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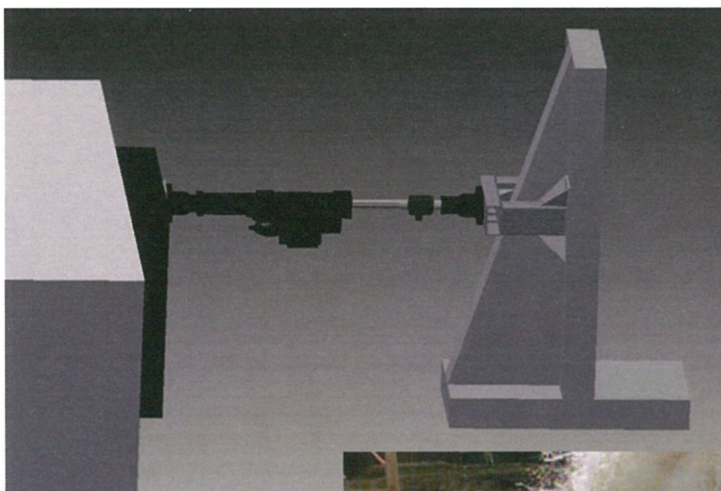
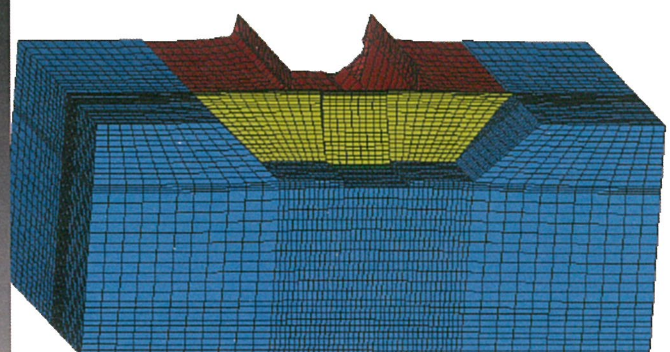
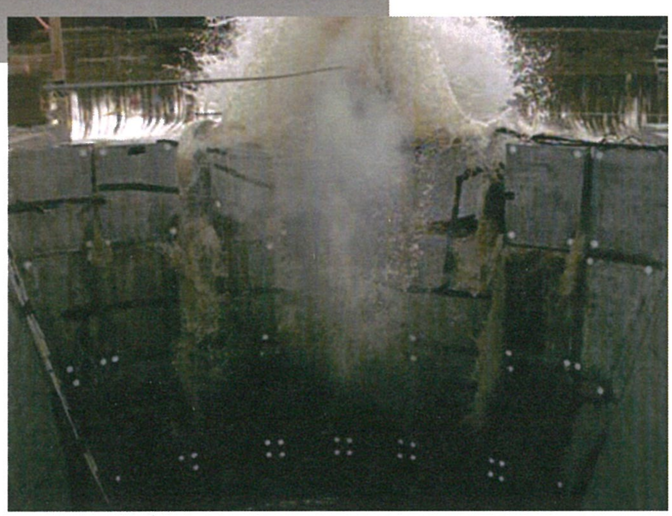
Finite Element Analysis Research Compilation

Dam Safety Technology Development Program







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14. ABSTRACT

The purpose of this Technical Memorandum is to compile all the work that has been performed in the past to validate the finite element analyses that have been performed by the Waterways and Concrete Dams Groups 1 and 2. This TM was written in conjunction with the research project 'Dam Performance during Seismic Loading Research Program'

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Mission Statements

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to Indian Tribes and our commitments to island communities.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Report DSO-2016-1

Bureau of Reclamation

Dam Safety Technology Development Program
Waterways and Concrete Dams Group 1, 86-688110

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Acronyms

2-D	two-dimensional
3-D	three-dimensional
ALE	arbitrary Lagrangian-Eulerian
CSCM	Continuous Surface Cap Model
EM	engineering monograph
FE	finite element
LS-DYNA	Nonlinear Dynamic Analysis
Reclamation	Bureau of Reclamation
TM	Technical Memorandum

I. Introduction

The purpose of this Technical Memorandum (TM) is to compile all the work that has been performed in the past to validate the finite element analyses that have been performed by the Waterways and Concrete Dams Groups 1 and 2. This TM was written in conjunction with the research project 'Dam Performance during Seismic Loading Research Program.'

II. Literature Review

Several research projects have been performed in recent years to better understand results and validate the finite element software and methods used for analysis of concrete structures including various types of dams and waterways. The purpose of these validation studies, as well as a brief discussion of their results, are presented below.

A. Effects of Hourglassing Control in LS-DYNA for Static Loads

The discussion presented here was summarized from [1]. Further discussion regarding the details of this research project such as (but not limited to) graphs and numerical data can be found in [1].

1. Background and Purpose

The explicit part of LS-DYNA is an effective and efficient method for seismic, blast, and nonlinear analyses. Explicit analyses use simple eight-noded solids with single integration points. This allows for fast integrations through the elements.

The issue of hourglass modes is best described by the following text taken from the LS-DYNA Theory Manual:

> *The biggest disadvantage to single-point integration is the need to control the* ***zero energy modes*** *which can arise, known as hourglassing modes. Undesirable hourglass modes tend to have periods that are typically much shorter than the periods of the structural response, and they are often observed to be oscillatory. However, hourglass modes that have periods that are comparable to the structural response periods may be a stable kinematic component of the global deformation modes and must be admissible. One way of resisting undesirable hourglassing is with a viscous damping or small elastic stiffness capable of*

stopping the formation of the anomalous modes but having a negligible effect on the stable global modes.

Since hourglass deformation modes are orthogonal to the strain calculations, work done by the hourglass resistance is neglected in the energy equation. This may lead to a slight loss of energy; however, hourglass control is always recommended for the underintegrated solid elements. The energy dissipated by the hourglass forces reacting against the formations of the hourglass modes is tracked and reported in the LS-DYNA output files MATSUM and GLSTAT. In assessing hourglassing in LS-DYNA, it is helpful to know how internal energy within the model is calculated. Hourglass energy should be about 10 percent or less of internal energy to ensure that the model is functioning properly with respect to hourglassing modes.

2. *Method of Study*

To study the effects of hourglassing parameters particular to LS-DYNA, a finite element model was generated and analyzed for static loading conditions. The finite element model is shown below in Figure 1. A number of internal variables within the program were determined to be likely key players in the hourglassing behavior of the model and were varied to determine their effects on the model as a whole, as well as the individual materials of which the model was composed.

The model used in this study was a simplified representation of Elephant Butte Dam in New Mexico. The modeled representation of the dam had a cross section identical to Elephant Butte Dam above elevation 4215, making it 195 feet tall. An example of the layout is shown in Figure 1.

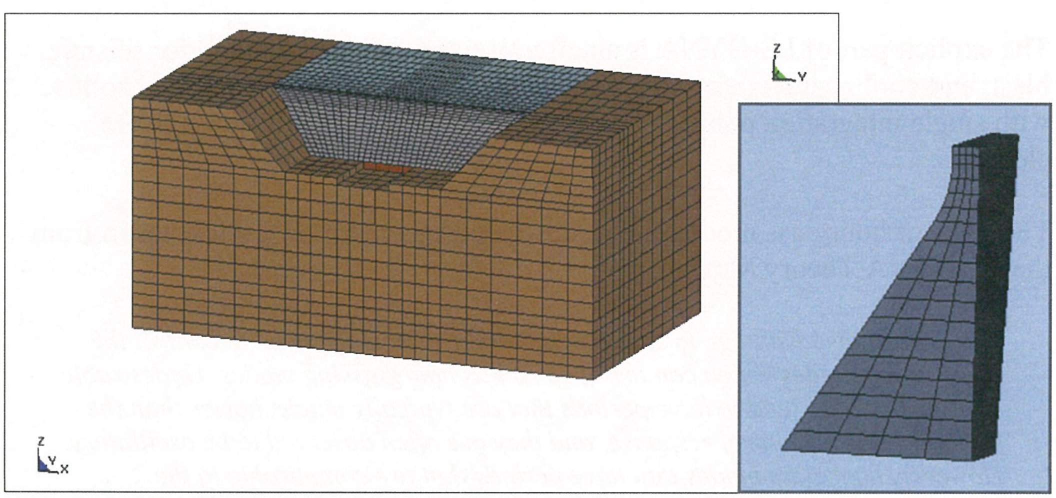


Figure 1.—The LS-DYNA model used in the hourglassing study.

There were a total of twelve models created with six different variations of key model parameters to determine their impact on hourglassing. These variations included:

1. IHQ = 3 or 5 for the dam and foundation.
2. IHQ = 1, 3 or 5 for the water.
3. QM = 0.05, 0.10 or 0.15 for the dam and foundation.
4. QM = 0.0001, 0.05, 0.10 or 0.15 for the water.
5. Foundation to dam relative mesh size at ratios of 2-to-1, 1-to-2 or 1-to-1.
6. Contact surface characteristics.

The IHQ (hourglass control type) options available in LS-DYNA include 3 (Flanagan-Belytschko viscous form), 5 (Flanagan-Belytschko stiffness form), or 1 (standard LS-DYNA viscous form). QM is an hourglass coefficient which varied on the recommendation of LS-DYNA from 0.0001 to 0.15. Eleven of the twelve models for this project used a tied, or fixed, surface between the dam and foundation. The twelfth model had a tied surface created using AUTOMATED_SURFACE_TO_SURFACE_TIEBREAK with high tiebreak values while all other variables remained the same as the other models.

Reclamation has typically used a stiffness hourglassing control type for most materials in a model. In LS-DYNA, this is identified as IHQ = 5.

3. *Summary of Results and Recommendations*

The models were compared numerically and graphically in order to draw some conclusions for obtaining optimal results in terms of reducing hourglassing. The technical data can be reviewed in detail in the original report [1].

Based on the results of this study, the following recommendations were developed for the purpose of providing guidance on limiting hourglassing. It should be noted that these recommendations are not necessarily universal; they are specific to the models involved in this study.

- For all materials except fluid elements, stiffness IHQ = 5 is recommended by LS-DYNA.

- Dam (slave surface) mesh can be made comparatively finer than foundation (master surface) mesh as a means to prevent hourglassing.

- Varying QM within the range of 0.05 – 0.15 for dam and foundation may be helpful in reducing hourglassing. Smaller values of QM should be used first, as they appear to yield slightly lower hourglassing energy results.

- Default values should be used for bulk viscosity. If any of the defaults need to be changed, it can be done for a single material in the *HOURGLASS card for that material, or globally in the CONTROL_ BULK_VISCOSITY card. It is not anticipated that changing the bulk viscosity default values, or even turning off bulk viscosity altogether (by assigning very small values to the coefficients in the *CONTROL_ BULK_VISCOSITY card) will have a significant effect on the results.

- The default hourglassing viscous IHQ = 1 value should be used for fluids. In conjunction with this, use *HOURGLASS (QM = 0.001 to 0.0001).
  - Hourglassing energy was greater when IHQ = 1 was used, it was still within acceptable levels (less than 10 percent of internal energy).
  - Water pressure in the reservoir compared well to theoretical values for both IHQ = 1 and IHQ = 5.
  - LS-DYNA has recommended using IHQ = 1 and QM = 0.001 to 0.0001 for fluid elements.
  - Current studies indicate that an IHQ = 2 with QM = 0.1 allows the water to flow with a dam moving on a foundation.

- High normal and shear tiebreak values used with contact surface type AUTOMATIC_SURFACE_TO_SURFACE_TIEBREAK (analysis 6) may limit hourglassing along the contact; however, penetration of one surface into the other may also occur.
  - Analyses using TIED_SURFACE_TO_SURFACE (analysis 2) did not exhibit penetration of the dam into the foundation when viewed at high magnification. Analysis 2 experiences larger amounts of hourglassing in the foundation (4x) and rock block (2x), as compared with the small amount observed in analysis 6.

B. Use of Eulerian-Lagrangian Interaction for Seismic Loading

The discussion presented here was summarized from [1]. Further discussion regarding the details of this project such as (but not limited to) graphs and numerical data can be found in [1] as the purpose of the discussion herein is to summarize and present the results from the research project.

1. Background and Purpose

The primary reason for attempting to use an Eulerian mesh is to better demonstrate the behavior of a fluid material. In Eulerian codes, the model consists of fixed cells through which the fluid flows. Lagrangian modeling of fluid is possible; however, since fluids have little or no shear strength, significant deformation of the material in a Lagrangian analysis can reduce time steps and cause an analysis to diverge. It is also sometimes difficult to properly model sliding interfaces between Lagrangian fluid elements and solid elements. Overly constrained nodes at intersections of multiple contact surfaces may cause hourglassing and pressures in the fluid that are not accurate.

LS-DYNA offers a way to model the fluids in a Lagrangian model with an arbitrary Lagrangian-Eulerian (ALE) capability. ALE capability was developed to better model high pressure impulse type loadings in a fluid such as blasts and impacts, where large deformations of the fluid are likely. There has been some concern with using ALE modeling for analyses extending to the time limits that are required for seismic analysis. ALE calculations are appropriate for a short duration problem like blast or impact. However, when ALE is used for a much longer duration analysis, the run time may be impractical. A test of the capabilities of a coupled Eulerian/Lagrangian model considering static and seismic loads is summarized here.

2. *Method of Study*

a. No Void Static Analysis

ALE modeling uses a solid element mesh defined with fluid material properties and ALE material grouping to estimate the behavior of an Euler type mesh. An ALE model can consist of one or more connected parts/meshes making up the ALE domain. Multiple physical materials or multi-material-groups (MMG) may flow through this ALE domain. When the reference domain is allowed to move, it is referred to as ALE. If this domain is fixed in space, it is called Eulerian. The material is allowed to flow between the initially defined shape of the fluid body and whatever fluid area is defined around the initially water-filled area.

One of the major benefits that LS-DYNA offers in modeling water using ALE is that all of the interfacing between ALE cells and Lagrangian elements is done internally by the code. The user is not required to specify these interfaces as would be required in a purely Lagrangian analysis.

Details regarding the model are similar to what was provided for Item A. The water was modeled using ALE cells, and the foundation and dam were modeled using Lagrangian elements.

A benefit to using ALE is that loads can be applied very quickly. Gravity was applied to the model over a span of only 0.1 second of computational time and the resulting hydrostatic pressures were accurate. In addition, the model behaved very well under static loads. In a Lagrangian system, gravity is usually ramped up over a number of seconds to ensure stability of the water elements.

b. Void Static Analysis with Sliding

Next, the model was modified to include a void space downstream of the water where the middle dam block was located. When the dam block was released and allowed to slide, the water was expected to flow into the void space to fill the gap. Figure 2 shows the added block of void into which the water could flow when the dam block slid.

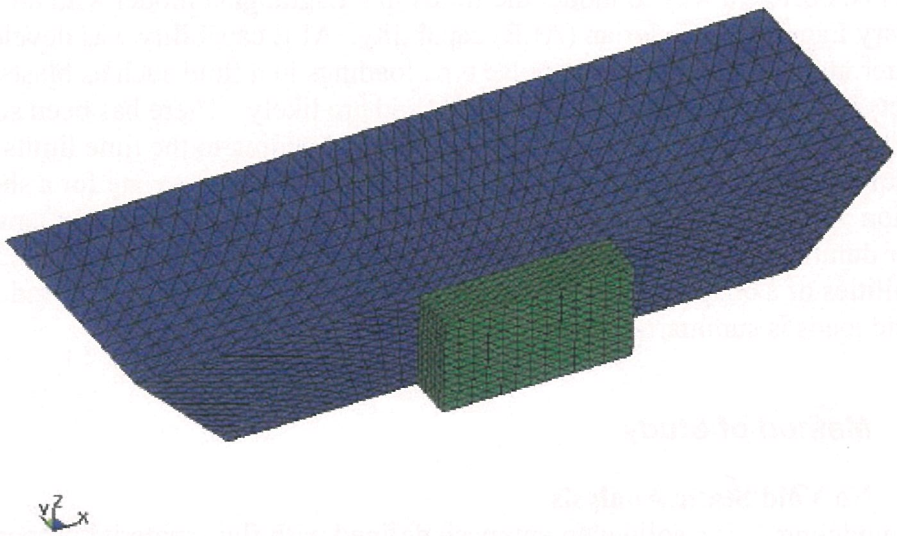


Figure 2.—Modified water mesh to include void space.

This version of the model resulted in water pressures which were incorrect after the dam began to slide. Specifically, the uniform water pressure profile was lost when sliding occurred, and incorrect pressures were calculated in the water.

c. Earthquake Analysis with Tied Dam Blocks

The model was then modified to tie the dam to the foundation so sliding was not allowed. An earthquake load was applied to a horizontal plane of nodes deep in the foundation, and the water pressures on the dam were measured. The results of this analysis were compared to the results from an analysis of the same model using Lagrangian elements for the water.

For this example, the water did not respond correctly with the applied seismic load. The dynamic change in pressure seen in the Lagrangian elements was not captured with an Eulerian water mesh.

These sensitivity analyses show that when water was modeled using Eulerian cells, a single large wave hit the dam at approximately 13 seconds, and no other significant interaction occurred between the water and the dam during the seismic event. This may be the result of incorrect application of boundary conditions to the model.

3. *Recommendations and Summary*

The results of this study indicate that there can be difficulties in using ALE for seismic analyses of dams. In general, ALE is a more accurate way of applying hydrodynamic loads to dams. ALE must be investigated further, specifically how to resolve issues with boundary conditions and how to use the proper procedure in creating and running this type of analysis.

C. Application of Traction Loads

The discussion presented here was summarized from [1]. Further discussion regarding the details of this project such as (but not limited to) graphs and numerical data can be found in [1] as the purpose of the discussion herein is to summarize and present the results from the research project.

1. *Background and Purpose*

At Reclamation, seismic time history loads have traditionally been applied using accelerations or velocities at depth in the foundation. These loads were applied on a horizontal plane of nodes within the foundation. They could not be applied on a model face that also had a boundary condition that allows dissipation of energy (nonreflecting boundaries). Application of accelerations or velocities at a plane in the foundation results in a "fixed" or rigid plane at which the seismic input originates. Complications in the propagation of vertical waves occur because the vertical waves are reflected back up at the plane of application resulting in trapped energy. This manifests itself as spikes at various frequencies on the response spectrum.

When loads are applied as traction loads, the wave is not trapped by the plane of application, and energy can be better removed at the nonreflecting boundaries at the base and vertical sides of the model foundation.

2. *Method of Study*

The model used for this study was a parallelepiped with dimensions that are typical of foundation models used to model Reclamation concrete dams. The length, width and depth of the model were 5475 feet, 5465 feet and 2950 feet, respectively, as shown in Figure 3. The load was applied along a plane located 925 feet above the base. The model was first analyzed using velocity time histories at nodes as seismic load input. The analysis was repeated using traction forces at element faces as seismic load input, and the results were compared. The response of a node at the center of the top surface was considered. No damping was applied in these analyses.

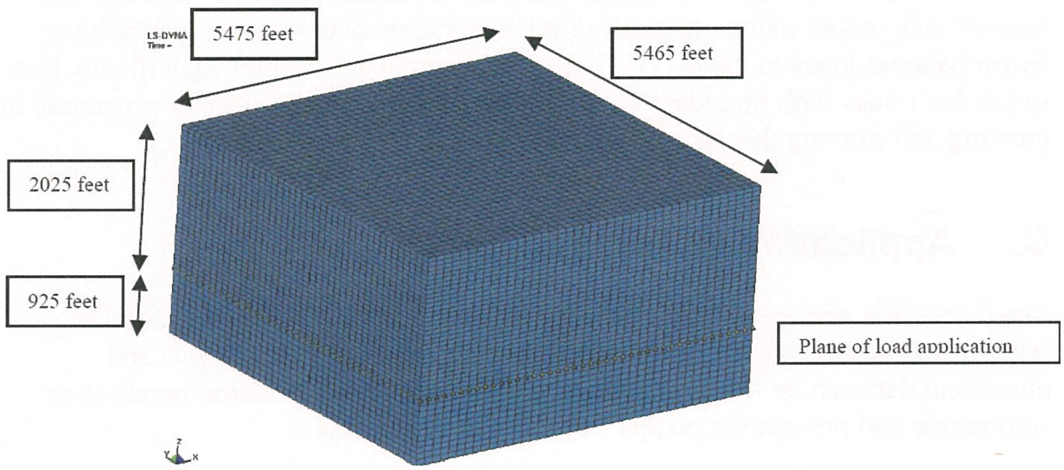


Figure 3.—Finite element model used for traction loads study.

The command to apply traction loads in LS-DYNA is *LOAD_SEGMENT_SET_NONUNIFORM. The information applied by the user includes a load curve, a set of element faces along the plane of load application, a scale factor, and the direction cosines. A faceset is defined in TrueGrid at the plane of load application. The *LOAD_ SEGMENT_ SET_NONUNIFORM command allows pressures loads to be applied in each of the three global directions.

3. *Recommendations and Summary*

The application of seismic events using acceleration time histories results in errors because of trapped energy in the model. This can be seen in a mismatch of the response spectrum generated by the finite element model as compared with the free field. Seismic loads should be applied using traction loads as waves are not trapped by the plane of application.

D. Shear Key Research

The discussion presented here was summarized from [2]. Further discussion regarding the details of this research project such as (but not limited to) graphs and numerical data can be found in [2] as the purpose of the discussion herein is to summarize and present the results from the research project.

1. Background and Purpose

Concrete arch dams are usually constructed of adjacent cantilever sections that are separated by vertical contraction joints. The contraction joints, typically spaced 40 to 50 feet apart, release tensile arch stresses caused by shrinkage and temperature drops in mass concrete that can otherwise create radial cracking. Shear keys in the contraction joints are an important component intended to maintain the arch shape of the dam by resisting upstream/downstream relative displacements between adjacent cantilevers through the transfer of shear forces.

The objectives of this study were to determine how much strength and resistance the shear keys can develop based on a detailed joint model compared to capacities demonstrated with contraction joints as they are typically implemented in finite element (FE) models of concrete arch dams. The purpose of the study was to give risk teams a more defensible basis when judging if an arch dam maintains its arch action during seismic events that move the dam upstream and open contraction joints. The study also was intended to help risk teams better estimate probabilities of failure for arch dam contraction joints with and without shear keys.

The stress distribution in a concrete arch dam is typically described in terms of arch and cantilever actions. The stability of an arch dam is provided by the compressive arching action in the horizontal plane that causes loads to transfer into the abutments. The transfer of loads into the abutments reduces the percentage of load carried by cantilevers. While compressive arching action utilizes the relatively high compressive strength of concrete, cantilever actions can create tensile forces that can damage concrete.

The amount of load carried by arching action and cantilever action depends on the relative stiffness of the arch, cantilevers, foundation, and abutments. In an indeterminate structure, the stiffer components carry more load and less-stiff components carry less load. During seismic events, the relative load carrying capacity provided by the arch and cantilever actions also becomes dependent on the resistance across the contraction joints as arch stiffness is reduced when inertia loads cause upstream displacements and joint opening.

2. Method of Study

This research project was separated into two parts: a literature review regarding the effects on the response of concrete arch dams; and a finite element analysis of a two-dimensional shear key joint.

a. Literature Review Summary

The literature review provided an indication of developed strength of the shear keys and insight about the assumed strength and displacement behavior of the shear keys relative to the overall concrete arch dam response.

A fracture mechanics approach to determine shear key strength is based on cracking and crushing of the concrete and shows that the shear strength of a key is approximately half of the concrete compressive strength when the normal stress is about half the concrete compressive strength and the shear strength diminishes to zero when the normal stress reaches the concrete compressive strength. The shear strength of the key also tends to zero for low normal stress.

A rock mechanics approach that considers tangential and normal displacements indicates that the initial relationship between shear and normal stress depends on the friction angle and the inclination angle of the asperities in the joint. The rock mechanics approach shows higher shear strengths than that predicted by the fracture mechanics approach.

The fracture and rock mechanics approach show good agreement in shear failure prediction for normal stresses up to about one-half the compressive strength of concrete. After this point, the rock mechanics approach predicts much higher shear strength than the fracture mechanics approach. The fracture mechanics approach is considered to provide a more accurate shear failure condition because it is based on the geometry of a shear key, and the rock mechanics approach is based on the geometry of small asperities in rough, undulating surfaces. However, the fracture mechanics approach does not incorporate normal displacements so this method cannot predict the shear strength of a partially opened joint. A finite element model was developed in LS-DYNA to explore the effects of stress redistribution for a partially opened joint.

Reclamation finite element studies [2] have typically shown that models with contraction joints capture the major changes in the response of a dam, even for those dams that assume strong shear keys that prevent tangential displacements in the joint. This suggests that joint opening, rather than joint shear strength, has a greater influence on the response of a concrete arch dam. Specifically, all the models that include contraction joints demonstrate that stresses are redistributed from the arches to the cantilevers, and the fundamental vibration period of the dam lengthens when joint opening occurs during the earthquake. These studies have also shown that joint openings are typically greater in the upper portions of the dam; however, the joints do not open enough to render the shear keys ineffective. Joint opening causes an increase in cantilever stress and was shown to cause tensile cracking in the upper portions of the downstream face of finite element analyses completed for Morrow Point Dam [2]. The results of these models indicate the importance of shear key strength for safe dynamic performance of concrete arch dams because the shear key properties have a large influence on the response of the dam.

b. Finite Element Model

A finite element analysis of a two-dimensional shear key joint was performed in LS-DYNA to study stress distribution during loadings that simulate possible load conditions experienced in a dam contraction joint during one pulse of a seismic event. The model for the finite element analysis captures a section of a typical keyed contraction joint in a concrete arch dam. Four variations of the model were developed with four different shear key configurations representative of typical shear keys found in concrete arch dams as shown in Figure 4. The variations are summarized as:

- **Analysis I – Sliding Initiation.**—To study the relative magnitudes of normal and shear loads that initiate sliding for a model with boundary conditions such that dilation can occur.
- **Analysis II – Closed Joint.**—To study the stress distribution for a closed joint with boundary conditions such that no dilation occurs.
- **Analysis III – Partially Open Joint.**—To study the stress distribution for a partially open joint with boundary conditions such that no further dilation occurs.
- **Analysis IV – Degraded Material Property.**—To examine material property degradation by reducing the modulus of the bottom part by one-half for a closed joint with no dilation.

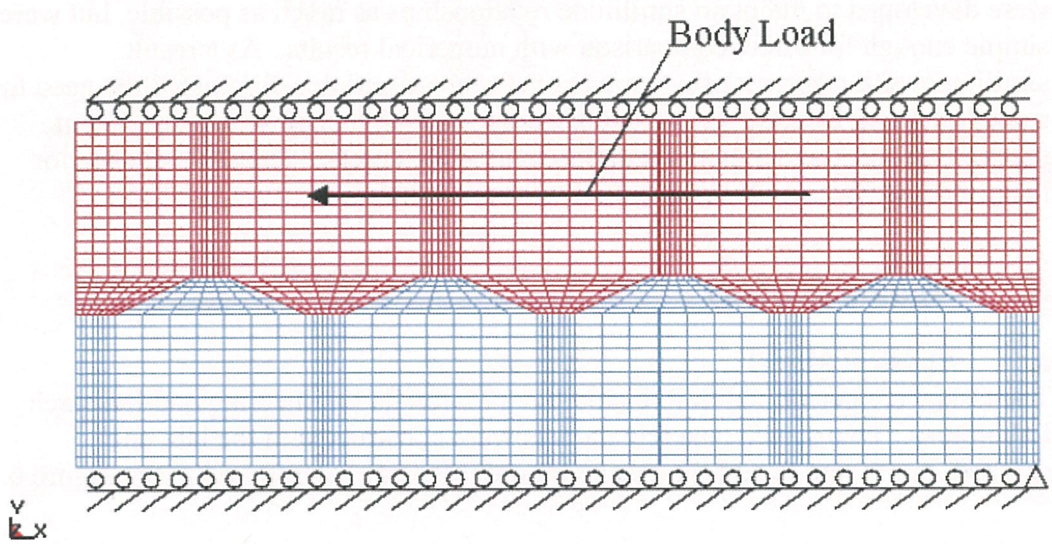


Figure 4.—FE model with loads and boundary conditions.

3. Finite Element Study Conclusion

The finite element analyses results provided information on the behavior of arch dam contraction joint shear keys under loading normal and transverse to the joint. Overall, the study results showed that the stresses which initiate joint sliding occur before shear failure for normal stresses less than about 1/5 of the concrete compressive strength. Essentially, if sliding begins before shear failure, then joints will experience dilation. Dilation allows for an increase in upstream displacements, which can result in higher cantilever stresses.

E. Investigation of the Failure Modes of Concrete Gravity Dams Physical Model Tests

The discussion presented here was summarized from [3] and [4]. Further discussion regarding the details of this research project such as (but not limited to) graphs and numerical data can be found in [3] and [4] as the purpose of the discussion herein is to summarize and present the results from the research project.

1. Background and Purpose

The purpose of this laboratory investigation was to produce physical model test results to develop a correlation between the material testing program and the material property requirements of the model used in the ABAQUS code. The geometry of the physical model was scaled from Koyna Dam. Physical models were developed to maintain similitude relationships as much as possible, but were simple enough for direct comparison with numerical results. As a result, similitude with reservoir effects was not attempted and this eliminated the need to model coupling effects. Two models were tested—a model with a natural but preexisting crack and a model without preexisting cracks. The scale chosen for these models was a 1:50 scale.

2. Method of Study

a. Physical Model

The tests were performed in Reclamation's Materials Engineering and Research Laboratory. Two scaled concrete models were constructed in the lab: one cracked, see Figure 5, and the other a monolithic (uncracked) model, see Figure 6.

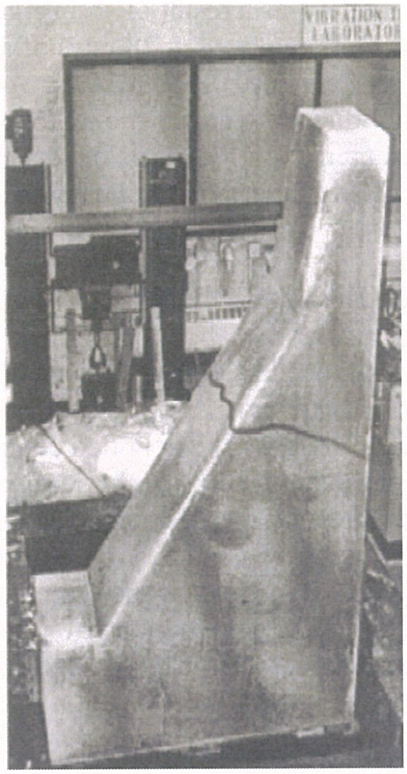


Figure 5.—Konya model with shrinkage crack mounted on shake table.

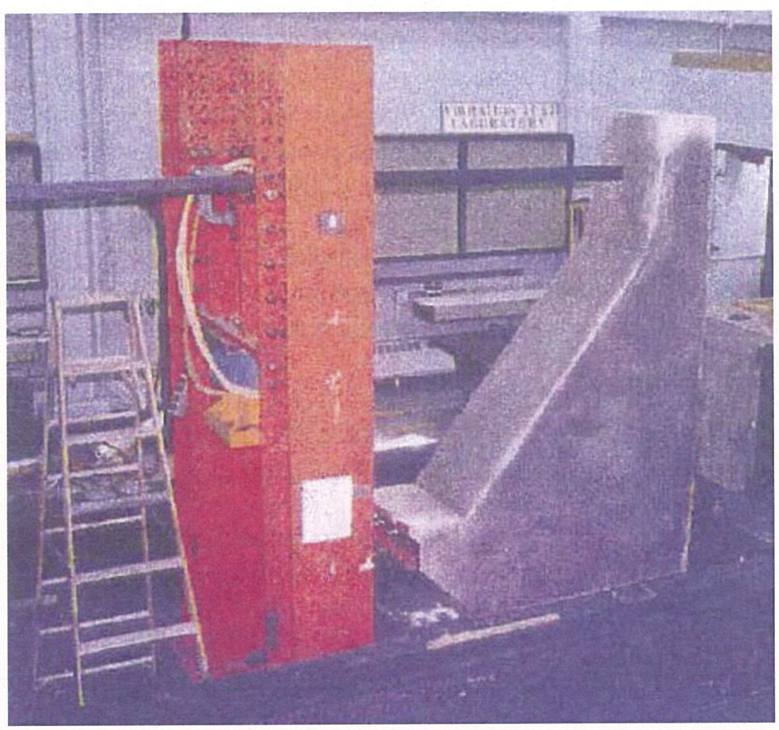


Figure 6.—Konya monolithic model mounted on shake table.

The crack in the cracked model was a result of shrinkage cracking which was discovered after the forms were removed at 28-days of age. The 1:50 scale models were 8.5-feet tall and weighed 7,850 pounds. A slab, representing a foundation, was cast monolithically with the model to provide a fixed condition at the base of the dam. Instrumentation measured displacements and accelerations of the model and input motion of the actuator during the test.

b. ABAQUS Model

All analyses used the same three-dimensional, solid numerical model created to match the physical dimensions of the laboratory model. The ABAQUS/Explicit dynamic numerical analysis procedure was used to analyze four different nonlinear finite element mesh numerical models of the uncracked Koyna scale model. Of special interest were to what extent tied contact surfaces influence numerical cracking analysis and to determine the sensitivity of cracking predictions to varying mesh density where no contact surfaces were used. The first two numerical meshes used tied contact surfaces and the last two used no contact surfaces. Each numerical model was then run until failure of the structure. Cracking patterns and time of failure predictions from the numerical model results were then compared to the physical uncracked model Koyna laboratory model results.

3. Conclusions and Discussion

- The first and second ABAQUS numerical meshes failed prematurely. Stress concentrations appear to develop along the tied surfaces causing cracks to develop where none would appear in a model without tied surfaces. This illustrates the need to properly size the elements near the contact as to avoid any stress concentrations.

- Cracking for ABAQUS numerical models using meshes three and four occurred at a more reasonable time and agreed with laboratory results. The cracking pattern and time at failure also compared well to laboratory experiments.

- The uncracked physical model failed with a material failure that was believed to be characteristic of cracks observed in the field.

- The cracked physical model and the uncracked physical model produced general mode shapes and damping that were similar for small accelerations.

- The nonlinear numerical model demonstrated that the initial bond on a typical shrinkage crack, even a crack visible to the eye on multiple faces, needs to be overcome before sliding can be initiated.

- Once sliding starts, the nonlinear effects create significant changes in the dynamic response under a constant frequency sinusoidal input motion. The amplitude of the acceleration of the portion above the crack in the model actually becomes less than the amplitude of the acceleration of the base, and the response is phase shifted. Put simply, the model base can slide back and forth beneath the top with the motion being nearly uncoupled.

- During the uncracked physical model test, a nonlinear change in the base fixed-boundary condition created a highly nonlinear and indeterminate boundary condition. This nonlinear change also showed large changes in the dynamic response of the model. Unfortunately, this boundary condition change observed for the physical model made exact time history matching with the numerical model results impossible.

- Both physical models failed at approximately 2.2 g's of acceleration. In the cracked physical model, the preexisting crack allowed a slow progressive sliding during the cyclic motion. In the uncracked physical model, a crack was initiated in less than 1/30 of a second and sliding occurred for a number of cycles before the top of the model toppled. The toppling is believed to be related to the vertical accelerations caused by the boundary condition change.

- Results from the physical kinematic failure model (sliding) can conceivably be time step matched to verify nonlinear numerical models. Results from the uncracked physical models can be verified in a general manner to confirm the cracking pattern and threshold accelerations required for failure.

F. Shaking Table Study to Investigate Failure Modes of Arch Dams

The discussion presented here was summarized from [3] & [5]. Further discussion regarding the details of this research project such as (but not limited to) graphs and numerical data can be found in [3] & [5] as the purpose of the discussion herein is to summarize and present the results from the research project. Comparison of model arch dams with ABAQUS finite element model results can be found in [5] and analysis of physical model arch dams and concrete material properties can be found in [3].

1. Background and Purpose

The purpose of this study was to evaluate how an arch dam might fail during an earthquake and to determine how well numerical models predict structural performance of a physical model during very large levels of shaking. In this study [5], large laboratory shake table tests were performed on five generic scale models of representative arch dams and reservoirs, ramping the dynamic loading up to a level to fail the model. A series of finite element numerical models were created and analyzed using the ABAQUS finite element code to determine how accurately the dynamic response and ultimate failure can be predicted.

2. Method of Study

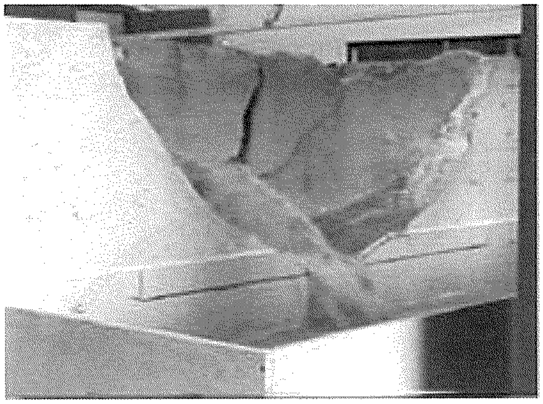
Physical models [5], each at a scale of 1:150, were built and tested on a shake table at Reclamation's Materials Engineering and Research Laboratory. Five physical models, each with a different joint configuration were constructed and tested. The five joint configurations modeled were:

1. Monolithic, not shown in Figure 8.
2. A single vertical joint at the arch center.
3. A single horizontal joint at mid height.
4. 17 vertical joints spaced evenly along the arch, not shown in Figure 7.
5. 17 evenly spaced vertical joints combined with 2 horizontal joints.

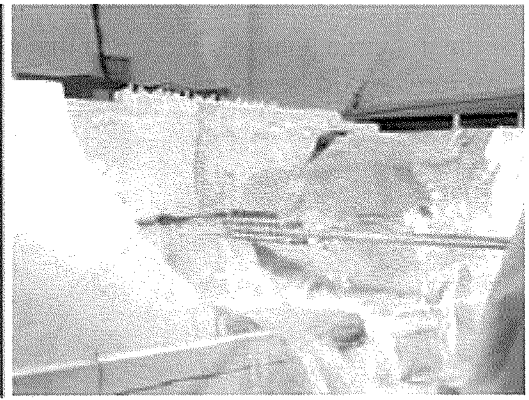
The physical models, see Figure 7, were placed in a concrete block foundation on the shake table and tested to failure.

Physical model parameters, such as dimensions and material properties, were adjusted to approximate similitude. A reservoir was modeled behind the scaled structure, but it should be noted that the fluid used was pure water with no adjustment for viscosity similitude. Input motions and structural response were measured, and high quality video was used to capture the failure modes.

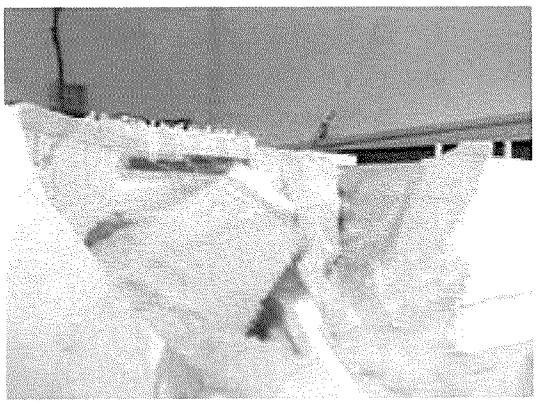
A major goal of this research was to determine how well numerical models predict the structural failure of laboratory models. Therefore, in parallel with the physical model studies, numerical ABAQUS finite element models were developed and a series of studies were performed, see Figure 8. The studies included a series of linear elastic analyses, a series of analyses used to calibrate the non-linear material properties, and a series of five non-linear analyses used to model the dam's response using the five different joint configurations selected for the physical model study.



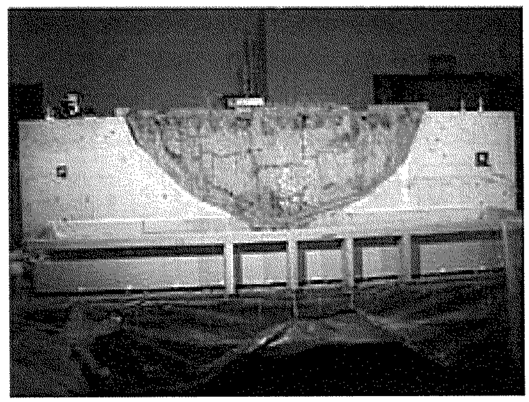
Monolithic Model



Horizontal Joint Model



Vertical joint model



17 vertical, 2 horizontal joint model

Figure 7.—Lab models at time of failure.

Linear elastic static, modal and dynamic analyses of the monolithic numerical model with and without discrete modeling of the foundation were compared to quantify the effect of the removal of the foundation model from the analyses. The comparison of results from these analyses indicated that finite element models of the dam with fixed boundary conditions can be used to accurately simulate the conditions in the shake table physical model tests with a two thirds reduction in the time required to complete the numerical analyses. Therefore, the remainder of the numerical studies was completed using finite element models with fixed boundary conditions rather than using a discrete modeled foundation.

A second study was conducted to determine the most appropriate version of ABAQUS for use in the remainder of this study. Specifically, the explicit version of ABAQUS was compared with the standard version of ABAQUS by performing a linear elastic analysis of the uncracked dam using both versions of the ABAQUS program. Results were compared to determine if a preferred version could be identified, and to verify that both versions of the program would produce the same results linear elastically. The results of this comparison indicated that either program could be used equally well for static loads. However, the results of the dynamic analyses indicated that the explicit version of the ABAQUS program should be used for time history analysis of the earthquake loads.

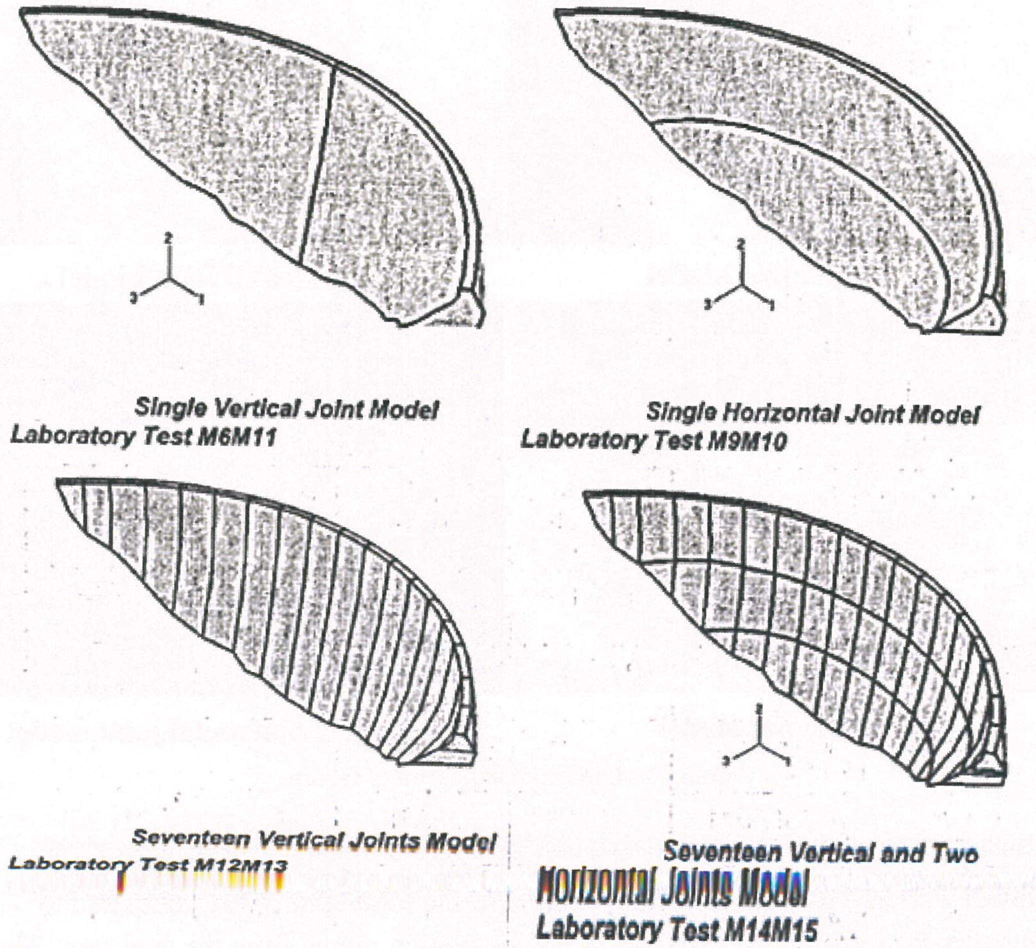


Figure 8.—Corresponding numerical arch dam models.

Static and dynamic linear elastic finite element analyses were completed for each of the five physical model joint configurations tested on the shake table. Stresses, joint conditions, and displacements resulting from these five numerical models were compared with the physical model laboratory results in terms of acceleration and displacement time histories prior to cracking. This series of analyses were linear elastic in terms of the material property models used, however geometric non-linearities such as opening, closing, and sliding along joint surfaces and, in some cases, sliding and rocking of independent concrete blocks, were modeled. The monolithic dam numerical model was used to calibrate the non-linear material properties. The results of the non-linear analyses were compared to laboratory physical model results in terms of the input amplitude at which cracking was observed to initiate, the amplitude at which failure occurred, crack

patterns, and failure modes. Each of the five joint configurations was then analyzed using the numerical nonlinear material properties, and results were compared to laboratory physical model results.
The physical models were built at a scale of 1:150 and consisted of concrete with physical properties based on averaged values of tested properties of various existing dams. A new, low-strength concrete mix was designed based on previous work done to produce a similitude-appropriate concrete mix for the scaled physical model. Early tests with this material demonstrated that all scaled parameters desired could not be met simultaneously so coefficients for calculating the required similitude relations were derived.

For the physical model studies, a shake table was constructed that had movement constrained to a single axis (horizontal only). The table was tested for its fundamental response modes and also tested in motion with accelerometers to determine its capabilities for use at higher frequencies. The table responded well for input frequencies below 22 Hz, which was below the table's lowest natural frequency of 30 Hz, but higher frequencies were eliminated for testing. Response of the table was best at frequencies of 26 Hz and below. For this reason, a similitude simulation of an earthquake motion was not used. Rather, for practical reasons associated with the table, and for simplicity in numerical model calibration, a sinusoidal motion was selected.

3. *Conclusions and Discussion*

a. Static Linear Elastic Analyses

The analysis results demonstrated some of the benefits of including vertical joints in dams. The physical models with 17 vertical joints exhibited lower and more evenly distributed stresses. Increasing the number of joints in the numerical model increased the amount of energy absorbed in frictional effects at contact surfaces, which introduces a source of damping in the model. There were no laboratory measurements for static loading conditions associated with the physical models, so no comparisons were made for the shake table tests under static loads. The ABAQUS results for static loading seemed reasonable for all five numerical models.

b. Dynamic Linear Elastic Analyses

All the numerical analyses resulted in the same general deformation sequence, which matched the deformation sequence observed in the laboratory physical model shake table test results. The central portion of the dam crest was observed to deflect back and forth in the upstream-downstream direction. In numerical models with vertical joints, these joints tend to open as the dam crest deflects in the upstream direction and close as the dam crest deflects in the downstream direction. This joint opening action was also observed in the shake table physical models as a very rapid vibration along the joints followed first by the loss of the reservoir through the joints, then additional cracking (in locations other than along joints) along with the formation of independent blocks, and finally the loss of arch support as the displacement of large blocks. Comparisons of dynamic displacements showed that the numerical analysis results match the physical model shake table measurements quite well at the lower amplitude input motions (prior to model cracking). It was difficult to make comparisons at the top of the physical model because instrument failure at this location was a problem in most of the shake table tests. This dam crest is also the location of the maximum displacements and, in three of the models, the location of a vertical contraction joint. The dynamic accelerations resulting from the numerical analyses did not match the physical model laboratory test results. The numerical analysis results tended to be higher in both amplitude and frequency.

c. Dynamic Non-Linear Analyses

Results of the ABAQUS smeared crack numerical analyses corresponded with the respective laboratory physical model test results with respect to cracking, although interpreting the results did require some engineering judgement. The finite element analyses predicted concrete cracking in the same locations and orientations as the cracking that occurred in the laboratory physical models. The cracking thresholds computed from the analyses and recorded during the laboratory tests were within the same range in most cases. The numerical analyses also indicated that the first cracks tended to form the upstream face of the dam, which could not be recorded in the laboratory physical model experiment. A good correlation was found between the failure modes seen in the laboratory physical model tests and those that could be extrapolated from the ABAQUS smeared crack analyses. The laboratory physical model failures were kinematic failure modes. Specifically, concrete cracking led to the formation of independent blocks, which were subjected to shaking for 30-second time periods at each amplitude level until a kinematic block failure occurred. In the numerical models, failures were associated with non-convergence of the numerical analysis. Specifically, when the numerical models showed significant cracking in a localized area, the resulting loss of stiffness in the cracked elements led to excessive distortion in neighboring elements. Further increases in the input amplitude resulted in nonconvergence in the numerical model. The smeared cracking analysis, however, is limited in that the full kinematic failure cannot be modeled without redefining the mesh to model discrete cracks as they form to create independent blocks. Where independent blocks were incorporated into the model, such as in the 17 vertical and 2 horizontal joints model, the ABAQUS analysis did model the resulting kinematic failure well.

In general, the ABAQUS analyses were very successful at predicting the failure modes observed in the laboratory physical model tests, although engineering judgement is required in extrapolating final crack configurations and the final failure modes based on the crack patterns that exist at the time when the analysis terminates. The laboratory physical model tests using arch dam models with five different joint configurations all resulted in similar failure modes in response to a 14 Hz sine wave input acceleration applied in the upstream-downstream direction. Typically, the models cracked vertically at the center of the arch, diagonally downward from the arch quarter points, and horizontally near the base. Independent blocks were observed to form and then displaced downstream. Different joint configurations affected the size and number of independent blocks which formed, but the shape of the deformation of the dam in response to this input motion was always the same and, as a result, crack patterns were always very similar. The monolithic and single horizontal joint physical models were able to withstand higher amplitude input motions than the other joint configurations.

G. Seismic Earth Pressures on Cantilever Retaining Structures

The discussion presented here was summarized from [6] and [7]. Further discussion regarding the details of this research project such as (but not limited to) graphs and numerical data can be found in [6] and [7] as the purpose of the discussion herein is to summarize and present the results from the research project.

1. Background and Purpose

It is very important to validate any finite element analysis results with experimental data. Professor Nicholas Sitar at the Department of Civil and Environmental Engineering, University of California, Berkeley, measured accelerations and moments of earth retaining walls during seismic excitation in a centrifuge experiment as part of Linda Al Atik's doctoral thesis, see Figure 9 [7].

2. Method of Study

The Berkeley centrifuge experiment consisted of two aluminum structures retaining Nevada Sand. One of the aluminum structures was composed of relatively stiff walls with a base moment connection while the other structure was composed of flexible walls with a similar base. The shaking events consisted of Loma Prieta-SC-1, Kobe-PI-2 and Loma Prieta-SC-2. Moments at the base of each of the walls were compared with finite element model results. Figure 10 shows the deformed finite element mesh.

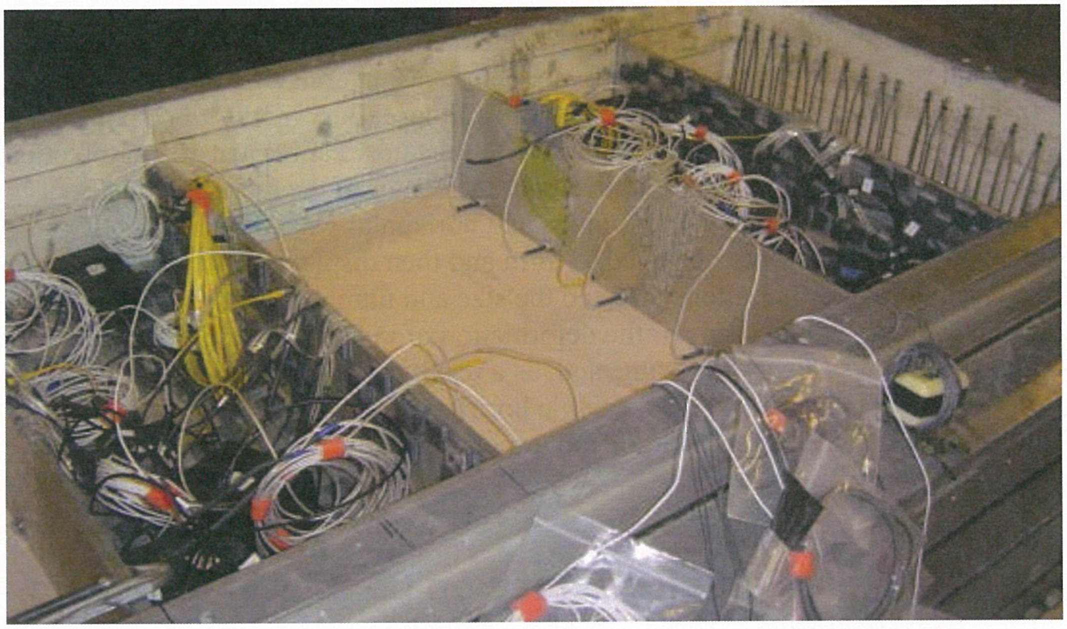


Figure 9.—Model under construction.

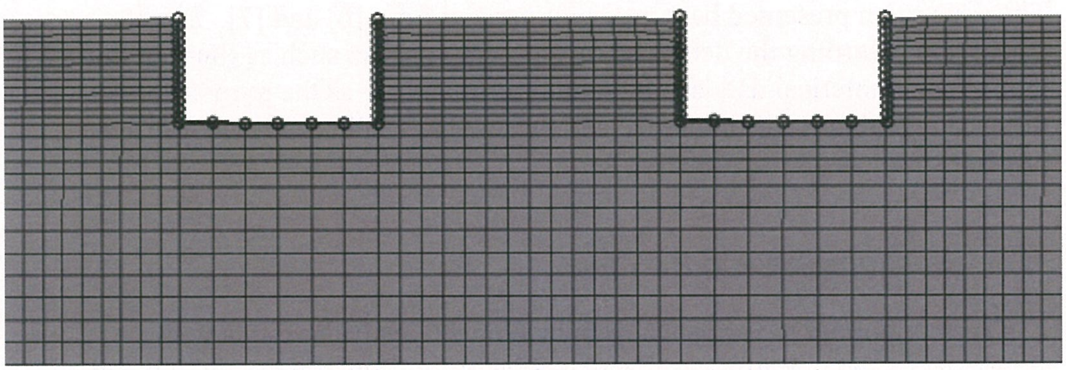


Figure 10.—Deformed finite element mesh for Loma Prieta-SC-1 shaking event.

3. Conclusions and Discussion

The centrifuge experiment was modeled in LS-DYNA using two different soil material models. The first was Material Model 25, this Material is an inviscid two invariant geologic cap model. The advantages of this model over other classical pressure-dependent plasticity models is the ability to control the amount of dilatency produced under shear loading and its ability to model plastic compaction. With Material Model 25, the computed base moment time histories compared well to the Berkeley experimental centrifuge model results in terms of peak locations and shape of the curves, see figures A-2 and A-4. The LS-DYNA analysis results indicates static moment values, peak values and residual increase in static post-earthquake moment values for the flexible wall that are less than those predicted by the centrifuge model. However, for the stiff wall, LS-DYNA over predicts the peak values during the seismic event and the static post-earthquake moment, while accurately predicting the static moment produced by the centrifuge model.

The second material model used in LS-DYNA was Material Model 16, which incorporates the Mohr-Coulomb yield surface with a Tresca limit and can be used to model soil structure interaction. Again, with Material Model 16, the moment time histories compare well to the Berkeley results in terms of peak locations and shapes of the curves, see figures A-1 and A-3. Using Material Model 16, LS-DYNA does a better job in predicting the moment time history response for the flexible wall in terms of peak moments and static post-earthquake as compared with Material Model 25. LS-DYNA again over predicts the peaks for the stiff wall, however, does a better job in predicting the static post-earthquake moment, as compared with Material Model 25.

H. Reinforced Concrete Pressure Pipe Stress Distribution Study

The discussion presented here was summarized from [8]. Further discussion regarding the details of this research project such as (but not limited to) graphs and numerical data can be found in [8] as the purpose of the discussion herein is to summarize and present the results from the research project.

1. Background and Purpose

Design of buried concrete pipe has traditionally used methods presented in Reclamation's Engineering Monograph No. 6 (EM6)—Stress Analysis of Concrete Pipe [9] that is based on the work done at the Iowa Engineering Experiment Station at Iowa State College, which was presented in Bulletin 112 from Iowa State [10]. A finite element analysis study of a pipe conditions consistent with Bulletin 112 was developed in order to compare results between current technologies and accepted, older traditional methods, see Figure 11.

2. Method of Study

The investigation, reported in Bulletin 112, included measurement of radial earth pressure on a 36-inch concrete pipe with 8 feet of soil backfill above the top of the pipe. The pipe was placed in a foundation cradle with a 90 percent projection condition (74 degree bedding angle), see Figure 12. This study was done to analyze pipe stresses resulting from the soil. Complete pipe design stresses result from several additional loadings (water inside the pipe, internal water pressure and vehicle wheel loads) that were not addressed in this study.

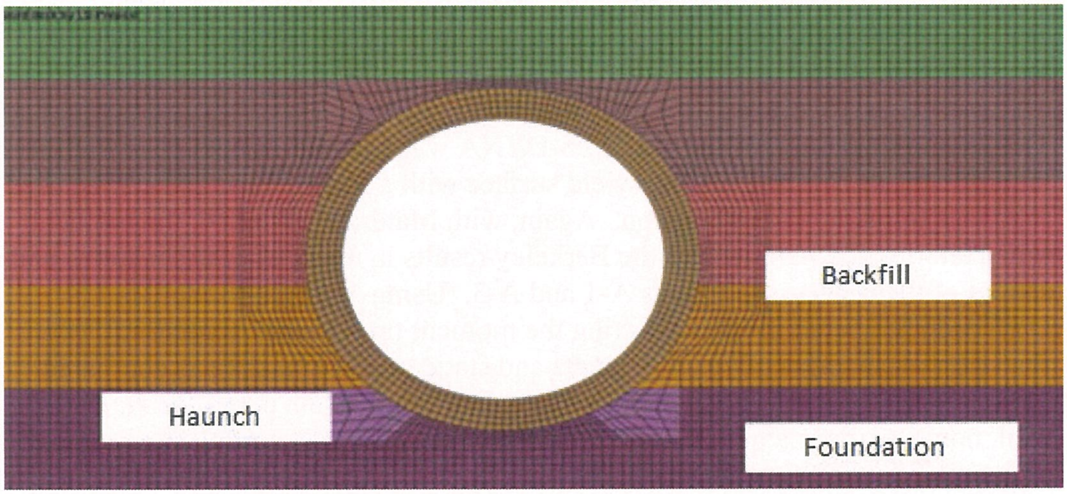


Figure 11.—LS-Dyna conduit model.

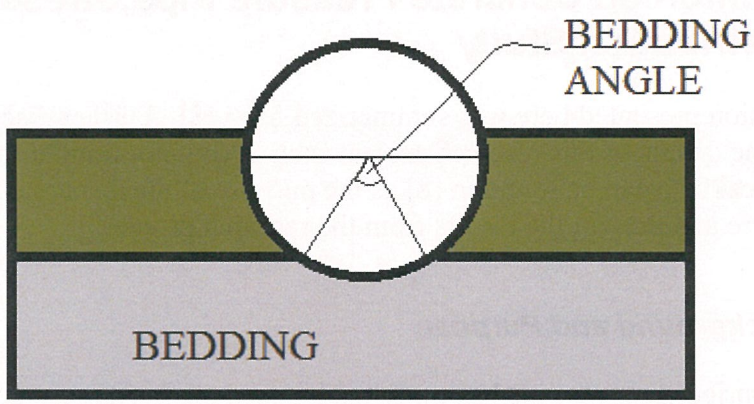


Figure 12.—Pipe bedding angle.

LS-DYNA was used to perform a 2-D finite element analysis. The LS-DYNA analysis used a bedding angle of 64 degrees instead of the 74 degrees used in the study producing Bulletin 112.

3. *Conclusions and Discussion*

- Over the top half of the pipe, all LS-DYNA analysis results indicated greater pressures on the pipe wall as compared with Bulletin 112 and slightly smaller pressures on the pipe as compared with Olander [9].
- LS-DYNA analysis results generally indicate higher pressures than either Bulletin 112 or Olander in the foundation soil with use of a Poisson's ratio of value of 0.2, and lower soil pressures with use of a Poisson's ratio value of 0.40.

- Using Poisson's Ratio of 0.2, LS-DYNA model results produce higher computed moments and shears in the pipe wall as compared with those of Olander because more side soil pressure is assumed for the Olander model. These results can be seen in Figure 13.

I. Nonlinear Dynamic Analysis (LS-DYNA) of Bradbury Dam Spillway Including Soil Structure Interaction

The discussion presented here was summarized from [11]. Further discussion regarding the details of this research project such as (but not limited to) graphs and numerical data can be found in [11] as the purpose of the discussion herein is to summarize and present the results from the research project.

1. Background and Purpose

Bradbury Dam and spillway were originally designed for a deterministic earthquake with a magnitude much less than the probabilistic earthquakes considered in the context of risk analysis. A 3-D numerical finite element study of the retrofitted spillway was developed for the purpose of reevaluating the modified structure. The 3-D finite element model accounted for topographic effects, soil structure interaction, and incorporated nonlinear concrete and reinforcement behavior. Two concrete material models were used, the Karagozian & Case concrete model and the Winfrith concrete model.

The Karagozian & Case concrete 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 showing the element completely cracked or crushed.) This model has parameter generation capacity based solely on the unconfined compression strength of the concrete.

The Winfrith model was developed by Broadhouse and Neilson over many years and has been validated against experiments. Rebar may be included using the smeared rebar approach or by the use of discrete rebar beam elements. Cracks are displayed and oriented as determined by the state of the stress in the element.

Since nonlinear material models were used for concrete and reinforcement in the evaluation of the spillway crest structure, it was judged to be important to validate the behavior of these material models.

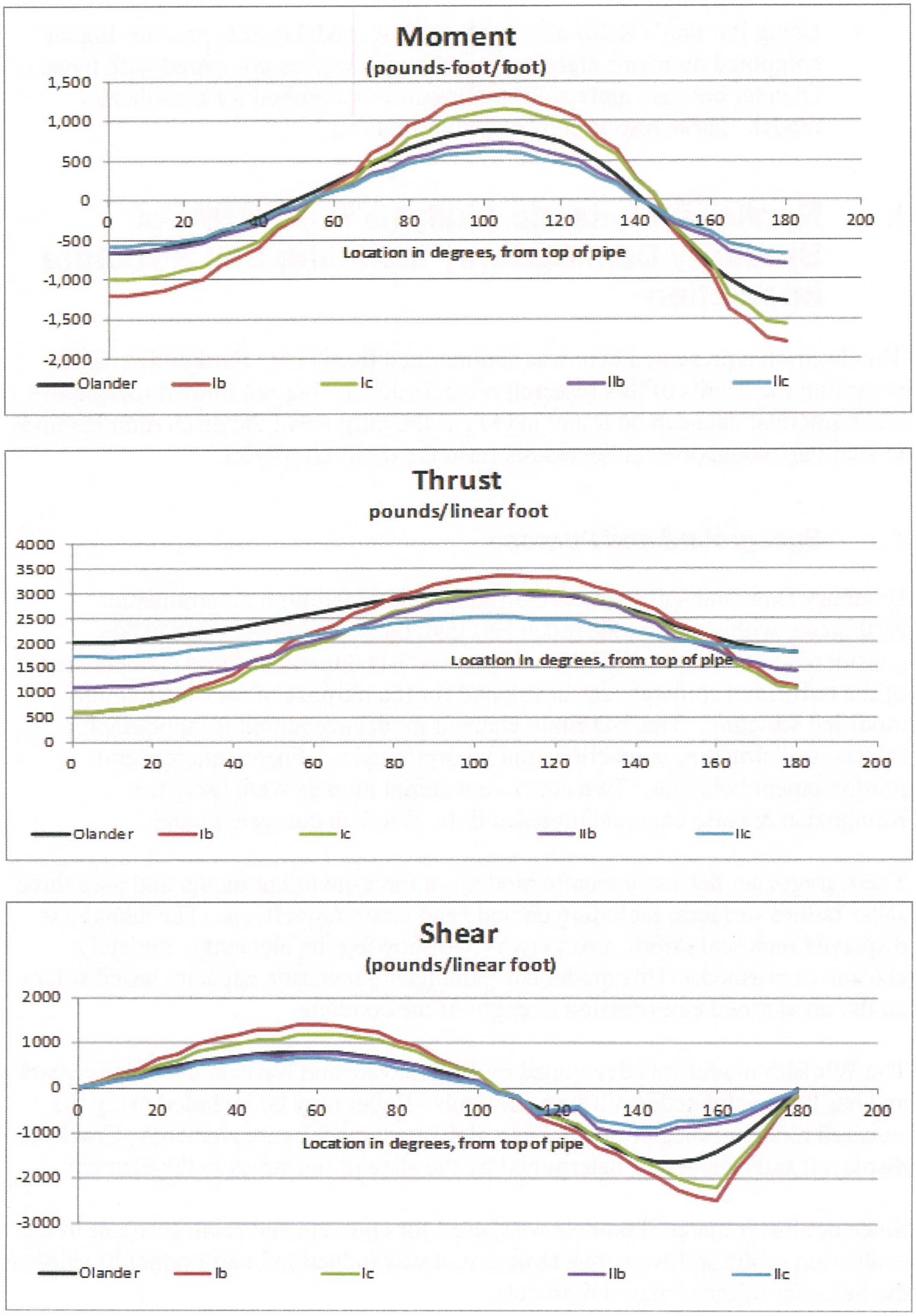


Figure 13.—Moments, thrusts, and shears in pipe compared.

J. Nonlinear Dynamic Analysis (LS-DYNA) of Bradbury Dam Spillway Including Soil Structure Interaction

The discussion presented here was summarized from [11]. Further discussion regarding the details of this research project such as (but not limited to) graphs and numerical data can be found in [11] as the purpose of the discussion herein is to summarize and present the results from the research project.

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Since nonlinear material models were used for concrete and reinforcement in the evaluation of the spillway crest structure, it was judged to be important to validate the behavior of these material models.

2. Method of Study

Two LS-DYNA concrete material models were used in the analysis of the Bradbury spillway crest structure. The two material models considered included Model 72R3, which is the Karagozian & Case concrete model and Model 84, which is the Winfrith Concrete model.

First, the behavior of a modeled column in tension and compression was considered. The column was unreinforced, as shown in Figure 14, in order to illustrate the tension and compression behavior of these two materials. Displacement control was used, since any failure of the unreinforced concrete is expected to be sudden. Next, a simply supported concrete beam with reinforcement was studied. This beam was subjected to a uniform load and flexural capacity was compared to classical solutions.

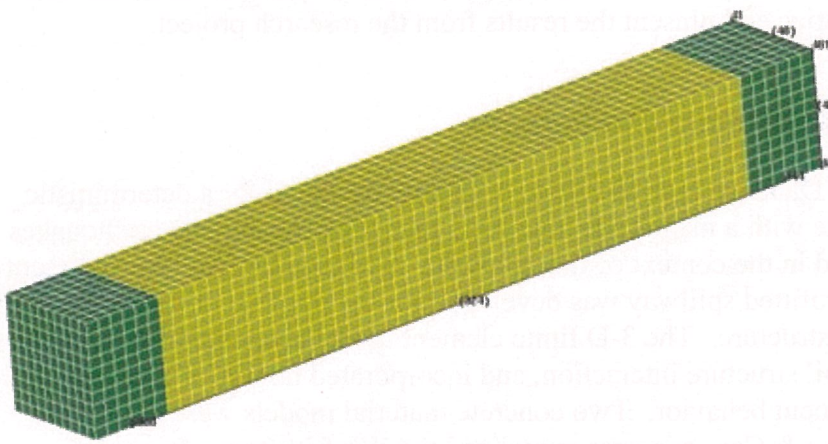

Figure 14.—Concrete column.

3. *Conclusions and Discussion*

Both concrete material models were evaluated for a variety of conditions ranging from axial tension and compression to flexural bending with different percentages of reinforcement steel. In general, if the reinforcement controlled (failure governed by reinforcement yielding), the analysis results showed close agreement with theoretical solutions. However, if concrete tension or compression is the main failure mode, results were highly dependent on the particular material model formulations. Since concrete structures are generally designed to be under-reinforced, these concrete dominated failure modes do not tend to control structural behavior and the use of the LS-DYNA material models is judged to be acceptable.

K. Spillway Wall Structural Capacity

The discussion presented here was summarized from [12]. Further discussion regarding the details of this research project such as (but not limited to) graphs and numerical data can be found in [12] as the purpose of the discussion herein is to summarize and present the results from the research project.

1. Background and Purpose

This research project concentrated on determining the structural capacity of a scaled counterforted spillway wall. Reclamation has numerous spillways and other earth retaining structures founded on soil or rock and backfilled with various embankment materials, some in high seismic areas. A laboratory physical model of a small counterforted wall was constructed and a finite element analyses was completed as part of an ongoing effort to validate the use of finite element models.

2. Method of Study

a. Lab Model

It was important to determine the correct size of the lab physical model such that one can capture all pertinent desired effects. This included choosing a model size big enough so that enough rebar detail can be incorporated, yet small enough that construction of the wall could be completed in a lab environment.

Instead of placing soil behind the wall and loading the wall with this backfill, a loading ram was used in the laboratory. This simplification still gave a good indication of the actual capacity of these walls but without the complicated behavior of soil loading, which introduces uncertainty into the system. Sinusoidal loading was applied at mid-height of the wall.

b. Finite Element Model

Truss elements were used to represent the reinforcement. The counterforted wall finite element mesh grid was aligned in such a way that the horizontal and vertical reinforcement along with the sloped reinforcement nodes were coincident with the concrete nodes. Contact surfaces were used between the counterforted wall and the hold down mechanism. These surfaces allowed for slip with minimal friction specified. A front bearing plate restricted movement of the wall and the contact force on the plate served as a good representation of the applied ram force.

Two LS-DYNA concrete material models were used in the analyses of the counterforted wall, namely the Continuous Surface Cap Model (CSCM) (MAT 159) and the Karagozian and Case Concrete (Model 72R3).

The CSCM is an elasto-plastic damage material model with rate effects and is equipped with two surfaces: the failure surface and hardening cap. A continuous intersection is maintained between the surfaces.

As described earlier, the Karagozian & Case concrete 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 showing the element completely cracked or crushed.) This model has parameter generation capacity based solely on the unconfined compression strength of the concrete.

3. *Conclusions and Discussion*

In general, the finite element analyses showed a much greater area of cracking occurring sooner with all reinforcement (counterfort moment steel, vertical and horizontal shear bars) sharing in carrying the load. The lab results indicate more discrete cracks with less load sharing between the counterfort moment steel and all other bars. The finite element analyses supported the locations of early cracking along with crack orientation. See Figure 15 for visual similarities in cracking between the lab and numerical models.

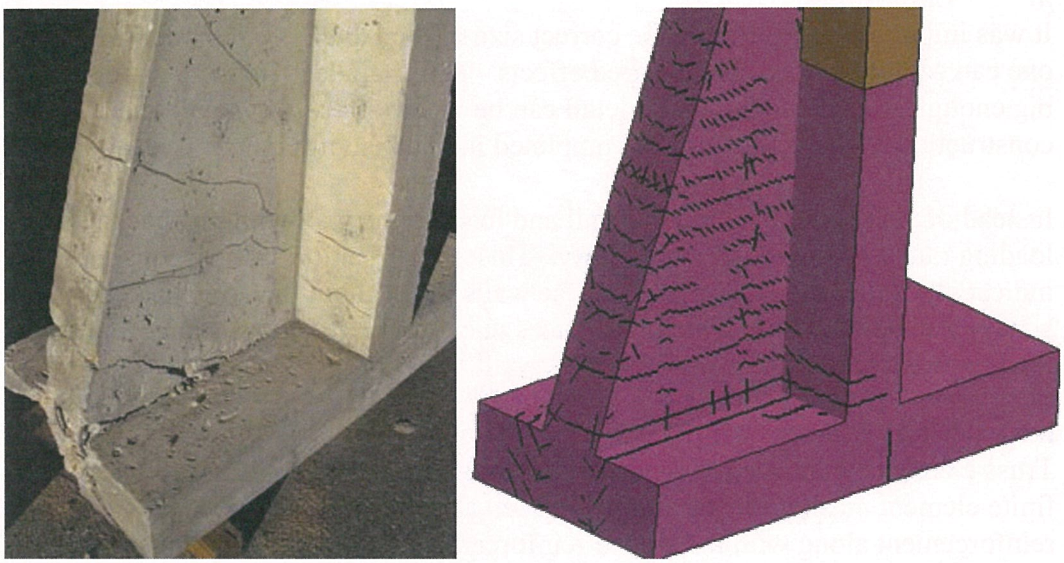

Figure 15.—Cracking in lab and numerical models.

The finite element model indicated that it takes more ram force to displace the wall a certain amount as compared with lab results. This is contrary to the belief that it takes less force to deflect a more damaged structure since the stiffness is reduced as damage increases. Finite element model results indicating that it takes more ram force to displace the wall a certain amount as compared with lab results may not be conservative. Part of this discrepancy may be due to the fact that the finite element model does not model localized concrete/reinforcement interaction. More testing is recommended to obtain a better statistical sample.

L. Comparison of EACD3D96 Computed Response to Shaker Tests on Morrow Point Dam

The discussion presented here was summarized from [18]. Further discussion regarding the details of this research project such as (but not limited to) graphs and numerical data can be found in [18] as the purpose of the discussion herein is to summarize and present the results from the research project.

1. Background and Purpose

Morrow Point is a double-curvature thin arch dam completed in 1968 with a structural height of 468 feet. The dam is 12-feet-thick at the crest and 52-feet-thick at the base. The goal of this study was to determine appropriate material properties, damping, and reservoir-bottom wave reflection coefficients for use in finite element analyses for Morrow Point Dam.

2. Method of Study

In 1985, acceleration response amplitudes were measured on a physical model of Morrow Point Dam [13] from forces induced on the dam by eccentric shakers attached to the model crest. Four basic subdivisions of numerical material properties were used to hone in on the most appropriate material property configurations to represent the responses of the physical model:

1. **H-Series.**—This series used the baseline material properties used by Hall and Duron in 1985 [13].

2. **J-Series.**—Baseline material properties determined by Jackmuah in 2000 for dynamic analysis for the Issue Evaluation of Morrow Point Dam [14] were used for this series .

3. **F-Series.**—This series used properties determined from a sensitivity study by Jackmauh in 2001 using SAPIV with massless foundation and no reservoir [15].

4. **R-Series.**—The material property for this series was slightly adjusted after reviewing the results of the F-series. Also, the shape of the reservoir finite element mesh was modified to 1) match the reservoir bottom elevation, and 2) to match the existing canyon shape, see Figure 16. In this analysis a large value for α and a foundation modulus fairly close to the concrete modulus were used.

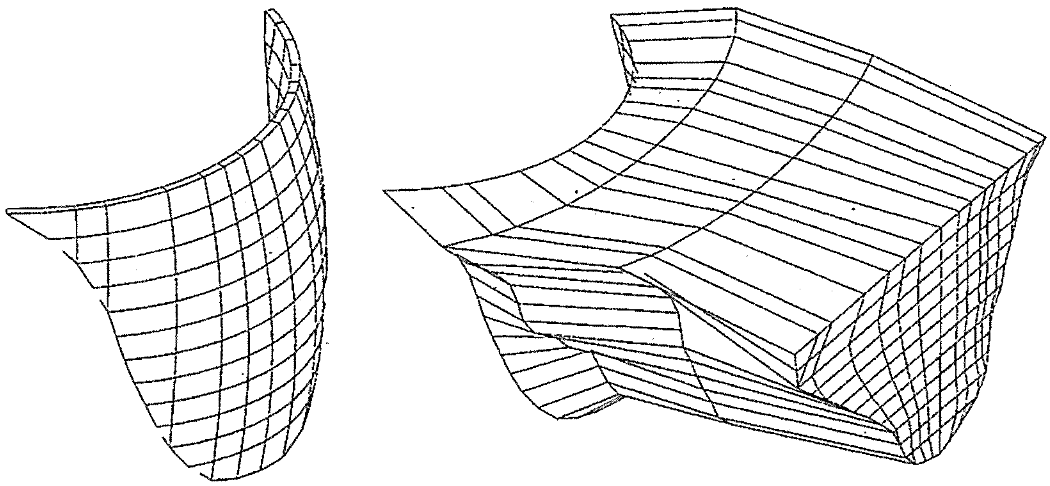

Figure 16.—R-Series dam and reservoir FE model.

3. *Conclusions and Discussion*

1. The computer acceleration response amplitudes from EACD3D96 match the measured amplitudes reasonably well if the material properties are correctly chosen and the finite element model of the reservoir conforms the existing topography, see Figure 17D. Figure 17 shows:
   a. Displacement response (real component)
   b. Displacement response (imaginary component)
   c. SRSS of a and b
   d. Acceleration response in (g/KIP)

2. Hydrodynamic interaction using compressible water with a large α matched the response curve much better than the incompressible formulation.

3. Using higher values of α matched the measured response better than lower values.

4. Material properties by Jackmuah in 2000 produced results that were greatly below the measured response.

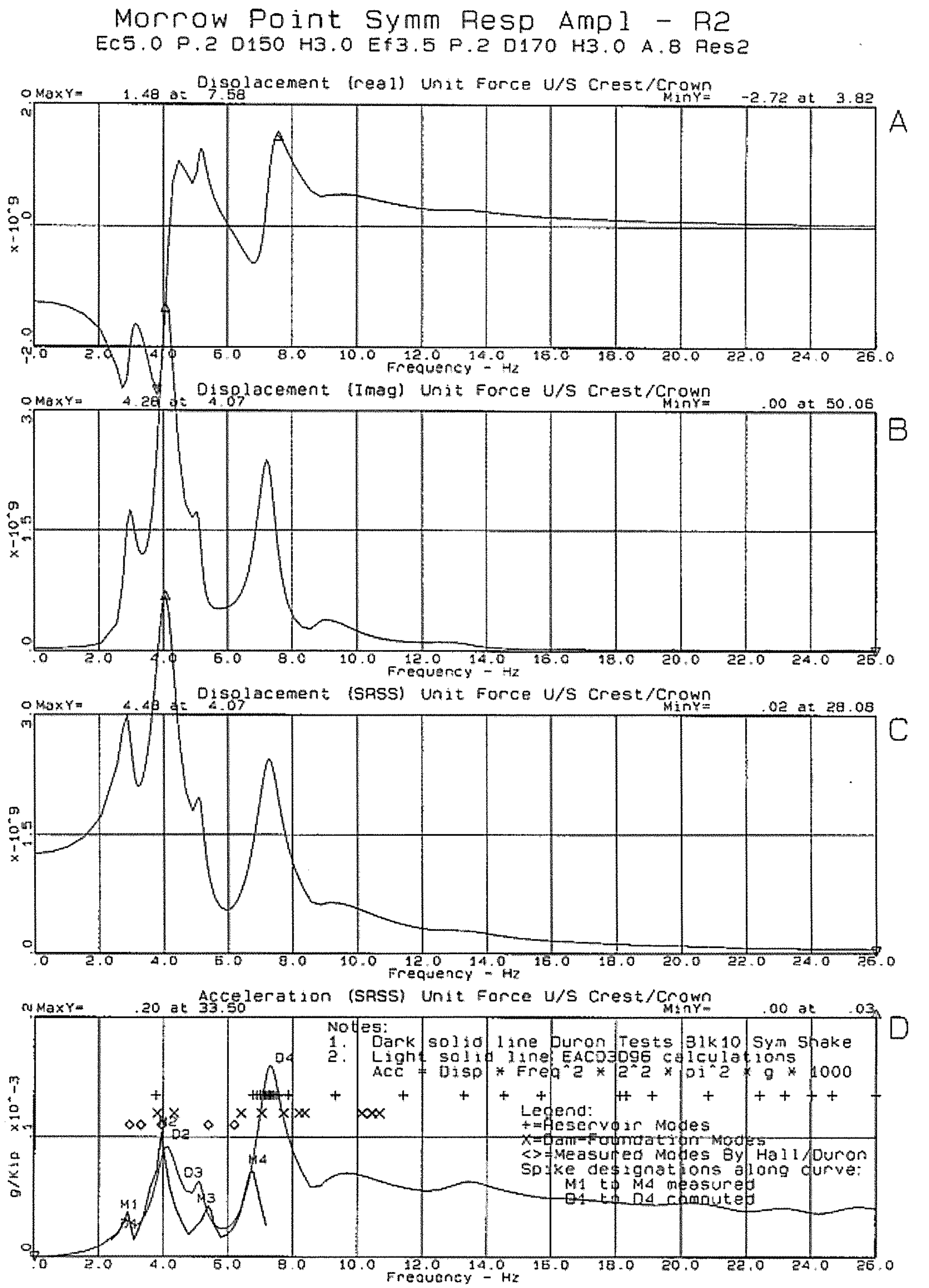

Figure 17.—R-Series numerical material and physical tests.

M. Nonlinear Dynamic Analysis of Scoggins Dam Spillway Including Soil Structure Interaction

The discussion presented here was summarized from [16]. Further discussion regarding the details of this research project such as (but not limited to) graphs and numerical data can be found in [16] as the purpose of the discussion herein is to summarize and present the results from the research project.

1. Background and Purpose

Scoggins Dam was constructed in the 1970's using modern materials and techniques but was designed for a significantly lower seismic event than current analyses predict.

2. Method of Study

Reclamation's Waterways and Concrete Dams Group 1 created an LS-DYNA model of the Scoggins dam, see Figure 18. Although the main purpose of the model was to evaluate the spillway, a comparison of the behavior of the embankment itself to FLAC results from work done by the Reclamation Geotech group was done. This comparison focused on crest deformations away from the concrete spillway near the center of the embankment dam.

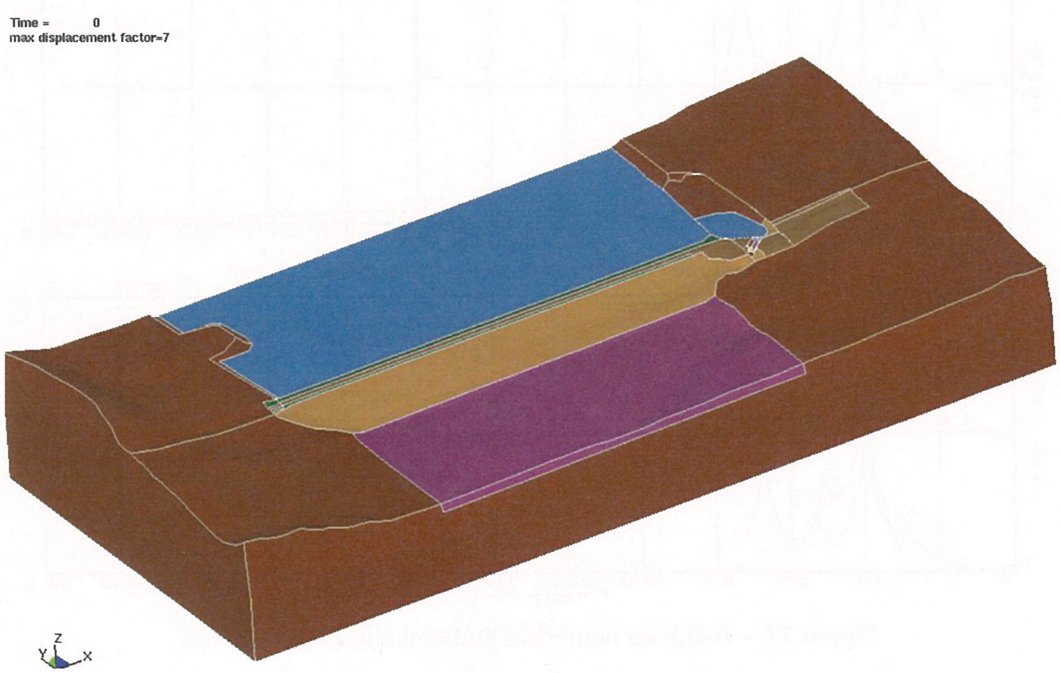

Figure 18.—Scoggins Dam LS-DYNA model.

3. Conclusions and Discussion

The downstream displacements include any vertical subsidence experienced at the crest of the dam. As can be seen in Table 1, the differences between the two models are never greater than 1 foot. The similarities of these results give confidence in the earth material models that are currently being used in these and similar soil-structure interaction models.

Table 1.—LS-DYNA and FLAC Downstream Displacements

Return Interval	Local Event LS-DYNA	Local Event FLAC	Subduction Event LS-DYNA	Subduction Event FLAC
50k	13 feet	13 feet	---	---
10k	8 feet	7 feet	23 feet	---
5k	6 feet	6 feet	17 feet	---
1k	3 feet	2 feet	8 feet	9 feet

N. Model vs Physical Blast Comparisons

The discussion presented here was summarized from a classified document. For this reason some data could not be presented in this document, but the information available to be presented here provides a source of validation of concrete structures modeled with finite element analyses.

1. Background and Purpose

DYNA, a non-commercial version of LS-DYNA, was used to simulate blast loads.

2. Method of Study

A scaled physical model of a typical arch dam was constructed in the field and filled with water. A charge was detonated behind the dam at a certain depth. Damage to the top two arch rings was observed as shown in Figure 20. At the time the photo was taken, the initial shock from the blast has already occurred, transforming the top arch rings into a series of blocks stacked on top of each other. Figure 20 shows upstream suction taking place.

A finite element model of this scaled dam was also created, see Figure 19. Contact surfaces were used to model the lift lines along with vertical contraction joints. An Eulerian grid of cells behind the dam was used to simulate the water. A charge was modeled using an equation-of-state and was detonated. Figure 19

(FE model) shows the damage and block movement at the same time as shown in Figure 20 (physical model).

3. *Conclusions and Discussion*

Qualitatively comparing both figures, one sees the block movements and damage to the dam to be very similar. This shows that the finite element code is capable of capturing phenomena such as initial blast, suction and proper energy distribution.

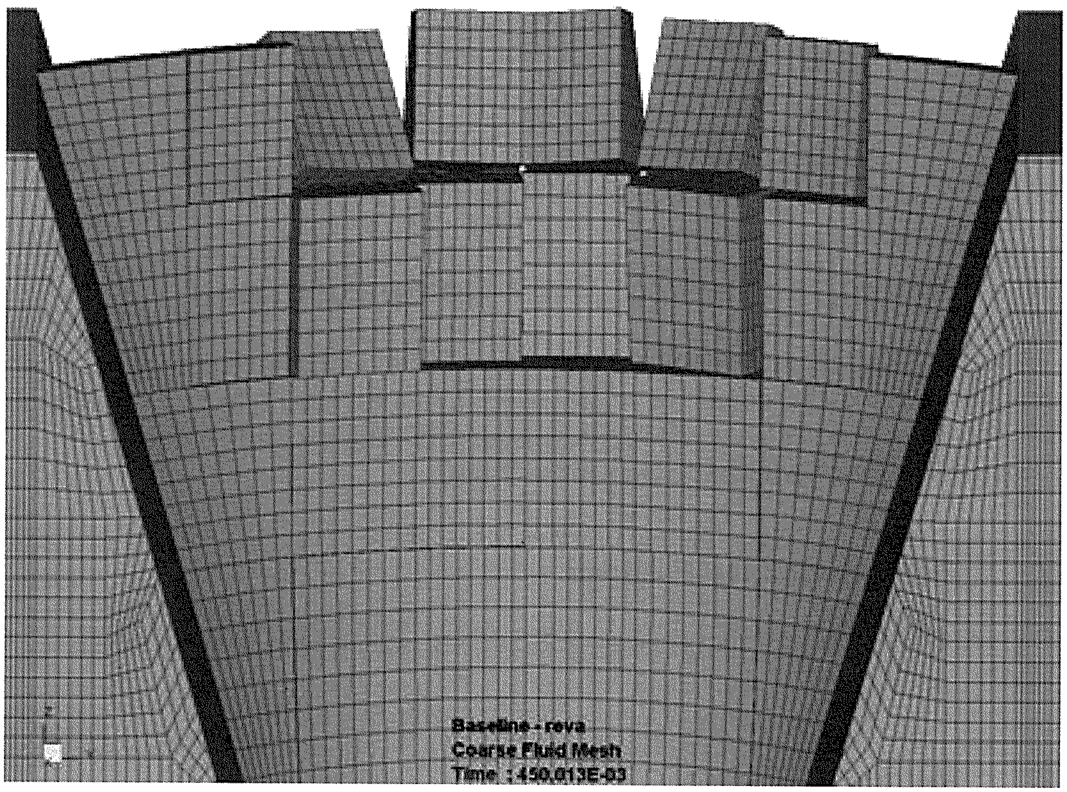

Figure 19.—LS-DYNA model.

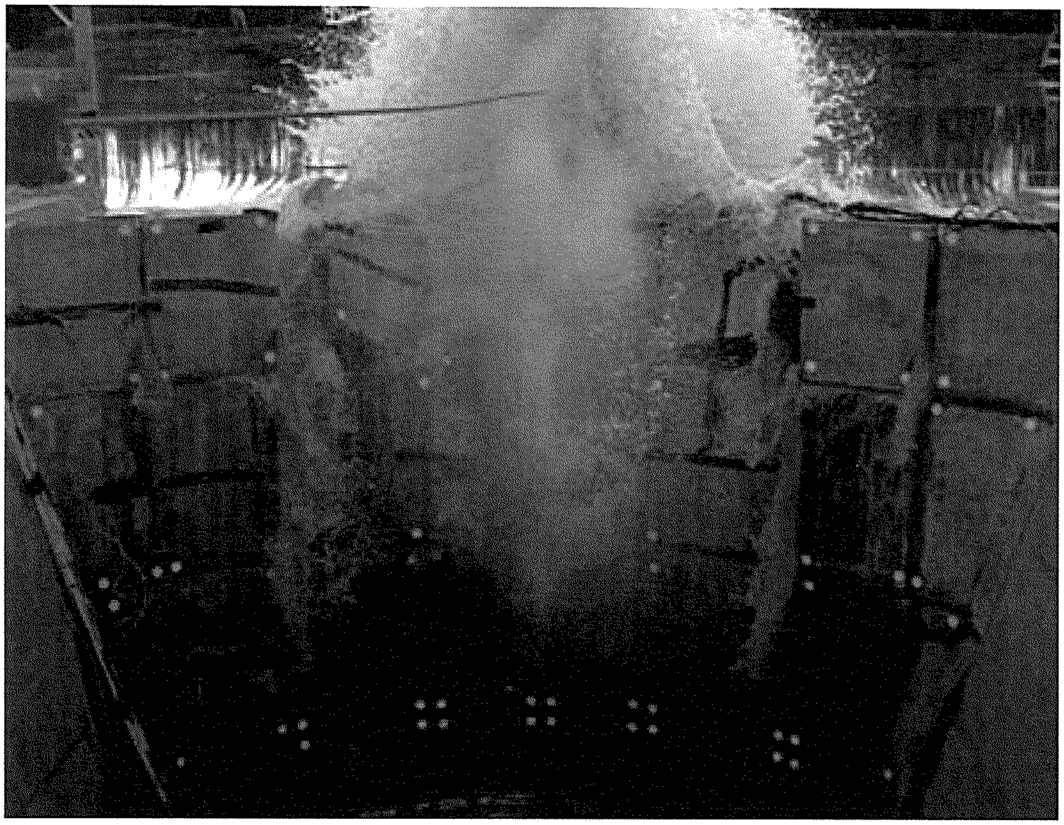

Figure 20.—Physical model.

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APPENDIX

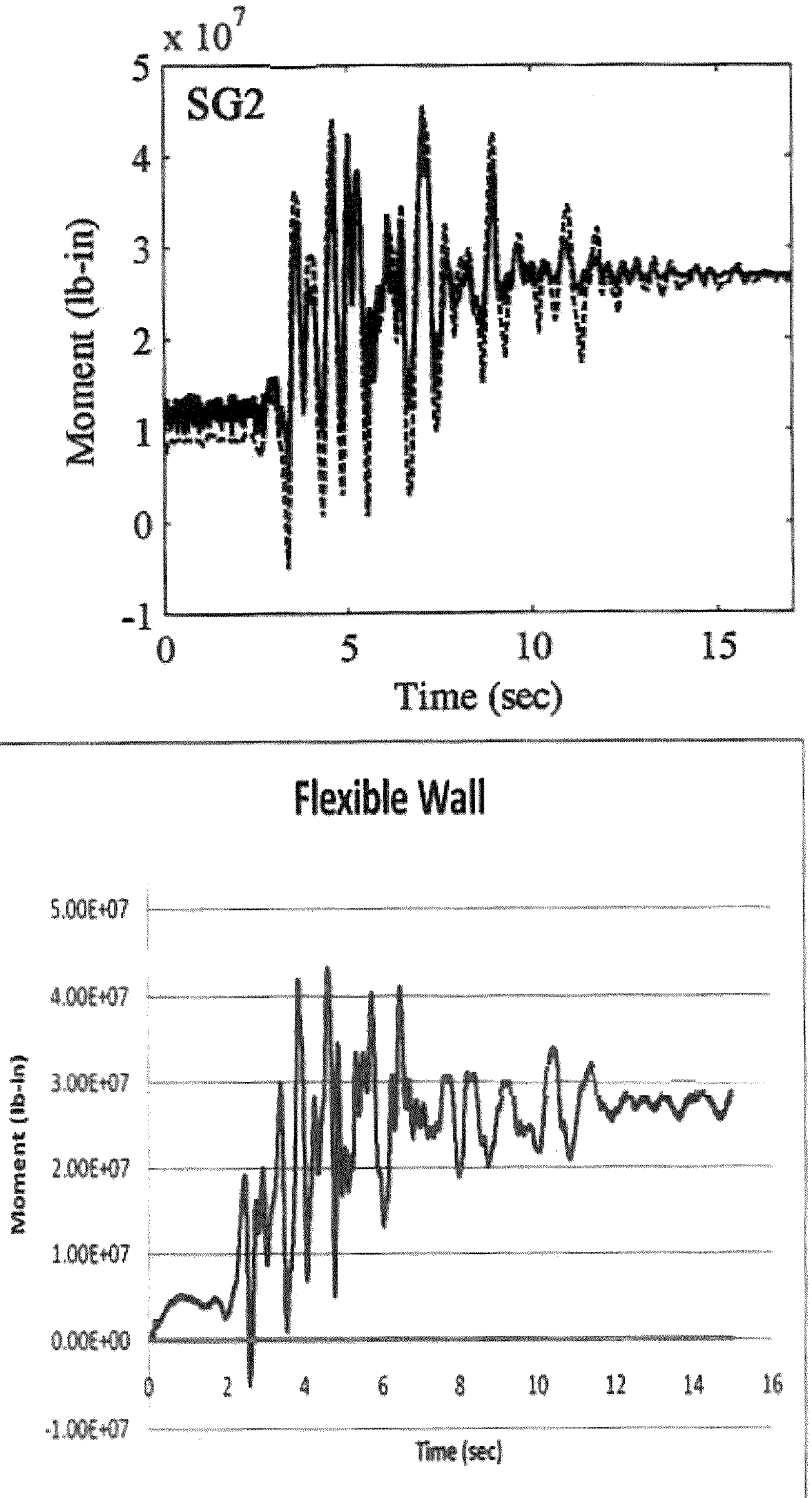


Figure A-1.—Moment time history near bottom of flexible wall (Material 16).

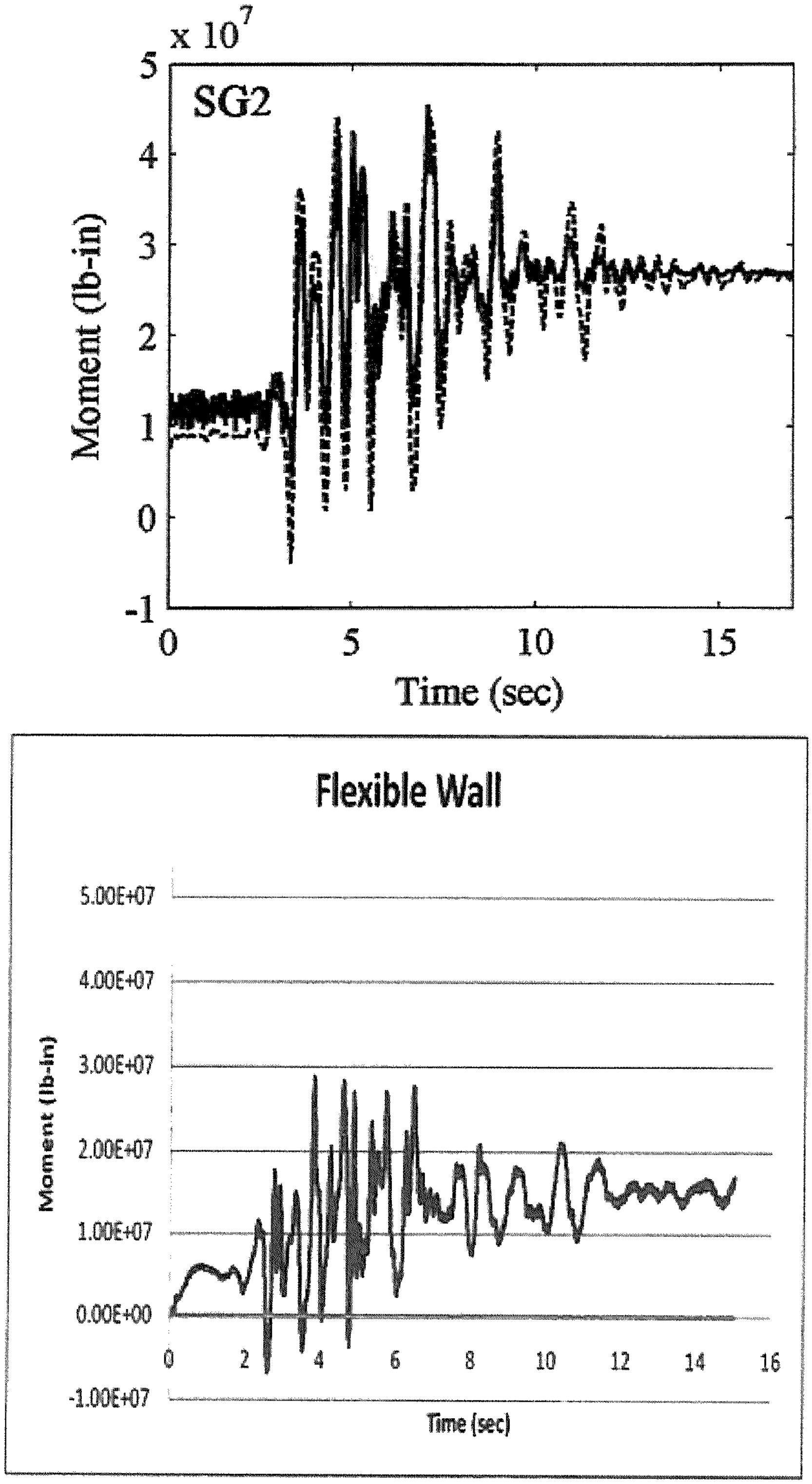


Figure A-2.—Moment time history near bottom of flexible wall (Material 25).

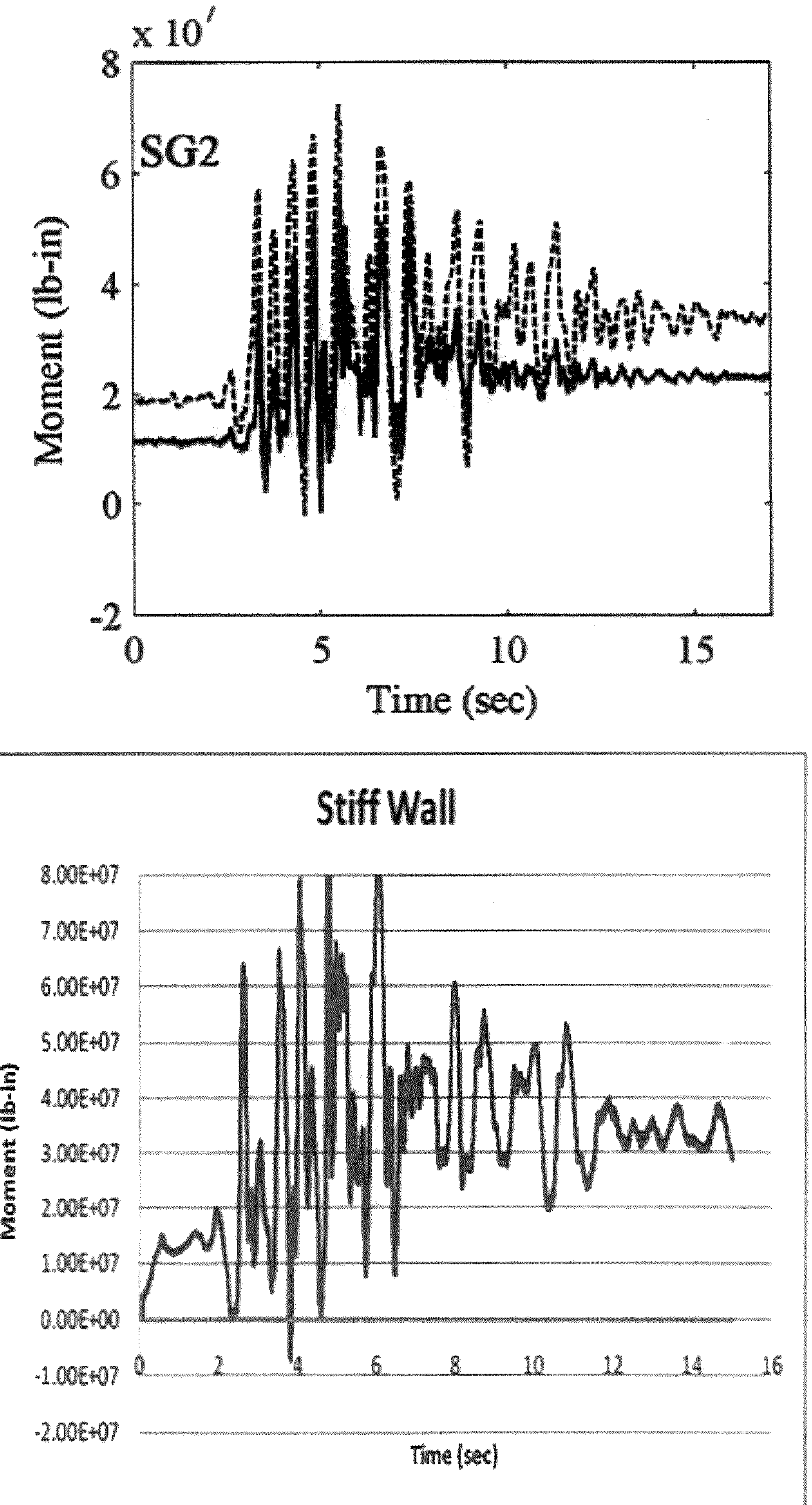
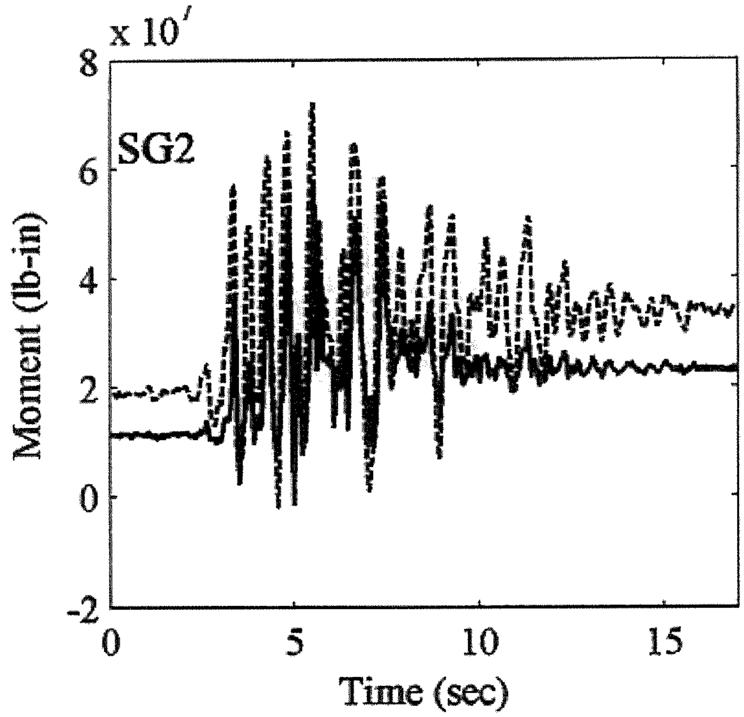


Figure A-3.—Moment time history near bottom stiff wall (Material 16).



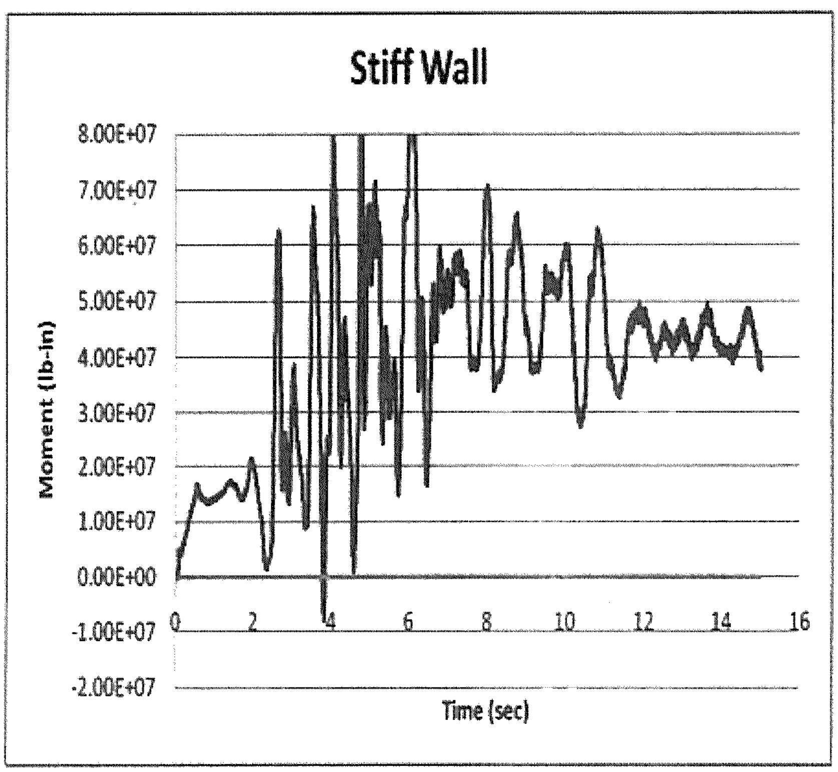


Figure A-4.—Moment time history near bottom of stiff wall (Material 25).