13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 2493

IMPLICIT AND EXPLICIT NONLINEAR DYNAMIC ANALYSIS OF A LARGE THIN-ARCH DAM USING MASSIVELY PARALLEL COMPUTING

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SUMMARY

This research and development project was sponsored by the United States Bureau of Reclamation (USBR), who are best known for the dams, power plants, and canals it constructed in the 17 western states. The mission statement of the USBR's Dam Safety Office, located in Denver, Colorado, is "to ensure Reclamation dams do not present unacceptable risk to people, property, and the environment." The Dam Safety Office does this by quickly identifying the dams which pose an increased threat to the public, and quickly completing the related analyses in order to make decisions that will safeguard the public and associated resources. The research study described in this report constitutes one element of USBR's research and development work to advance their computational and analysis capabilities for studying the response of dams to strong earthquake motions. This project focused on the seismic response of Morrow Point Dam, which is located 263 km southwest of Denver, Colorado.

Studying the response of concrete dams to earthquake ground motions requires three general steps

- selecting earthquake ground motions at the site
- modeling the concrete dam, flexible foundation, and reservoir, and performing a transient analysis
- analyzing or evaluating the dynamic response, including the assessment of post-earthquake stability. Stability plays a large role in the analysis of these structures, especially Morrow Point Dam. Morrow Point Dam is a highly segmented structure that has a potential large "rock" or wedge defined by three foliation planes that sits directly under the dam's left abutment, which might effect the structural response of the dam.

The sequence of segmented lifts typical of Morrow Point Dam's construction has a significant impact on the static stress fields induced in the dam. An analysis of a monolithic dam may show artificially high tensile stresses within the dam, especially near the abutments. The contraction joints in arch dams cannot develop tensile stresses and may open and close throughout the duration of the earthquake. Furthermore, the opening of contraction joints may reduce the tensile stresses, but may increase the compressive arch stresses in regions where the joints are closed [Ref 1].

One objective of this study was to perform a detailed evaluation of the effect of various modeling idealizations, and assumptions on the predicted response of Morrow Point Dam. This included consideration of nonlinearities in the dam due to the shear keys across the vertical contraction joints, nonlinearities due to

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the extensive use of contact surfaces between dam/reservoir/foundation/abutment wedge, and material nonlinearities due to concrete cracking. In addition, the accurate geology topography was modeled for the foundation to study the effects of wave scattering in the canyon and a model for hydrodynamic interaction was implemented into LLNL's finite element codes for fluid representation in the three-dimensional dam system finite element model.

A large number of simulations and parameter studies were performed in this study in order to provide understanding of the significance of modeling assumptions and the differences between models of different sophistications, ranging from simple monolithic dam models to complex fully nonlinear models. The major conclusions reached during these extensive studies include:

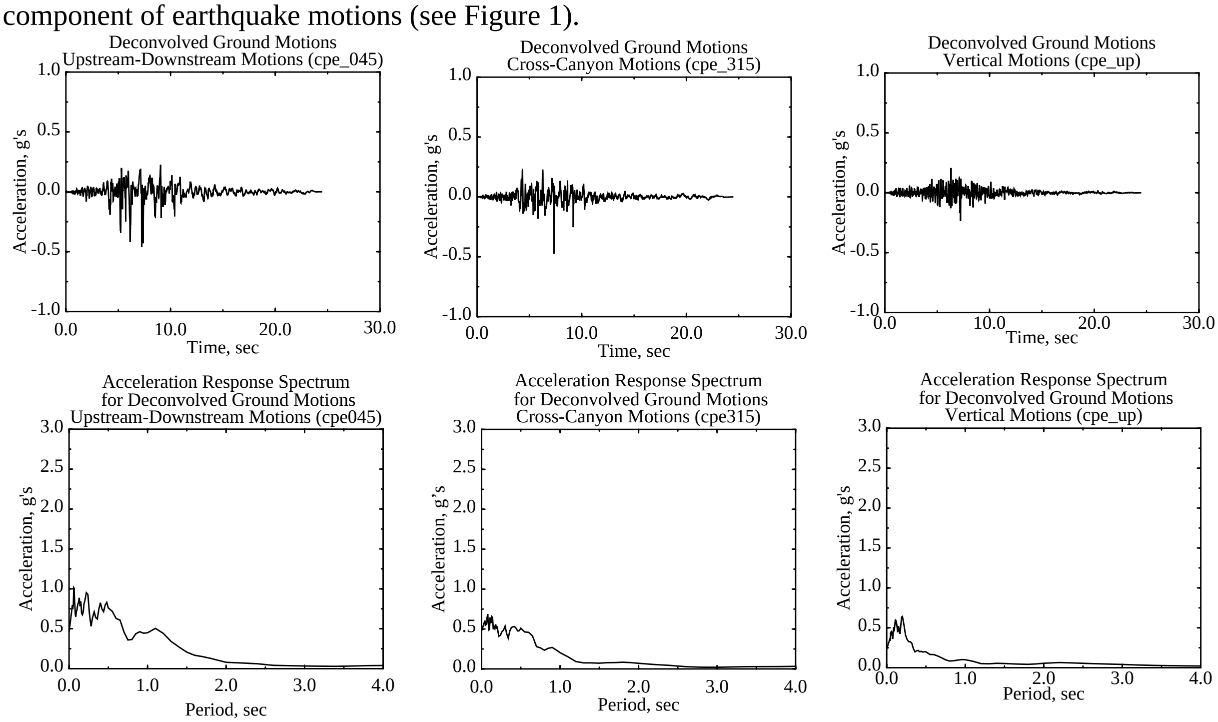
- Using discrete elements to model the joint behavior resulted in gap openings of 0.29 inches at the dam quarter point, whereas when using a sophisticated slide surface that took into account the shear key geometry, the gap openings were only as much as 0.05 inches at the dam quarter point. However, a peak gap opening of 0.375 inches occurred near the left abutment. When the left abutment wedge was allowed to move freely from both the dam and foundation, the peak gap opening near the left abutment increased to 0.65 inches and when uplift pressures and a tied with failure slide surface was used at the dam/foundation interface, a peak gap opening near the left abutment was 0.91 inches.
- The response of the dam was highly dependent on how the input ground motions were applied to the 3-D topographically accurate flexible foundation model. It was discovered that when using non-deconvolved ground motions (motions that were not deconvolved down to the base of the foundation), the accelerations or site response at the dam/foundation interface was too high. Furthermore, when the ground motions were applied at the base of the foundation using base accelerations, which meant there was a non-transmitting boundary at that surface, the dam response was much larger than if a transmitting boundary was used.
- The accurate geology topography had the effect of reducing the site response slightly at the base of the dam.
- Two separate models studied the effects of concrete material nonlinearity and a tied with failure contact surface between the dam and foundation. Both models had similar peak upstream-downstream displacements at the top center of the dam with the model that did not include these nonlinearities. The models predicted that the dam would remain stable throughout the duration of the earthquake.
- By accounting for either the hydrostatic uplift (along the foundation) or a low temperature condition on the dam, the dam response (e.g. peak upstream-downstream displacement) was similar to the response of the models that did not include these sophistications.
- The peak upstream-downstream dam displacement predicted by this study is 2.86 inches, which is very close to the 2.9 inch value predicted by the USBR's finite element study using the code EACD3D96 [Ref 2]. In addition, the maximum cross canyon displacement of the left abutment wedge was calculated to be only 0.83 inches.

INTRODUCTION

Morrow Point is a thin-arch, double-curvature dam located approximately 35 km (22 miles) east of Montrose on the Gunnison River in southwestern Colorado. The dam, which was constructed between 1965 and 1967, impounds approximately 144 million cubic meters (117,000 acre-ft) of water in the Morrow Point Reservoir. The reservoir extends approximately 19 km (12 miles) upstream. The dam structure is 143 m (468 ft) high with a crest length of 221 m (724 ft). The thin arch structure ranges in thickness from 3.7 m (12 ft) at the crest to 16 m (52 ft) at the base. The dam structure consists of a number of vertical blocks which are in contact across the vertically extending contraction joints in the dam. The vertical contraction joints of the dam are keyed to enhance shear transfer normal to the face of the dam. A typical contraction joint detail is shown in Figure 3. Under service load conditions of gravity and hydrostatic loading, the contraction joints are under a state of high compression.

EARTHQAUKE GROUND MOTIONS

Morrow Point Dam is located in the Black Canyon of the Gunnison River, a 2,460 foot deep gorge carved into folded precambrian metamorphic and igneous rock. The foundation is entirely quartzite and mica schist that are cut by granite pegmatite dikes, shears, and joints. The Cimarron fault is approximately 1 km from the dam. The Cimarron fault is a west-northwest-striking fault between Montrose and Blue Mesa Reservoir. The western end of the fault is parallel to State Highway 50 and the Gunnison River. The fault begins in the Black Canyon of the Gunnison National Monument, continues southeast past Powderhorn and Iron Hill, and terminates south of the southeastern end of Huntsman Mesa. The United States Bureau of Reclamation performed site specific seismotectonic studies for Morrow Point Dam and three ground motions were developed using the results of an initial hazard calculation. The USBR developed ground motions representing an earthquake with a return period of 1 in 50,000 years and representing a magnitude M 6.5 to M 6.7 earthquake on the Cimarron Fault. The earthquake motions that are used throughout this study are the empirical records developed from the Cerro Prieto (cpe) recording from the M 6.5 Victoria, Mexico earthquake. The cpe_045 component is the component applied in the upstream-downstream direction, the cpe_315 is the component applied in the cross-canyon direction, and the cpe_up is the vertical component of earthquake motions (see Figure 1).



a.) **b.)** **c.)**

FIGURE 1. Morrow Point deconvolved ground motions and acceleration response spectrum for Cerro Prieto time history. a) upstream-downstream motions; b) cross canyon motions; c) vertical motions.

Calculation of Force Time Histories

In LLNL's explicit code DYNA3D [Ref 3], there are two methods for applying earthquake ground motions. One can either apply the ground motions as accelerations (and velocities) or as nodal forces. The easiest method is to apply the ground motions as base accelerations. However, if one wants a non-reflecting boundary condition at the same location as the earthquake ground motions are to be applied, base accelerations may not be used. When using the non-reflecting boundary conditions in DYNA3D, one must modify the force time histories to account for the dampers at the input location. In the finite element models presented here, the force time histories were input at the base of the foundation geometry. This is also the location of a non-reflecting boundary. Any forces placed at the same location as a non-reflecting boundary, will be lowered due to the dashpots or dampers at that same node. To verify that the dam would be loaded with the correct ground accelerations, the three components of force time histories were applied to the base of a 520 meter deep rectangular box of foundation material. Acceleration time histories were gathered at the top of the foundation box for comparison with the USBR non-deconvolved ground accelerations. In addition, the force time histories were also applied to the base of the topographically correct foundation model to see what effect the topography would have on the ground accelerations at the Morrow Point Dam location. Figure 2 compares the USBR ground motions with the DYNA3D ground motions gathered from the simplified foundation model and the topographically correct foundation model. The topography tends to lessen the response across the entire spectrum at the base of the dam. This tends to be especially true for the horizontal components.

CONTRACTION JOINT DETAIL

Two methods were used to model the contraction joint behavior of Morrow Point Dam. During the first phase of this study, the modeling of the contraction joints included utilizing various combinations of contact surface models and discrete elements (i.e. essentially a two force member which applies a user specified force - displacement relationship between two specified nodes) to model the contact and connectivity across the expansion joint. A requirement of the contraction joint model was that the contraction joints allow free relative motion in a vertical direction between adjacent dam segments as the gravity dead load was applied. This relative motion prevents the generation of large vertical direction shear stresses which transfer large loads to the upper abutment region of the dam - which the actual construction process prevents. For the dead load initialization, a model was constructed for the NIKE3D [Ref 4] implicit finite element program. The contraction joints were modeled with frictionless contact surfaces for the NIKE3D initialization. This prevents friction between adjacent blocks as the dead loads are applied and does not allow inter-block vertical shears to develop. To obtain displacement compatibility in the direction normal to the dam, discrete elements were placed across each interface to transfer stresses between blocks in the normal direction. The discrete elements only allowed compression, so that tensile forces were not generated across the contraction joints if they were to open. Ten discrete springs - five for each side - were modeled approximately every 30 vertical feet along each vertical contraction joint. In addition, during the NIKE3D static initialization, a “sliding with voids” interface without friction was used along the contraction joints. The springs were modeled such that they could not resist loads in tension, but they could resist loads in compression. In compression, they were given an extremely high stiffness of 1.0E+09 lbs/in. in order to resist the compression forces when the dam blocks or contraction joints were placed in shear. The sliding interface was used so that penetration of the contraction joints would not occur. During the seismic analysis, a friction value of 0.3 was used between the vertical joints.

Although the discrete element representation for the contraction joints worked very well, they still could not represent the true behavior of the contraction joints. Due to the geometry of the shear keys, if a slight opening formed along a contraction joint, the vertical blocks could move in the upstream-downstream direction only a little before contacting the shear keys again. In reality, a 6 inch opening would have to occur before the joints could freely move in the upstream-downstream direction. The discrete elements could not model this behavior. If an opening occurred along a joint, the blocks were virtually free to move in the upstream-downstream direction. Therefore, one of the objectives of Phase 1 was to modify the existing NIKE3D and DYNA3D contact surfaces to allow appropriate modeling of the available shear transfer across the contraction joints. A finite element formulation has been proposed and implemented by Lau, et. al. [Ref 5] for the case of rectangular cross-section keys. For keys of rectangular cross-section, the kinematics are much simplified because lateral motion is prohibited even when the joint is partially open. In the case of beveled cross-sections, the joint may move a restricted amount laterally as a function of the joint separation.

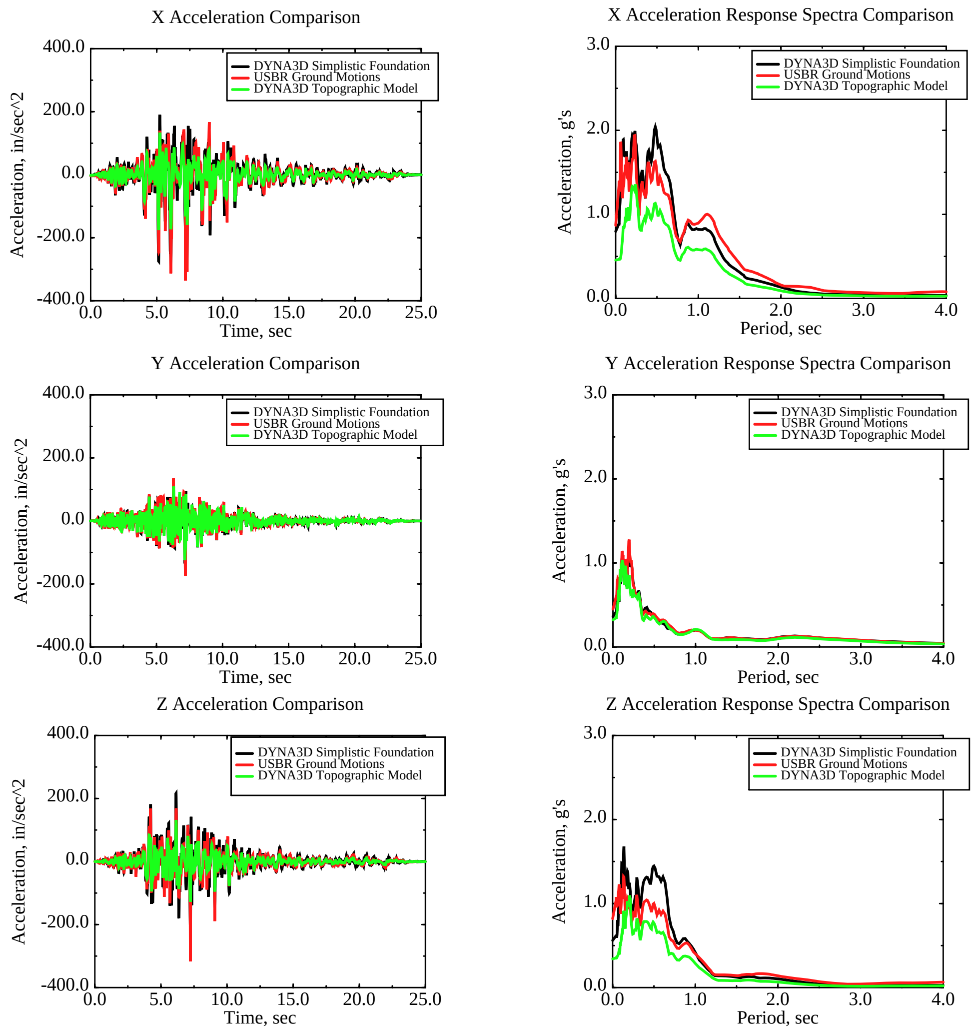


FIGURE 2. Comparison between USBR input ground motions, DYNA3D ground motions at base of dam using simplified foundation, and DYNA3D ground motions using topographically correct foundation. The DYNA3D simulation includes 3.4% damping for first mode of Morrow Point Dam.

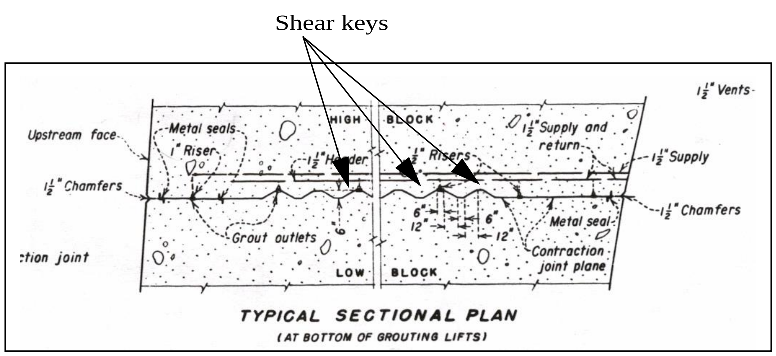
LLNL developed a formulation of a contraction joint model for the analysis of beveled contraction joints, based upon the general mechanics and resulting finite element implementation used in NIKE3D [Ref 6]. To verify that the contraction joint slide surface was indeed working as intended, two dam-size blocks, of similar size to that of Morrow Point Dam, were pushed past each other. As the one block is being pushed past the other, the shear keys cause the block to be pushed outward. Because of the transverse pressure placed on the block being pushed, it is allowed to slide back into another shear key and contact the adjacent block. The time history of the transverse displacement plotted against the upstream-downstream displacement as well as a pictorial description of the Morrow Point shear keys is shown in Figure 3. By comparing the plan view of the shear keys and the displacement time history, the sliding interface does a good job of representing the behavior of the shear keys without having to explicitly model them using finite elements.

MORROW POINT DAM FINITE ELEMENT MODELS

A wide variety of finite element models were used for this study (see Figure 4). The following is a brief list of some of the features used in the finite element models (Note: not all of these features were used in every finite element model):

1. Westergaard added mass for fluid-structure interaction [Ref 7]
2. Water explicitly modeled using an elastic material in NIKE3D and a fluid material in DYNA3D
3. Vertical contraction joints
4. Topographically accurate flexible foundation
5. Left abutment wedge explicitly represented
6. Ground motions input as either base accelerations or force time histories
7. Non-reflecting boundaries
8. Sliding contact between reservoir/foundation and reservoir/dam
9. Transmitting boundary on upstream side of reservoir
10. Hydrostatic uplift along wedge and foundation contacts
11. Thermal load applied to dam to represent a low-temperature condition
12. Concrete damage plasticity model used for dam
13. Tied with failure slide surface for dam/foundation contact interface

a.)



Plan view

b.)

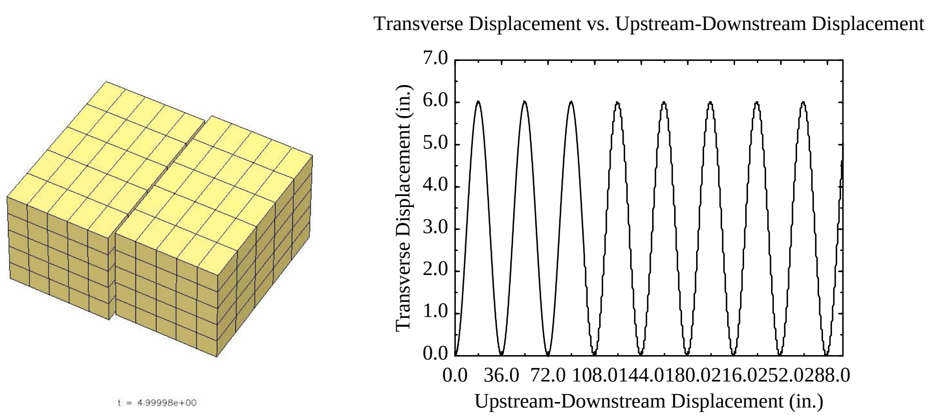


FIGURE 3. a) Plan view of Morrow Point Dam shear keys; b) transverse displacement vs. upstream-downstream displacement time history for contraction joint test problem.

Contraction Joint Finite Element Model with Rigid Foundation and Westergaard Added Mass

For this finite element model, the base of the dam is considered fixed in all three directions. This finite element model consists of 23,195 brick elements and either 1,640 discrete elements to model the contact and connectivity across the expansion joints or it can use the contraction joint contact surfaces. A requirement of the contraction joint model was that the contraction joints allow free relative motion in a vertical direction between adjacent dam segments as the gravity dead load was applied. This relative motion prevents the generation of large vertical direction shear stresses which transfer large loads to the upper abutment region of the dam - which the actual construction process of the dam prevents. The contraction joints were modeled with frictionless contact surfaces for the NIKE3D static initialization. This prevents friction between adjacent blocks as the dead loads are applied and does not allow inter-block vertical shears to develop. During the seismic analysis stage, an assumed coefficient of friction value of 0.3 is added to the contraction joint contact surfaces. To simulate the influence of the fluid on the dam structure, Westergaard added mass was included into this finite element model.

Contraction Joint Finite Element Model with Flexible Foundation and Westergaard Added Mass

This model consists of the same dam model as that described in the previous section, but instead of having a fixed base it has a flexible foundation. To achieve an accurate geology topography for the finite element model, a 1983 USGS topographic map was scanned and used to generate an IGES surface for the TrueGrid mesh generator. This model consists of approximately 101,000 brick elements and 1,640 discrete elements. To connect the dam model to the foundation model, a tied slide surface was used. This model also used Westergaard added mass to simulate the fluid-structure interaction.

Contraction Joint Finite Element Model with Flexible Foundation and Water Explicitly Modeled

For this model, the water is now explicitly modeled instead of using Westergaard added mass for the fluid-structure interaction. For the static initialization in the NIKE3D implicit finite element program, an elastic material was used to simulate the water. A low elastic modulus of 189.7 psi and a high poisson's ratio of 0.4999 were used to achieve a low shear modulus and the bulk modulus of fresh water. For the seismic analysis, which was done using the DYNA3D explicit finite element program, the fluid material (Material 9) and an equation of state, which specified the bulk modulus, were used to model the water. A pressure cutoff and viscosity coefficient of 0.0 were assumed. To connect the water to the foundation in this model, a tied slide surface was used. A sliding with voids slide surface, however, was used between the water and the dam. This was done so that the water could slide downwards next to the dam during the gravity initialization, preventing any unwanted stresses to be formed on the dam surface. During the second phase of this project, both the reservoir/foundation and reservoir/dam contact surfaces were changed to a sliding only (with no gaps) slide surface for both the static and seismic analyses.

Contraction Joint Finite Element Model with Flexible Foundation, Water Explicitly Modeled, and Left Abutment Wedge

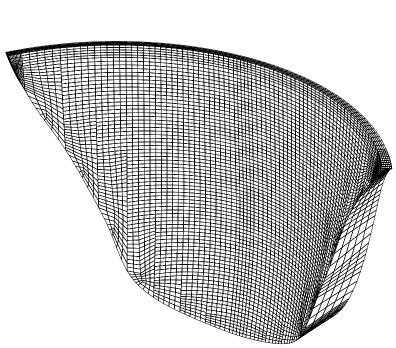
This finite element includes a new feature called an abutment wedge. This wedge, or large rock, in the foundation is defined by three foliation planes - a base plane, side plane, and release plane. To assist in modeling the abutment wedge, the USBR provided LLNL with coordinates and unit normals to be used in TrueGrid for defining the foliation planes or the contact surfaces this abutment wedge slides along. A transition region was used to connect the larger elements of the foundation with the smaller elements of the abutment wedge. A tied slide surface was used between the foundation and transition region. During the static initialization in NIKE3D, a tied slide surface was used between the wedge and transition region. For the seismic analysis in DYNA3D, this slide surface was changed to a sliding with voids surface with a high coefficient of friction.

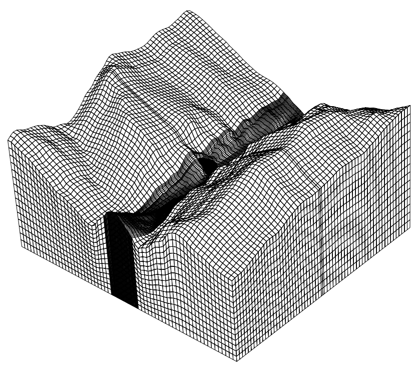
Homogeneous Finite Element Model

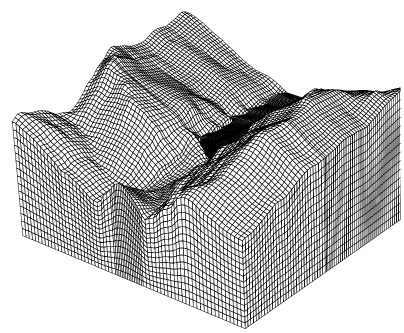
Figure 4 shows a homogeneous finite element model of Morrow Point Dam, or a model that does not include the vertical contraction joints. This model was used in conjunction with the water explicitly modeled and a flexible foundation to assist in validation and comparison with other finite element programs, such as EACD3D96.

Finite Element Analysis Procedures of Morrow Point Dam

Two finite element analysis procedures were employed for this study. The first procedure was the base acceleration method. This used a fixed bottom boundary and base accelerations to excite the earthquake motions. The second method, the force time history method, used a non-reflecting boundary condition at the bottom boundary and force time histories instead of base accelerations to excite the structural models. Force time histories were used because base accelerations and non-reflecting boundaries cannot be used in conjunction with each other.



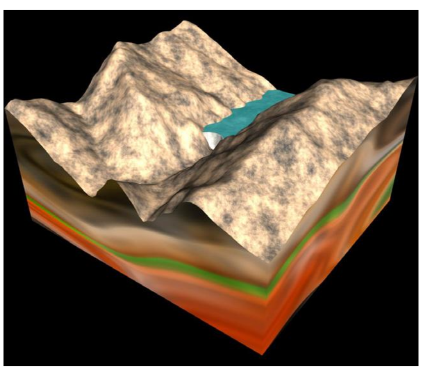


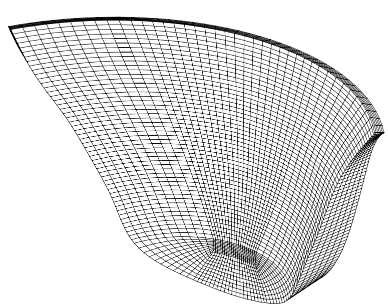


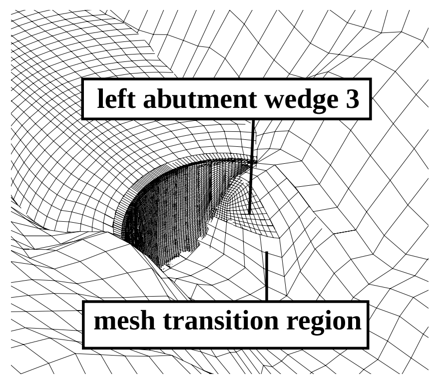
Contraction Joint Finite Element Model

Flexible Foundation with Westergaard Added Mass

Flexible Foundation with Water Explicitly Modeled







Pictorial Representation of Full 3-D Model

Homogeneous/Monolithic Dam Model

Full 3-D Model with Left Abutment Wedge

FIGURE 4. Finite element models used for seismic study.

The finite element procedure for the force time history method is graphically presented in Figure 5. First, the NIKE3D and DYNA3D finite element models are generated using the TrueGrid mesh generator. Once the models are generated, a static gravity initialization is performed using the NIKE3D implicit finite element program. The bottom and sides of the foundation have been given a zero displacement controlled boundary condition in the direction normal to the foundation. NIKE3D computes reaction forces for nodal degrees of freedom with prescribed displacement boundary conditions. The reason for using boundary conditions on the sides of the foundation is that the canyon would open up during the gravity initialization without them, resulting in very high stresses in the dam. Displacement boundary conditions were used instead of fixed boundary conditions, because the DYNA3D model uses nonreflecting boundary conditions on these same sides. Nonreflecting boundary conditions do not work with fixed boundary conditions, but will work if reaction forces have been placed at the same location as the nonreflecting boundary conditions. After the static initialization, the reaction forces from the zero displacement boundary conditions are gathered and imported into the DYNA3D finite element model. Three components of force time histories were applied to the base, instead of base accelerations, for the seismic analysis in DYNA3D. The seismic analyses are run using the DYNA3D explicit finite element program. 3.3% mass proportional damping for the fundamental mode has been assumed for all analyses presented in this study. Once the analyses are complete, the post-processor GRIZ is used to view and analyze the results.

EIGENVALUE ANALYSIS

To assist in validating the finite element models, a number of eigenanalyses were completed and the results were compared with those computed by Fenves et al [Ref 8], Tan and Chopra [Ref 9], and Duron and Hall [Ref 10]. Furthermore, Duron and Hall performed forced vibration tests on Morrow Point Dam using two Kinemetrics vibration generators (5000 lb force capacity each) placed side by side on the dam crest at the center of the dam. Symmetric responses were obtained by shaking in the upstream-downstream direction, and antisymmetric responses were obtained by shaking in the cross canyon direction. The modal results from the literature and Duron and Hall's experimental data along with LLNL's eigenvalue analyses are compared in Figure 6. To calculate the mode shapes and frequencies, gravity and hydrostatic loads were placed on the structural model using NIKE3D. Once the static initialization was complete, an eigenanalysis could be completed using NIKE3D and the stress and displacement fields calculated by the static analysis. It should be noted that the coefficient of friction for the contraction joints was 0.0 for the static initialization and 0.3 for the eigenvalue analysis. Three finite element models were studied

1. contraction joint slide surface model with empty reservoir and fixed base
2. contraction joint slide surface model with diagonal added mass and fixed base
3. homogeneous/monolithic model with diagonal added mass and fixed base

By examining the data, it can be concluded that the contraction joint slide surfaces have the effect of lowering the frequencies in comparison to the results from a model that uses discrete elements. By comparing the contraction joint models to that of the homogeneous/monolithic model, the contraction joints appear to make the structure more flexible. In addition, the homogeneous model appears to match the experimentally determined fundamental mode, whereas the second symmetric mode of the contraction joint model more closely matches better with the experimentally determined second symmetric mode. Finally, the reservoir has the effect of lowering the natural frequencies of the structure.

TRANSIENT ANALYSIS

For the transient analyses, the Cerro Prieto earthquake ground motions were used. Each analysis was statically initialized using the NIKE3D implicit finite element program for both gravity and hydrostatic loads. Once the NIKE3D analyses was complete, a NIKE3D stress binary file, which contained stresses and displacements from the static initialization, was used as input into DYNA3D. This statically initialized state would be the first state in the DYNA3D explicit analysis. For each model described, a mass proportional damping of 3.3-3.4% for the first mode was used.

As discussed previously, there were two phases to this seismic study. The phase 1 finite element models focused on studying the differences between using a fixed base approach or a flexible foundation and also using Westergaard added mass versus modeling the reservoir explicitly. In addition, the phase 1 models allowed the opportunity to study the differences in using deconvolved ground motions as well as the response of the left abutment wedge. For the finite element models that explicitly modeled the reservoir, the reservoir was considered tied to the foundation with a sliding with voids slide surface between the reservoir and dam. In addition, the reservoir was restrained at the edge of the finite element model using a translational boundary condition; hence, there was a non-transmitting boundary at the far upstream side of the reservoir. For the flexible foundation phase 1 models, the base acceleration analysis procedure was used. In other words, the bottom surface of the foundation had a reflecting boundary, which resulted in a more severe response in the dam structure. The phase 1 models also used discrete elements to model the contraction joint behavior.

All of the phase 2 models incorporated a flexible foundation and an explicit reservoir model. One of the objectives of this second phase of the study was to start with a simple model and include more features or complexity in each analysis after the first in order to analyze the effects of each model feature. Furthermore, all of the phase 2 models used the deconvolved ground motions and the force time history analysis procedure. Therefore, the reflecting boundary at the base has been effectively removed from these models. In contrast to the phase 1 models, a transmitting boundary was placed at the far upstream side of the reservoir and a sliding only contact was used between the reservoir/dam and reservoir/foundation surfaces. For the phase 2 analyses that model the contraction joints, the joints are modeled using the contraction joint interface and not the discrete springs. Figure 7 shows a comparison of upstream-downstream displacement time histories, gap opening time histories, and cross canyon wedge displacement time histories between four different finite element models that have varying degrees of sophistication.

CONCLUSIONS

A summary of the upstream-downstream displacements for all of the analyses are provided for comparison in Table 1. Based on the extensive analyses performed, a number of conclusions can be stated:

- When modeling a foundation, it is important to deconvolve the ground motions to the base of the foundation model. By not deconvolving the ground motions, the earthquake accelerations may be larger than wanted at the dam/foundation interface.
- When modeling a flexible foundation, it is also important to use non-reflecting boundaries along all sides of the finite element mesh. If a reflecting boundary was placed at the base of the foundation, for example, it was seen that the response was greater across the entire acceleration response spectra in comparison to the response that would be calculated if a non-reflecting boundary was used.
- The topography had an effect of reducing the ground motions seen by the dam structure.
- By placing a sliding with voids interface between the dam and wedge instead of a tied contact surface, the permanent wedge displacements doubled in value.
- The USBR calculated peak displacement using EACD3D96 compared very well to the models that used non-reflecting boundaries throughout the finite element model and deconvolved ground motions.
- It can be concluded from the finite element model that used a concrete damage plasticity model for the dam structure, that the damage to the concrete for this magnitude earthquake would be minimal.
- The tied with failure surface models suggest that the concrete dam structure is very stable throughout the earthquake loading.
- Hydrostatic uplift pressures at the dam/foundation interface has little effect on the peak upstream-downstream displacement. It may have an effect on the peak contraction joint gap opening near the left abutment.

- The low temperature condition analysis caused minimal differences in peak upstream-downstream displacements.
- The contraction joint openings are more severe when the wedge is not restricted or tied to the dam or foundation and when a tied with failure slide surface and uplift is modeled between the dam/foundation interface.

REFERENCES

1. Clough, R.W. "Nonlinear Mechanisms in the Seismic Response of Arch Dams." Proc., Int. Res. Conf., on Earthquake Engrg., 1980, Skopje, Yugoslavia.
2. Nuss, L.K. "Static and Dynamic Structural Linear Elastic Structural Analysis (EACD3D96) Morrow Point Dam." Technical Memorandum No. MP-D8110-IE-2002-2, Technical Service Center, U.S. Department of the Interior, Bureau of Reclamation, Denver, CO, September 2002.
3. Whirley, R.G. "DYNA3D: A Nonlinear, Explicit, Three-Dimensional Finite Element Code for Solid and Structural Mechanics." Lawrence Livermore National Laboratory Report UCRL-MA-107254-REV-1.
4. Maker, B.N., Ferencz, R.M., and Hallquist, J.O. "NIKE3D: A Nonlinear, Implicit, Three-Dimensional Finite Element Code for Solid and Structural Mechanics." Lawrence Livermore National Laboratory Report UCRL-MA-105268.
5. Lau, D.T., Noruziann, B., and Razaqpur, A.G. "Modelling of Contraction Joint and Shear Sliding Effects on Earthquake Response of Arch Dams." Earthquake Engineering and Structural Dynamics, 1988, 27:1013 - 1029.
6. Puso, M., Laursen,T., and Weiss, J. "Contact Improvements in Nike3d." Lawrence Livermore National Laboratory, 1996, UCRL-JC Report 125876.
7. Kuo J.S. "Fluid-Structure Interactions: Added Mass Computations for Incompressible Fluid." Earthquake Engineering Research Center, August 1982, Report No. UCB/EERC-82/09.
8. Fenves, G.L., Mojtahedi, S., and Reimer, R.B. "Effect of Contraction Joints on Earthquake Response of an Arch Dam." Journal of Structural Engineering, April 1992, Vol. 118, No. 4, 1039-1055.
9. Tan, H., Chopra, A.K. "Dam-Foundation Rock Interaction Effects in Earthquake Response of Arch Dams." Journal of Structural Engineering, May 1996.
10. Duron, Z.H., Hall, J.F. "Experimental and Finite Element Studies of the Forced Vibration Response of Morrow Point Dam." Earthquake Engineering and Structural Dynamics, March 1988, Vol. 16, 1021-1039.

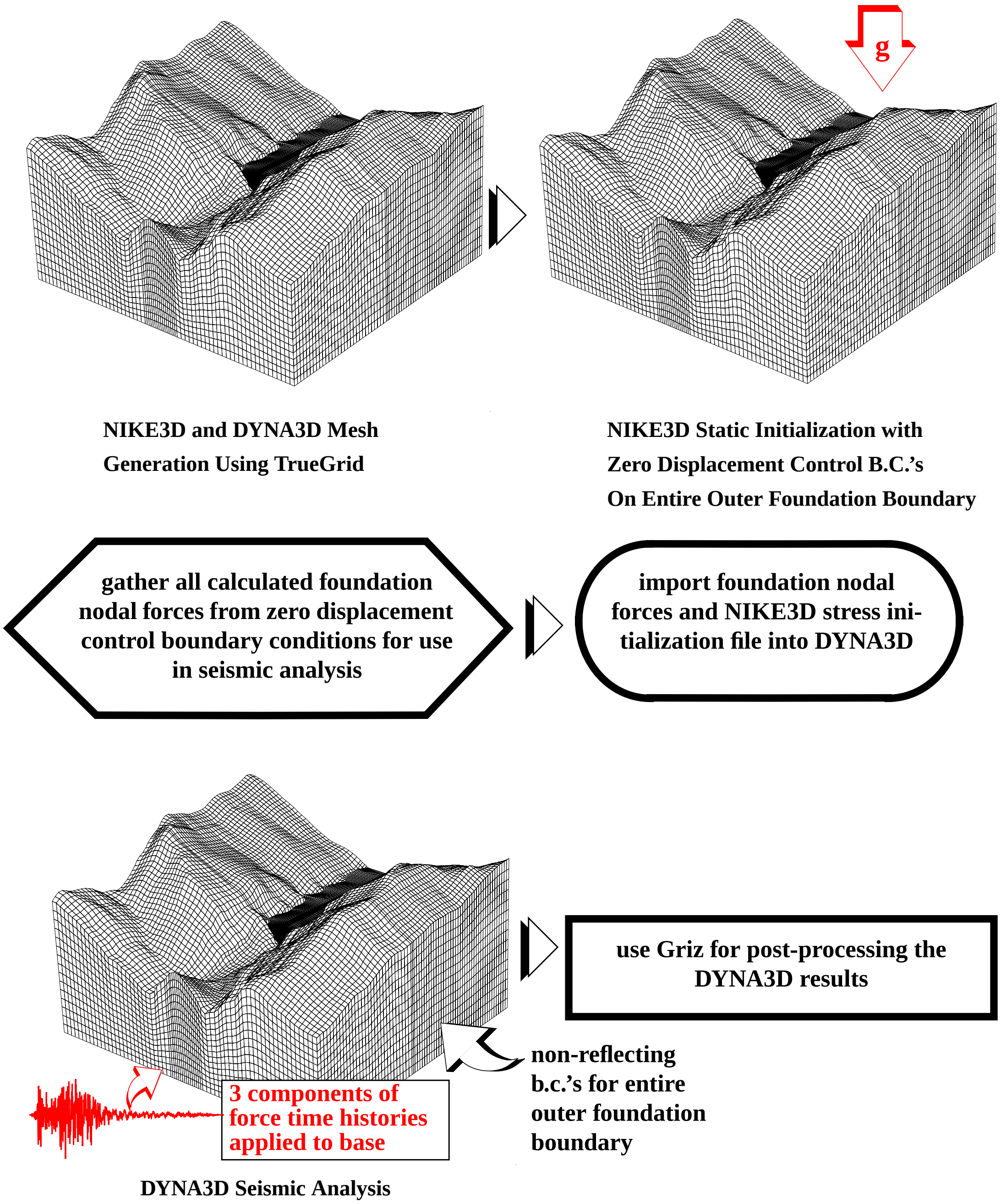


FIGURE 5. Morrow Point Dam finite element analysis procedure for force time history method.

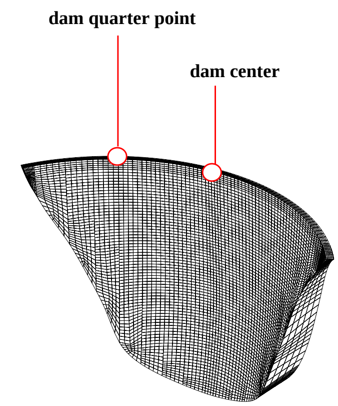
Vibration Frequency[a] (hz)										
Fenves et al			Tan and Chopra		Duron and Hall			NIKE3D†		
Empty Reservoir	Full Reservoir		Empty Reservoir	Full Reservoir	Experimental	Computed with Incompressible Water	Computed with Compressible Water	Contraction Joint Slide Surface Model with Empty Reservoir	Contraction Joint Slide Surface Model with Diagonal Added Mass	Homogeneous/Monolithic Model with Diagonal Added Mass
	Full Added Mass	Diagonal Added Mass								
3.23	2.80	2.5	4.27	2.82	S[b]: 2.95 A[c]: 3.3	S: 3.29 A:3.31	S: 3.05 A: 3.31	3.75	2.51	2.84
3.56	3.02	2.61	-	-	S: 3.95 A: 6.21	S: 5.33 A: 6.76	S: 4.21 A: 6.35	4.4	2.59	3.01
5.63	5.63	3.64	-	-	S: 5.4	S: 6.11	S: 5.96	6.4	3.9	4.4
5.96	4.82	3.98	-	-	-	-	-	6.7	4.2	4.9
6.43	5.78	4.38	-	-	-	-	-	7.4	4.8	5.76

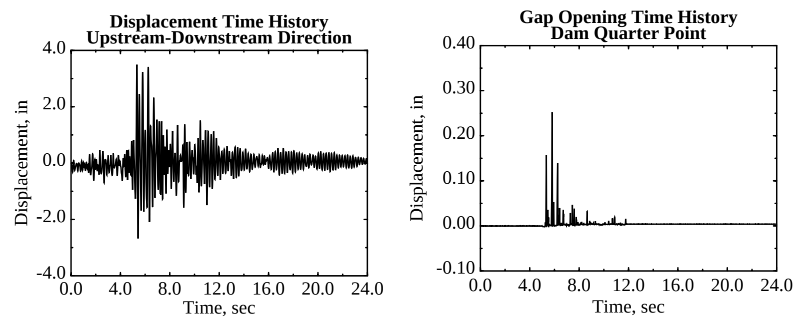
a.All computed frequencies used a rigid foundation.

b.The ‘S’ corresponds to a symmetric shake.

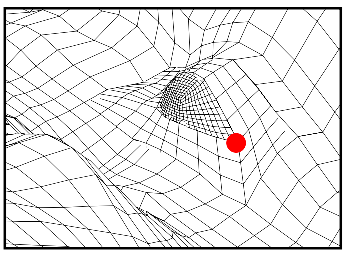
c.The ‘A’ corresponds to an antisymmetric shake.

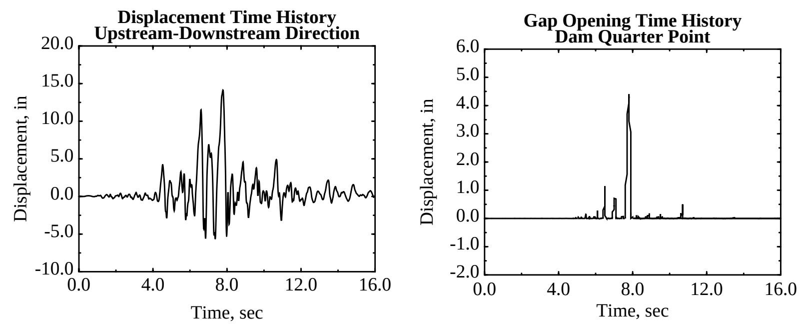
FIGURE 6. Comparison between measured and computed resonant frequencies for Morrow Point Dam.



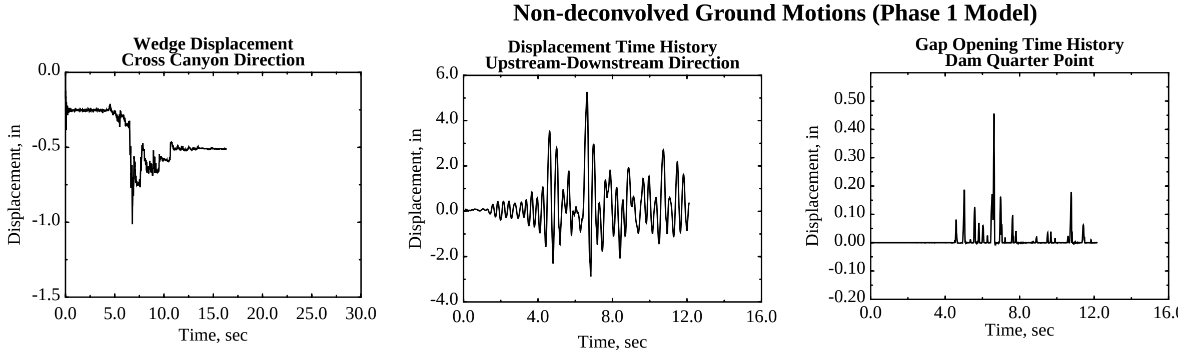


Fixed Base Model/Westergaard Added Mass (Phase 1 Model)

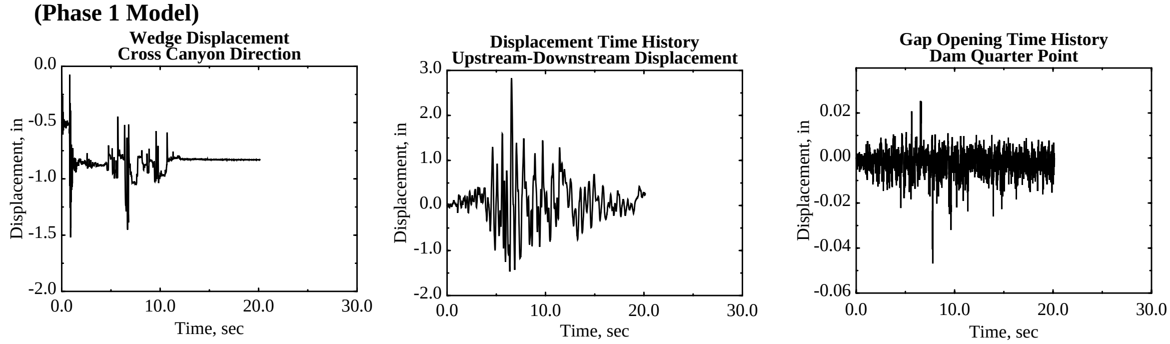




Flexible Foundation/Water Explicitly Modeled/
Reflecting Boundary at Bottom of Foundation/
Non-deconvolved Ground Motions (Phase 1 Model)



Flexible Foundation/Water Explicitly Modeled/Reflecting Boundary at Bottom of Foundation/
Deconvolved Ground Motions/Left Abutment Wedge Fixed to Dam and Free from Foundation
(Phase 1 Model)



Flexible Foundation/Water Explicitly Modeled/Nonreflecting Boundary at Bottom of Foundation/
Deconvolved Ground Motions/Left Abutment Wedge Not Fixed to Foundation or Dam
(Phase 2 Model)

FIGURE 7. Displacement time histories for center point of dam in upstream-downstream direction, gap opening time histories for dam quarter point, and cross canyon wedge displacement time histories for four different finite element models.

TABLE 1. Upstream-Downstream Displacement Comparison

Study	Finite Element Model	Maximum Upstream-Downstream Displacement (in.)
USBR	EACD3D96	2.9
LLNL (Phase 1)	Rigid Foundation, Westergaard Added Mass, Discrete Element Contraction Joint Model	3.5
	Flexible Foundation, Westergaard Added Mass, Discrete Element Contraction Joint Model, Reflective Bottom Boundary, Non-deconvolved Ground Motions	7.4
	Flexible Foundation, Water Explicitly Modeled, Discrete Element Contraction Joint Model, Reflective Bottom Boundary, Non-deconvolved Ground Motions	14.1
	Flexible Foundation, Water Explicitly Modeled, Left Abutment Wedge, Discrete Element Contraction Joint Model, Reflective Bottom Boundary, Deconvolved Ground Motions	5.27
LLNL (Phase 2) All Models Used Force Time History Analysis Procedure (Non-reflecting Bottom Boundary) and Deconvolved Ground Motions	Homogeneous/Monolithic Dam, Flexible Foundation, Water Explicitly Modeled, Wedge Tied to Dam and Foundation	2.76
	Model with Contraction Joints, Flexible Foundation, Water Explicitly Modeled, Wedge Tied to Dam and Foundation	2.6
	Model with Contraction Joints, Flexible Foundation, Water Explicitly Modeled, Wedge Not Fixed to Foundation	2.73
	Model with Contraction Joints, Flexible Foundation, Water Explicitly Modeled, Wedge Not Fixed to Foundation or Dam	2.83
	Model with Contraction Joints, Flexible Foundation, Water Explicitly Modeled, Wedge Not Fixed to Foundation, Concrete Damage Model for Dam	2.8
	Model with Contraction Joints, Flexible Foundation, Water Explicitly Modeled, Tied with Failure Sliding Surface at Dam/Foundation Interface	2.86
	Model with Contraction Joints, Flexible Foundation, Water Explicitly Modeled, Tied with Failure Sliding Surface and Uplift Pressures at Dam/Foundation Interface	2.84
	Model with Contraction Joints, Flexible Foundation, Water Explicitly Modeled, Wedge Not Fixed to Foundation or Dam, Low Temperature Condition Applied to Dam	2.8