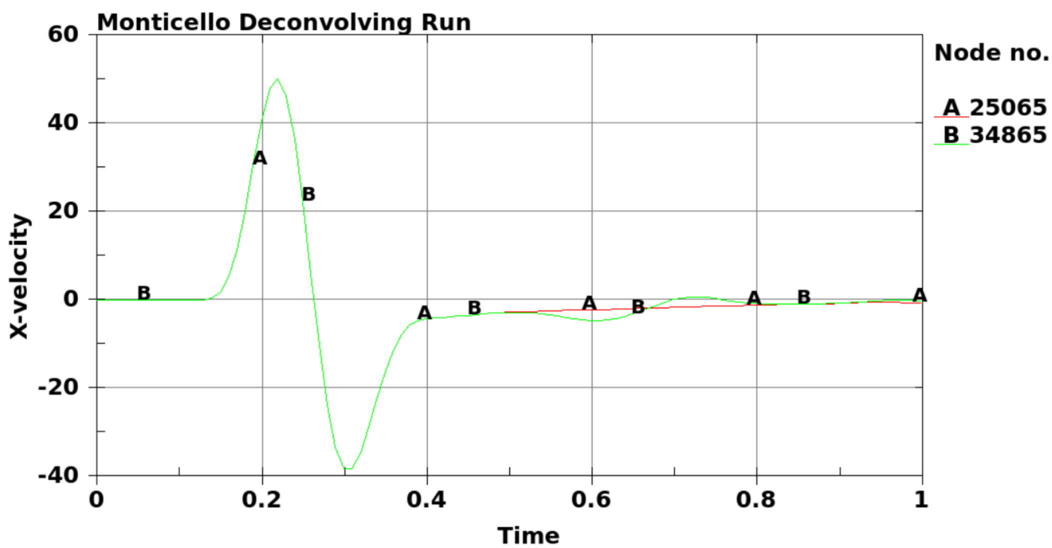


Figure 25. Flat Box Pulse Load Response – Depth Ratio of 25 – 2D Model

Dimensions of the flat box were them reduced to a ratio of 5, which is similar to the practice used by Reclamation through production runs. Using both non-reflective boundaries and perfectly match layers, commonly known as PML, a similar response is observed, as shown in Figure 26.

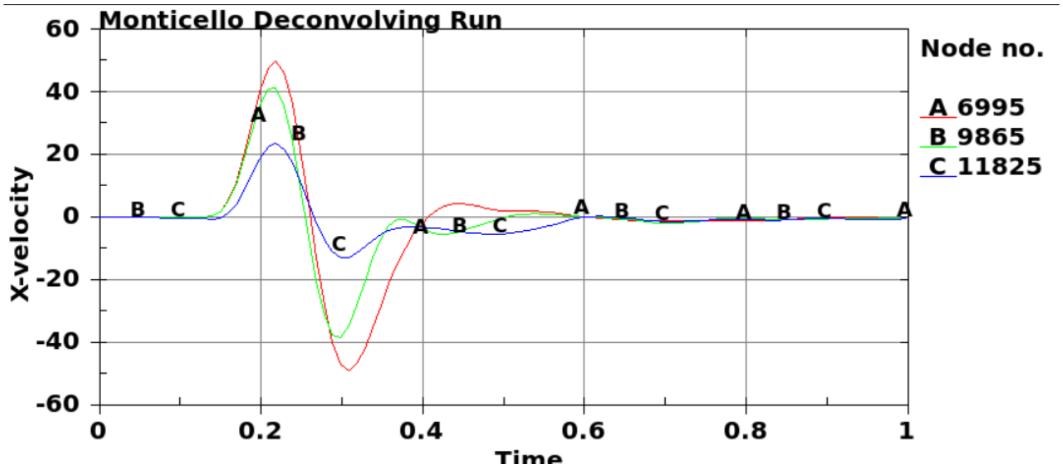


Figure 26. Flat box Pulse Load Response – Depth Ratio of 5 – 2D Model

Measurements taken at the center, quarter point, and near the edge of the model show that reflection still occurs near the edge, observed in the absence of a flat line towards the end of the pulse. When this effect is considered for a full 3-dimensional model of large size, results would experience contamination on some level due to the reflections. In order to test this theory, the flat box model was extended to a 3-dimensional size, while maintaing the width-to-depth ratio of 5. Figure 27 below illustrates that the effect from reflection becomes even greater.

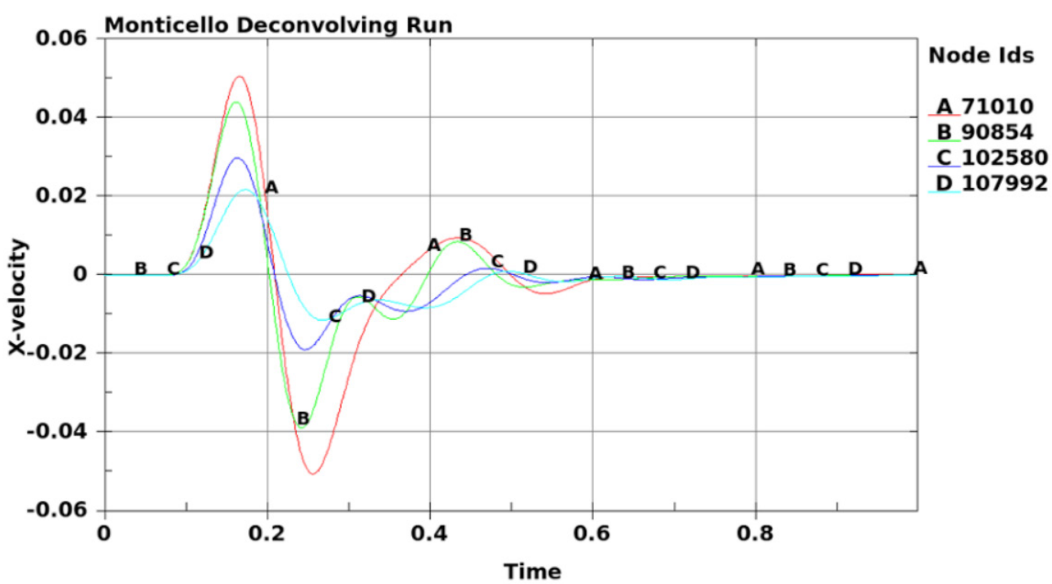


Figure 27. Flat Box Pulse Load Response – Depth Ratio of 5 – 3D Model

Conclusions
Based on the additional studies to investigate the cause of studies that have been performed to investigate the effects of reducing the size of the finite element models, the following conclusions can been used:

1) Although existing model boundaries reduce reflections, there are limitations in the model boundary formulations in LSDYNA to eliminate all the boundary reflections in our models.

2) Models where the width-to-depth ratio is about 5 (current Reclamation practice), there is significant reflection from the boundaries, contaminating the results. Models with larger width-to-depth ratios further reduces reflection however larger models are not efficient to run within a reasonable time frame, or cost.

3) Field measured motions used in the Monticello Dam analysis and study, may have included components, which cannot be modeled correctly using current numerical modeling software, such as surface motions.

Additional Studies

In 2016, Bruce Mueller, former Director of Reclamation's Safety, Security and Law Enforcement office, made a recommendation to ensure that the results produced from finite element models were validated for accuracy, prior to making high dollar decisions. The recommendation was key in supporting Reclamation's leadership among the numerical modeling community, source of guidance for other agencies, and in response to the varied results which resulted from the 2016 USSD conference Monticello Blind Prediction Analysis Workshop.

In response, a workshop was held during the 2017 USSD Conference and Annual Exhibition, the Concrete Dams Committee and Earthquakes Committee held a workshop titled, "Seismic Analysis of Concrete Dams". The purpose of the workshop was to return to the fundamental level of numerical modeling and review the basic parameters and methods that should be considered by an analyst performing a numerical structural analysis. The response to the workshop was successful, with over 100 participants attending from multiple agencies and dam owners. From the discussions, areas of agreement were focused in the importance of establishing a strong understanding of a numerical model response at the linear level, before expanding to nonlinear methods. In addition, participants relayed a concerns for the accuracy of the numerical models provided, which are often complex and multi-faceted.

Committee and Earthquakes Committee will hold another workshop during the 2018 USSD Conference and Annual Exhibition titled, "Evaluation of Numerical Models and Input Parameters in the Analysis of Concrete Dams". In an effort to identify the fundamental cause of variations of analysis results. The objective of the workshop include the tallest monolith analysis of a gravity dam, considering uncertainty in material properties and loads, identifying the advantages and disadvantages of analysis methods, discussion of accuracy of the numerical analyses solutions, identification of uncertainty and variability, understanding the importance in obtaining confidence in linear analysis, establishing list of parameters that warrant additional investigation, and expand on potential for alternate benchmark workshops for the numerical analysis community. The analysis will be limited to a linear investigation, eliminating potential error using complex nonlinear analysis materials and methods, in an attempt to identify the root cause of analysis discrepancies.

Based on findings and conclusions from this 2018 workshop, additional research avenues can be explored regard model accuracy and further investigation into the benefits and drawbacks of finite element model reduction.

Conclusion and Summary

The original scope of this research focused on the potential for reducing the size of FEMs used in current Reclamation practice, and establish a means for cost reduction. The analysis performed on Boca Dam was used as an appropriate application for exploring the effects of reduction. As discussed in this report, reduction of the FEM could overestimate results, due to trapped energy however, use of reduced model size could be used if limited to sensitivity studies to identify the potential effects in changing general parameters. This was specifically noted in the change from linear to nonlinear material models.

In parallel with the Boca Dam research, findings and discussions from the 2016 USSD Monticello Blind Prediction Study introduced an additional area of focus identifying the potential variance in methodology in numerical modeling within the engineering community. Questions arose regarding the level of confidence in standard FEMs based on the inability to match site measured motions in the benchmark study.

While cost should be carefully considered in the development and estimation for the complex numerical models used to evaluate Reclamation structures, careful consideration should also be given to how reducing the model size effects the results, and whether fundamental agreement can be established for each size. In conjunction, careful attention should be given to the loads used in the analysis, establishing potential checkpoints to ensure that the model filters the input loads as expected.

During the 2017 USSD Conference Workshop on Seismic Evaluation on Concrete Dams, agreement was reached on the importance of understanding numerical models at a linear level. Acknowledging that once nonlinearities are introduced, too many factors exist to verify and validate model results efficiently, and with repeatability.

Acknowledging the advantages in using nonlinear methods, a strong linear foundation should be established to provide confidence in initial results with results anchored on physical data and field testing. In this effort, an analyst must ensure an understanding of the model outputs and behavior based on the inputs (loading, material values, etc). As the complexity of FEMs increase based on more advanced project scopes, which may include the interaction of multiple features and increasing dynamic motions, it is imperative that one understand the FEM mechanism on an equivalent level.

Since the 2016 and 2017 USSD Workshops, Reclamation has increased efforts to established convergence in current analysis practice, which has proven to increase confidence from both the analyst, risk analysis team utilizing the FEM to make risk estimates, and Dam Safety Office. Efforts to match the measured motions at Monticello Dam have continued to be unsuccessful, but have introduced additional efforts to explore whether the FEMs are incorporating the motions used in a dynamic analysis correctly. While it has been suggested that the measured motions from Monticello Dam may have included additional surface waves or other complex seismic phases that violate basic assumption of the model, and cannot be filtered using current numerical modeling software, additional preliminary research has been performed to explore both this and response from Monticello Dam model using to simple pulse loadings. Additional effort for this research is currently on hold, pending the results of the 2018 USSD Workshop. Preliminary results from the research however, introduce the potential need to investigate the use of reflective boundaries differently to reduce energy reflection, and reduced overestimation of results. This is of significant importance given the weight placed on numerical model results during a risk analysis, and potential recommendations resulting in modification efforts.

Overall, support for reducing numerical models as a revised practice in evaluating dams to reduce cost, is not considered reasonable at this time. While it could certainly be explored for more simplified models or general sensitivity studies, additional uncertainty may be introduction for the numerical models used to evaluate dams, which are typically very complex.

Recommendations

Recommendations made from this research include support of research on the following topics, which may interconnect:

1) Explore use of alternate software to validate numerical models.

2) Establish an understanding for how seismic loads are applied to numerical models, output response from basic model, and potential limitations of boundary conditions (reducing reflected energy).

3) Explore reduction of numerical models using simplified models with basic features.

4) Consider findings and discussions for research needs from 2018 USSD Workshop.

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