TECHNICAL REPORT SL-79-14

EARTHQUAKE ANALYSIS OF THE MODIFIED GEOMETRY OF THE CONCRETE NONOVERFLOW SECTION RICHARD B. RUSSELL DAM

by

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Earthquake analysis of the modified geometry of the concrete nonoverflow section, Richard B. Russell Dam

Finite element computer analyses were performed for the modified geometry of the nonoverflow section of the Richard B. Russell Dam. Input for these analyses were synthetic earthquake records whose response spectra bounded the project design spectra prescribed by the Nuclear Regulatory Commission (NRC) Guide 1.60 at 0.25-g ground acceleration. Also the effects of variation in the elastic modulus of the concrete were studied using scaled time histories from the Helena and San Fernando earthquakes. A comparison of the results from these analyses is made with the actual earthquake experience at (Continued)
20. ABSTRACT (Continued)

Koyna Dam in India.

Acceleration, velocity, and displacement time histories along with the associated response spectra are presented in Appendix A.
Preface

This study was conducted during the period December 1978-September 1979 by personnel of the U. S. Army Engineer Waterways Experiment Station (WES) under the sponsorship of the U. S. Army Engineer District, Savannah.

This work was conducted under the supervision of Messrs. Bryant Mather, Acting Chief, Structures Laboratory (SL), W. J. Flathau, Assistant Chief, SL, and J. T. Ballard, Chief, Structural Mechanics Division (SMD), SL. Various phases of the analyses were performed by Messrs. William L. Boyt, Automatic Data Processing Center, and Harry E. Stone and C. Dean Norman, SMD. This report was prepared by Mr. Norman.

The Commanders and Directors of WES during the conduct of this study and the preparation of this report were COL J. L. Cannon, CE, and COL Nelson P. Conover, CE. Technical Director was Mr. F. R. Brown.
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U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

<table>
<thead>
<tr>
<th>Multiply</th>
<th>By</th>
<th>To Obtain</th>
</tr>
</thead>
<tbody>
<tr>
<td>feet</td>
<td>0.3048</td>
<td>metres</td>
</tr>
<tr>
<td>pounds (force) per square inch</td>
<td>6.894757</td>
<td>kilopascals</td>
</tr>
<tr>
<td>pounds (mass) per cubic foot</td>
<td>16.01846</td>
<td>kilograms per cubic metre</td>
</tr>
</tbody>
</table>
EARTHQUAKE ANALYSIS OF THE MODIFIED GEOMETRY
OF THE CONCRETE NONOVERFLOW SECTION

RICHARD B. RUSSELL DAM

Background

1. The original design for the tallest concrete nonoverflow section of the Richard B. Russell Dam assumed a founding elevation of 325 ft.* ** By mid-November 1978, the final foundation elevation had been established at 310 ft. The additional 15 ft of excavation in the original design was required to obtain foundation material consistent with foundation design requirements. Although the change in height of the tallest nonoverflow section is small (less than 9 percent of the original section height), additional earthquake structural analyses have been conducted to quantify the effect of the change in height on dynamic stresses. Since no lowering of the design foundation elevations of the intake or overflow sections was required, only the tallest nonoverflow section was reanalyzed. The original and modified geometries of the tallest nonoverflow section can be compared in Figure 1.

Input ground motions for analysis

2. The design earthquake for the concrete sections of the Richard B. Russell Dam is defined in Norman and Stone (1979)† as characterized by the Nuclear Regulatory Commission (NRC) 1.60 response spectra with 0.25-g ground acceleration. To effectively incorporate the design earthquake into the finite-element earthquake analysis, synthetic acceleration-time histories were obtained that closely bound the horizontal and vertical design response spectra. Horizontal and vertical components of

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is found on page 3.
** All elevations (el) cited herein are in feet referred to mean sea level.
the synthetic acceleration-time histories are shown in Figure 2. It should be noted that these records exhibit peak accelerations of approximately 0.4 g's and a wide range of frequencies. The response spectra generated by these records are presented in Figures 3 and 4. A damping value of 7 percent of critical was selected for computing the response spectra. This value is consistent with acceptable levels of damping for structures subjected to maximum design earthquake conditions and is accepted allowable damping for nuclear power plant design. The spectra (Figures 3 and 4) for the synthetic records bound the NRC 1.60 design spectra at all frequencies. This implies that the synthetic ground motion is more severe at discrete frequencies and in total than the maximum design earthquake.

Analyses using Helena and San Fernando earthquakes

3. In addition to the analysis using the synthetic records, a second set of finite-element runs was made. The purpose of these analyses was two-fold: first, the two most severe time histories (Helena and San Fernando) used to analyze the original section were used to analyze the modified section. This provided extreme conditions for comparison with analysis results presented in Norman and Stone (1979).* Second, runs were made using the Helena and San Fernando earthquakes with different dam elastic moduli. These runs provided results for evaluation of the effects of variation in elastic modulus on dynamic response. A complete list of the earthquake analyses conducted on the modified geometry is presented in Table 1. The motion-time histories and response spectra for the San Fernando and Helena records are presented in Appendix A.

Finite Element Analyses

4. All earthquake analyses performed on the modified section were done using the finite element code EADHI, which was discussed in Norman

and Stone (1979).* In general, EADHI is a linear two-dimensional finite-element computer code that includes hydrodynamic interaction between the dam and the reservoir. Input required by EADHI consists of horizontal and vertical acceleration-time histories defined at the base of the dam. The finite-element grid used is shown in Figure 5. Four modes of vibration are used in the analysis. Principal stresses in four critical elements identified in Figure 5 as HE, TO, UF, and DF were evaluated at each increment in time during a particular earthquake run to determine time of maximum stress. Then four principal stress contour plots for the entire cross section were made, one corresponding to each critical element, and evaluated at the time at which that element reached its maximum principal stress. Stress contour plots for all earthquake runs are presented in Figures 6-14.

**Discussion of Results**

5. Maximum principal stresses in the previously defined critical elements due to the horizontal and vertical components of the synthetic ground motion are listed below.

<table>
<thead>
<tr>
<th>Element</th>
<th>Maximum Principal Stress, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>HE</td>
<td>546</td>
</tr>
<tr>
<td>TO</td>
<td>361</td>
</tr>
<tr>
<td>UF</td>
<td>320</td>
</tr>
<tr>
<td>DF</td>
<td>356</td>
</tr>
</tbody>
</table>

As discussed in Norman and Stone (1979),* these stresses act for very short durations of time and are localized in relatively small areas of the cross section. Also the high stress at the heel is caused partly by the rigid base boundary condition imposed by the analysis procedure.

6. The extreme conditions imposed by the Helena earthquake resulted in the maximum stresses presented in Figure 9. It is not the intention here to evaluate the design of the modified section based on

the response to the Helena earthquake. These results are presented simply to illustrate that extreme conditions such as low levels of damping (5%) and high concrete modulus \( E_c = 5.0 \times 10^6 \text{ psi} \) still result in a stress state that is less severe than that which Koyna Dam in India (Norman and Stone 1979)* withstood. The maximum stress state occurring in Koyna Dam during the Indian earthquake of 1967 is presented in Figure 15. A significant increase in stresses was observed as the modulus used in the analysis for the Helena earthquake was increased from \( 2.6 \times 10^6 \) psi to \( 5.0 \times 10^6 \) psi. At the HE element this increase was from approximately 250 psi tension for \( E_c = 2.6 \times 10^6 \) psi to 710 psi tension for \( E_c = 5.0 \times 10^6 \) psi. When the San Fernando earthquake was used only a slight variation in stresses was observed for the same variation in \( E_c \). The variation in \( E_c \) used herein is large and can be reasonably expected to bracket the actual modulus of the dam after construction.

Conclusions

7. The synthetic earthquake time histories presented in Figure 2 provide an excellent set of input motions for evaluating the dynamic response of the modified nonoverflow geometry to the project maximum design earthquake. These records exhibit several high acceleration spikes in the range of 0.4 g and a broad range of significant frequency content. Furthermore, the response spectra generated from the synthetic records bound the NRC 1.60 design spectra at all points.

8. It is concluded that the modified geometry of the Richard B. Russell Dam nonoverflow section can be designed to safely withstand the project maximum design earthquake without failure of the dam.

Table 1
Earthquake Analyses Performed for Modified Nonoverflow Section, Richard B. Russell Dam*

<table>
<thead>
<tr>
<th>Earthquake Record</th>
<th>Concrete Modulus, $10^6$, psi</th>
<th>Damping Ratio percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Synthetic</td>
<td>4.0</td>
<td>7</td>
</tr>
<tr>
<td>Helena N90E</td>
<td>2.6</td>
<td>5</td>
</tr>
<tr>
<td>Helena N90E</td>
<td>4.0</td>
<td>5</td>
</tr>
<tr>
<td>Helena N90E</td>
<td>5.0</td>
<td>5</td>
</tr>
<tr>
<td>Helena N90E</td>
<td>2.6</td>
<td>10</td>
</tr>
<tr>
<td>San Fernando N90E</td>
<td>2.6</td>
<td>5</td>
</tr>
<tr>
<td>San Fernando N90E</td>
<td>4.0</td>
<td>5</td>
</tr>
<tr>
<td>San Fernando N90E</td>
<td>5.0</td>
<td>5</td>
</tr>
<tr>
<td>San Fernando N90E</td>
<td>2.6</td>
<td>10</td>
</tr>
</tbody>
</table>

* For more details on these earthquake records see Norman and Stone (1979).
a. Original geometry

b. Modified geometry

Figure 1. Original and modified geometries, Richard B. Russell nonoverflow section
Figure 2. Synthetic horizontal and vertical acceleration-time histories
Figure 3. Comparison of NRC 1.60 design response spectra (0.25-g horizontal ground acceleration) and response spectra for horizontal component of synthetic earthquake, 7 percent damping.
Figure 4. Comparison of NRC 1.60 design response spectra (0.25-g horizontal ground acceleration) and response spectra for vertical component of synthetic earthquake, 7 percent damping
Figure 5. Modified geometry of nonoverflow section, Richard B. Russell Dam. Material properties and natural frequencies for synthetic earthquake runs

\[ \gamma = 145 \text{ pcf} \]
\[ \nu = 0.17 \]
\[ E_c = 4.0 \times 10^6 \text{ psi} \]

<table>
<thead>
<tr>
<th>Mode</th>
<th>( f, \text{ Hz} )</th>
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<tbody>
<tr>
<td>1</td>
<td>7.63</td>
</tr>
<tr>
<td>2</td>
<td>16.49</td>
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<tr>
<td>3</td>
<td>21.89</td>
</tr>
<tr>
<td>4</td>
<td>28.72</td>
</tr>
</tbody>
</table>
Figure 6. Maximum principal stresses due to synthetic earthquake, 7 percent damping, 

$$E_c = 4.0 \times 10^6 \text{ psi}$$

(sheet 1 of 2)
c. HE, 4.95 sec

Figure 6. (sheet 2 of 2)

d. TO, 4.86 sec
Figure 7. Maximum principal stresses due to Helena earthquake, 5 percent damping,

$$E_c = 2.6 \times 10^6 \text{ psi}$$

(sheet 1 of 2)
Figure 7. (sheet 2 of 2)
Figure 8. Maximum principal stresses due to Helena earthquake, 5 percent damping, 
\[ E_c = 4 \times 10^6 \text{ psi} \] (sheet 1 of 2)
Figure 8. (sheet 2 of 2)
Figure 9. Maximum principal stresses due to Helena earthquake, 5 percent damping,
\[ E_c = 5 \times 10^6 \text{ psi} \] (sheet 1 of 2)
Figure 10. Maximum principal stresses due to Helena earthquake, 10 percent damping, 

$$E_c = 2.6 \times 10^6 \text{ psi (sheet 1 of 2)}$$
Figure 10. (sheet 2 of 2)
Figure 11. Maximum principal stresses due to San Fernando earthquake, 5 percent damping, $E_c = 2.6 \times 10^6$ psi (sheet 1 of 2)
Figure 11. (sheet 2 of 2)
Figure 12. Maximum principal stresses due to San Fernando earthquake, 5 percent damping, $E_c = 4 \times 10^6$ psi (sheet 1 of 2)
Figure 12. (sheet 2 of 2)
Figure 13. Maximum principal stresses due to San Fernando earthquake, 5 percent damping, 

\[ E = 5 \times 10^6 \text{ psi} \]  

(sheet 1 of 2)
Figure 13. (sheet 2 of 2)
Figure 14. Maximum principal stresses due to San Fernando earthquake, 10 percent damping, $E_c = 2.6 \times 10^6$ psi (sheet 1 of 2)
Figure 14. (sheet 2 of 2)
Figure 15. Envelope values of maximum principal stresses (maximum tension or minimum compression) for a nonoverflow monolith of Koyna Dam due to Koyna Earthquake. Static and dynamic stresses are combined.
Appendix A: Motion-Time Histories

Acceleration-, velocity-, and displacement-time histories for the Helena and San Fernando earthquakes listed in Table 1 are presented in this appendix. Also included are response spectra at 5 and 10 percent damping for each earthquake. The finite element analyses discussed in this report use the acceleration-time histories as the driving function. Only the horizontal motions are displayed herein. However, the actual vertical time history as measured for each of the listed earthquakes was also used in the analyses.
Figure A.1 San Fernando Earthquake, Hollywood Storage P.E. Lot, CIT D058, N90E component
Figure A.2 Response spectrum, 5 percent damping, San Fernando Earthquake, Hollywood Storage P.E. Lot, CIT D058, N90E component
Figure A.3 Response spectrum, 10 percent damping, San Fernando Earthquake, Hollywood Storage P.E. Lot, CIT D058, N90E component
Figure A.4  Helena, Montana, Earthquake, CIT DO25, N90E component
Figure A.5  Response spectrum, 5 percent damping, Helena, Montana, Earthquake, CIT D025, N90E component
Figure A.6 Response spectrum, 10 percent damping, Helena, Montana, Earthquake, CIT D025, N90E component