EVALUATION OF SEISMIC STABILITY OF CLAYTOR DAM USING LINEAR AND NONLINEAR TIME HISTORY ANALYSES

Yusof Ghanaat[1] and Bernhard J. Rasmussen[2]

ABSTRACT

This paper presents analyses and evaluation methodology used to assess seismic stability of Claytor Dam, a concrete gravity dam located on the New River, three miles upstream of Radford, Virginia. The study was undertaken as part of the Federal Energy regulatory commission (FERC) Part 12 safety inspection report to assess the probable level of damage and potential failure mechanisms that could affect safety of the dam. Three types of analyses including the linear time-history analysis, post-earthquake static analysis, and the nonlinear time-history analysis were conducted to assess stability of the dam. This study showed that the spillway piers and towers will suffer damage under the MCE with a magnitude of M_s 6.8 and a peak ground acceleration of 0.22g, but will remain stable with little impact on safety of the dam and operation of the gates. The results indicate that a nonlinear analysis capable of capturing dominant nonlinear mechanisms can be used effectively to assess stability of concrete dams to avoid unnecessary retrofits.

INTRODUCITON

The Claytor Hydroelectric Project is owned and operated by the Appalachian Power Company, a wholly owned subsidiary of American Electric Power (AEP), Columbus, Ohio. Constructed in 1939 Claytor hydroelectric Dam is a concrete gravity dam crossing the New River in Pulaski County, Virginia. The dam is located about three miles upstream from the city of Radford. The total length of the dam is approximately 1,142 feet, and consists of 10 non-overflow sections, 4 intake sections, 10 spillway sections, and a trash-way section. The maximum height of the dam is 145 feet above the bedrock. The spillway section is 539.5 feet long and is controlled by nine vertical lift slide gates. The gates are lifted and closed by a hoist above each gate supported by reinforced concrete towers that are in turn supported by concrete piers, as shown in Figures 1.

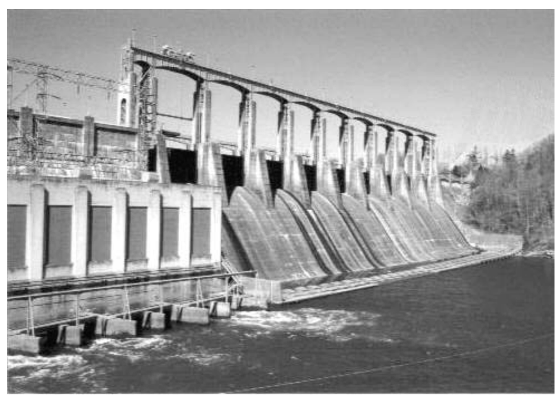


Figure 1: Downstream view of Claytor Dam

A 1999 seismic analysis of the dam had concluded that the reinforced concrete towers supporting the hoist and service bridge would fail under the MCE event and that the failure would be sudden and brittle [1]. A subsequent study was carried out in 2001 to determine what impact the failure of a tower would have on the integrity of spillway gates and thus uncontrolled release of water [2]. The 2001 study found that if a concrete tower were

[1] President; Quest Structures, Inc.; 3 Altarinda Road, Suite 203, Orinda, California 94563
[2] Civil Engineer; American Electric Power; 1 Riverside Plaza, Columbus, OH 43215

to fail, and if any part of the tower, service bridge, steel support frame, or hoisting equipment were to land on the spillway gate(s), it is likely that some or all of the gates could be destroyed or damaged beyond operability and repair. The current study was undertaken to substantiate the previous findings and if necessary to design remediation measures to preclude failure modes that could result in uncontrolled release of water [3].

EVALUATION APPROACH

The approach taken in the current seismic stability assessment of the Claytor Dam was to perform three-dimensional linear and nonlinear time-history analyses by which potential modes of failure can be identified and stability of the spillway towers and piers during and after the earthquake shaking can be assessed. This approach was developed in accordance with the FERC guidelines [4] and approval and was carried out by conducting three types of analyses. The first type consisted of a linear time-history analysis intended to substantiate the previous findings and to identify potential nonlinear mechanisms that could lead to failure. The second included a post-earthquake static stability analysis of the damaged structure to assess operation of the dam after the seismic event. The third analysis was a nonlinear time-history evaluation of the damaged structure to investigate whether or not the spillway piers and towers will remain stable during after-shock events as intense as the main event. The analyses were conducted using the material properties and loadings established in previous studies and geometry data that conformed to the as-built drawings.

EARTHQUAKE GROUND MOTIONS

A maximum credible earthquake with a surface-wave magnitude of M_s 6.8 at a hypo-central distance of 33 km had been estimated previously for the seismic analysis of Claytor Dam [5]. The FERC had approved this earthquake but indicated that the peak ground acceleration (PGA) for the MCE should range between 0.20 to 0.25g and the peak ground velocity (PGV) between 15 to 18 cm/sec. Furthermore, FERC requested that two specific ground acceleration records namely, Temple & Hope from the 1994 Northridge Earthquake and Lake Hughes No. 4 from the 1971 San Fernando Earthquake, scaled respectively by 0.85 and 1.30, to be used as the seismic input for dynamic analysis.

The application of the FERC scale factor of 0.85 to Temple & Hope records produced a PGV of 17 cm/sec with a PGA of 0.15g for the primary horizontal component. The scaled Temple & Hope records, therefore, met the FERC-specified design values for PGV, but not for PGA. For Lake Hughes records, the FERC scale factor of 1.30 resulted in a PGA of 0.22 with a PGV of 11.2 cm/sec, which is less than the specified PGV values of 15 to 18 cm/sec. The scaled Lake Hughes records, therefore, met the FERC design values for PGA, but not for PGV. Time histories of one of the horizontal components of the scaled records with response spectra of both horizontal components are shown in Figure 2. Note that this method of scaling produced time-history records with unusually high spectral peaks near several vibration periods of the structure (0.23, 0.21, 0.15, and 0.12 sec). As a result, the scaled records provided extremely intense shaking for the safety evaluation of Claytor Dam.

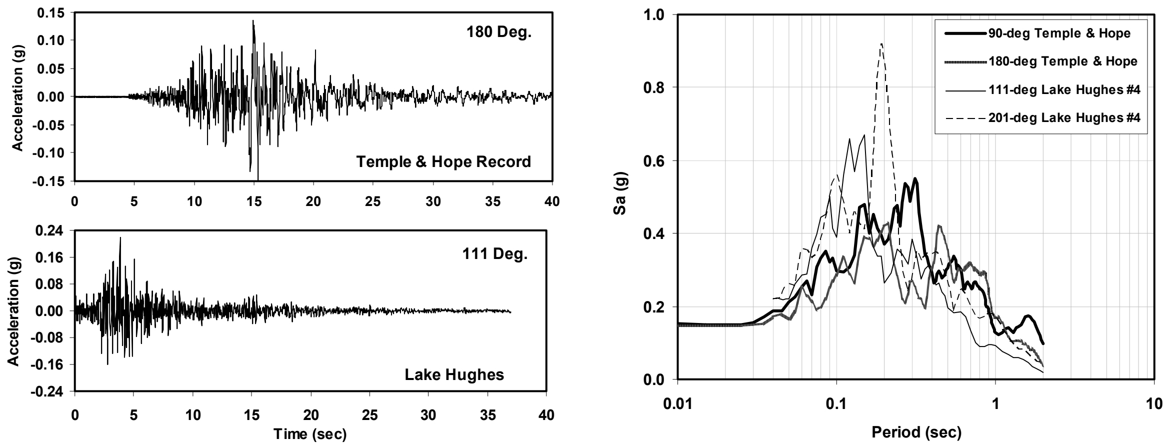


Figure 2. Acceleration time histories and response spectra of seismic input records

FINITE ELEMENT MODELS

The spillway section of Claytor Dam consists of nine overflow monoliths with similar geometry that are expected to respond similarly to static and dynamic loads; each monolith tends to resist loads independently with little support from the neighboring monoliths on either side. Therefore, by providing symmetric and anti-symmetric boundary conditions at the sides of a single monolith, its deflections and stresses can be computed independently. For this purpose, an elaborate 3D model was developed incorporating a single spillway monolith with the associated pier, concrete tower, steel frame support, and the service bridge. The monolith responses to symmetric and anti-symmetric loads were computed separately and then combined to obtain the total response. Symmetric boundary conditions were established by permitting movements only in the upstream/downstream and vertical directions, while anti-symmetric boundary conditions were developed by permitting movements only in the cross-stream direction.

As shown in Figure 3, the spillway monolith with the pier and the reinforced concrete tower were modeled with an assembly of 8-node solid elements, and the steel components of the support frame and service bridge were represented using standard frame and shell elements. The complete model consisted of 10,045 solid elements, 512 frame elements, 114 shell elements, and 12,716 nodal points.

Hydrodynamic effects of the impounded water due to seismic loading were represented by added mass coefficients computed using the Generalized Westergaard Method. The foundation rock was assumed rigid due to its minor effects on dynamic response of the spillway monoliths. Inertial forces of the lift gates due to earthquake excitation were represented by nodal masses distributed uniformly over the gate slot area. Concrete properties were obtained from test results of 19 cores removed from the dam. For analyses, a unit weight of 159 pcf with compressive strength of 7,000 psi was used. Steel properties for the service bridge and supporting frame were based on grade A36 structural steel, while Grade 40 steel properties were used for the concrete tower reinforcing steel.

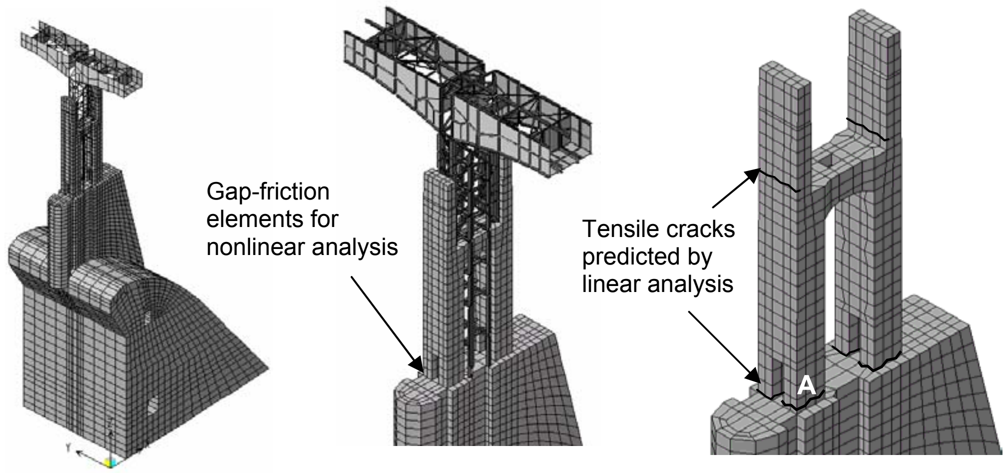


Figure 3. FE Model of Spillway Monolith with Close up Views of Tower and bridge

EVALUATION USING LINEAR TIME-HISTORYANALYSIS

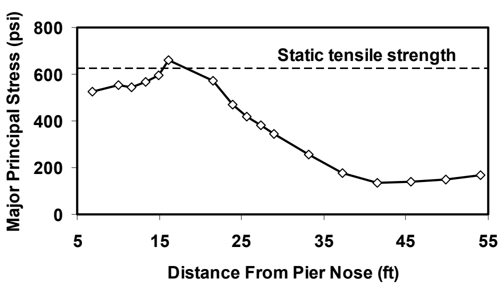
The linear seismic response of Claytor Dam was carried using the SAP2000 program. The time-history modal superposition method was used. Modal properties were computed using Ritz vectors for more efficiency. Superposition of 100 Ritz vectors accounted for more than 99% mass participation in each of the three directions, whereas as many as 300 eigenvectors were needed to achieve the same. Displacements, stresses, section forces and moments were computed separately for the symmetric and anti-symmetric loadings, and then were added to compute the combined effects of seismic loading along all three axes.

Linear Response to Temple & Hope Records

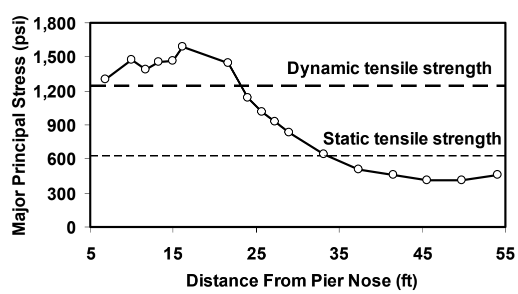
The results of linear-elastic analysis indicate that the spillway piers at Claytor Dam can resist ground shaking produced by the scaled Temple & Hope records with minor or no damage, as indicated by low stresses in Graphs *a* and *b* of Figure 4. However, axial force-bending moment demands at the base of towers exceed the section capacities represented by P-M interaction diagrams, as shown in Graphs *a* and *b* of Figure 5. This suggests that some damage in the form of concrete cracking and steel yielding would occur at the base of towers. The damage, however, is judged to be moderate because the axial force-bending moment pairs mostly remain within the dynamic elastic limit of the plain concrete. In Figure 5, dark and light curves are the factored and nominal P-M diagrams, respectively. While dashed straight lines represent the static elastic limits, and solid straight lines the dynamic elastic limits of the plain concrete. At locations above the base of towers near the cross-beam connections, axial force-bending moment pairs also exceed the section capacities but no point falls outside the static and dynamic elastic limits of the concrete. Thus only minor cracking might occur above the base of towers near the cross-beam connections (Figure 3). For Temple & Hope, the cracking at the base of towers is therefore viewed as local damage with no failure potential, especially when the estimated maximum displacement at the top of towers is only 0.65 inches.

Linear Response to Lake Hughes Records

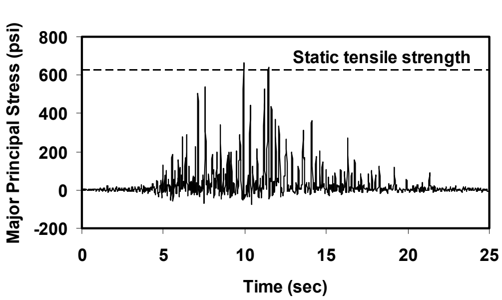
The application of the more intense Lake Hughes records produced more than twice larger displacements, stresses, and section forces and moments than the Temple & Hope records. The results indicate that the concrete towers could fully crack at the base, accompanied with significant yielding of the reinforcing steels. The piers are expected to suffer partial cracking near the upstream end of the base of the pier. The pier cracks will be shallow and would develop mainly over a 20-ft upstream region of the base where tensile stresses exceed tensile strength of the concrete (Fig. 4c). The peak principal tensile stress history in this region indicates 6 stress excursions in the range of 60 to 80 percent of the apparent dynamic tensile strength of the concrete, and only one of them exceeds the strength level (Fig. 4d). The duration of such stress excursions is so short that they are not capable of generating sufficient energy to extend the cracks through the entire base section. This issue was verified by nonlinear analysis and is discussed in the nonlinear evaluation section below. With respect to performance of the towers, the axial force-bending moment pairs at the base of towers exceeded the section capacities as well as the static and dynamic elastic limits of the concrete, as shown in Figs. 5*c* and *d*. This indicates that complete cracking of the tower legs and rupturing of the reinforcing steels would occur. The results also show some minor cracking might occur at locations near the tower cross-beam connections (Fig. 3). However, as discussed later the cracks at upper locations may not occur if the effects of cracks at the base of the tower are included in the analysis. This is because a cracked-base model reduces the seismic force demands and thus stresses in the tower. The maximum tower displacement in this case is 1.4 in.



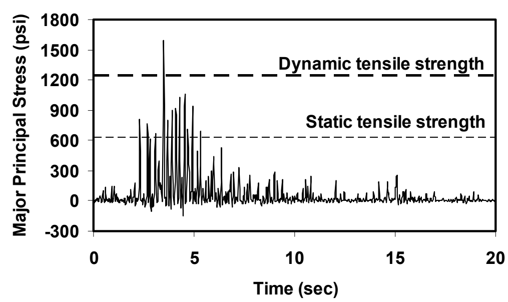
a) Due to Temple & Hope



c) Due to Lake Hughes

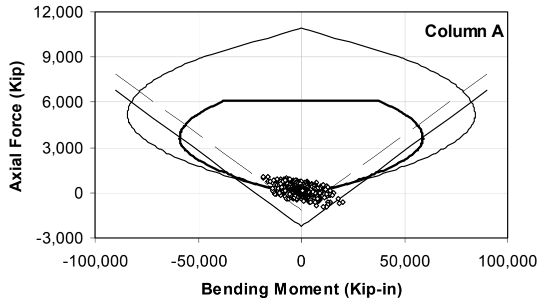


b) Due to Temple & Hope

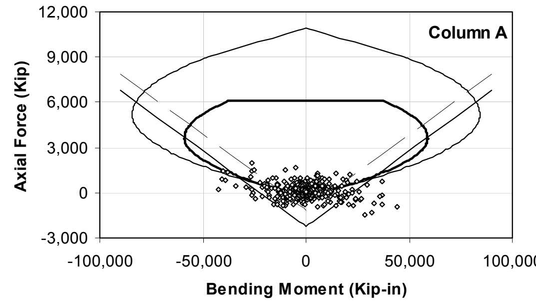


d) Due to Lake Hughes

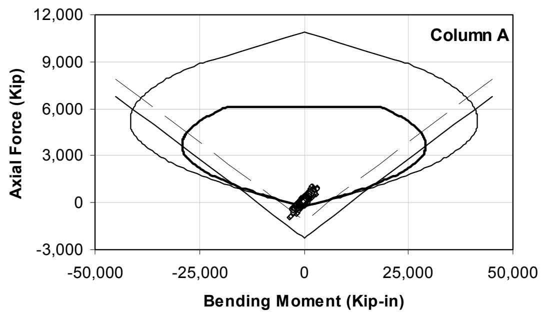
Figure 4. Envelopes of Major Principal Stresses at the Base of Pier with Time History of Peak Principal Stress due to Static Plus Earthquake Loading



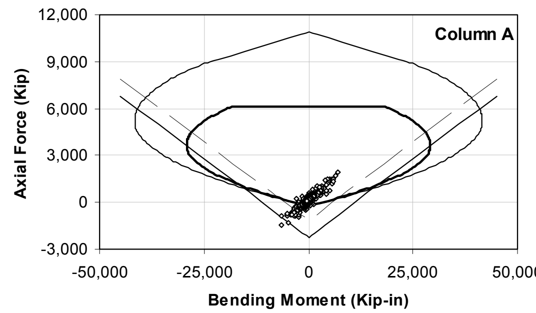
a) In-plane bending (Temple & Hope)



c) In-plane bending (Lake Hughes)



b) Out-of-plane bending (Temple & Hope)



d) Out-of-plane bending (Lake Hughes)

Figure 5. Comparison of Axial Force – Bending Moment Demands with Interaction Diagrams and with Static and Dynamic Elastic Limits at the Base of Tower Column A (see Fig. 3)

Based on the results of linear analyses it was concluded that Claytor Dam would suffer moderate damage but will remain stable when subjected to the MCE ground motions with characteristics similar to the Temple & Hope records. However, severe damage should be expected when the dam is shaken by the MCE ground motions with characteristics similar to the Lake Hughes records. Damage mechanisms appear to be full cracking and separation of the tower at the base with partial cracking of the pier in the upstream region of the base. The stability conditions of the separated tower and partially cracked pier are examined next under static and dynamic conditions.

POST-EARTHQUAKE STATIC ANALYSIS

The post-earthquake static analysis was performed to evaluate static stability of the partially cracked pier in its ultimate state. For this purpose it was assumed that cracks originated at the upstream end of the pier base might propagate along the shortest path through a horizontal construction joint. Accordingly, ultimate state stability analysis of the pier was carried out along the fully cracked construction joint at the spillway crest elevation. Static forces including dead weights, hydrostatic, and uplift pressure corresponding to the normal pool but consistent with the crack, drainage, and water stop conditions were used. The results showed that the cracked pier section, in its ultimate state, if it occurs, has adequate sliding factor of safety equal to 1.91 with a ratio of resisting to overturning moment of 3.95. Based on this calculation the assumed fully cracked pier would remain stable.

NONLINEAR TIME HISTORY EVALUATION

The linear time-history analysis indicated that the dominant nonlinear mechanisms for the spillway-pier-tower system are full cracking and separation of the tower legs at the base of the tower and partial cracking of the piers along the base joint. The nonlinear time-history analyses were performed to investigate whether or not the cracked tower would remain stable during the ground shaking and to what extent tensile cracking of the piers would propagate. To accomplish this, only sliding along the cracked sections, and opening and closing of the cracks at the base of the tower and piers were considered in the analysis, while the remainder of the structure was assumed to respond in a linear fashion. The results for nonlinear analysis included displacement histories and stress contour plots, as well as sliding displacements and the amount of crack opening or separation at the base of the tower. These results were evaluated in light of potential failure modes such as sliding and rotation that might affect stability of the spillway piers and towers.

Nonlinear Finite-Element Model

The nonlinear finite-element model consisted of the pier, concrete tower, steel frame, and the service bridge. The spillway monolith was not included in the model, but the spillway crest motions from the previous linear analysis were used as the seismic input. The cracked-base condition of the tower was modeled by gap-friction elements that resist bearing and shear parallel to the bearing plane but not tension. Thus the tower is permitted to undergo sliding and rocking under earthquake loading. Similarly the predicted cracked zone beneath the pier was also represented by gap-friction elements with no tensile resistance capability. The cracked zone for the pier, as shown in Figure 6, was conservatively assumed to extend over regions having tensile stresses (computed from the linear analysis) in excess of 600 psi; the static and dynamic tensile strengths of the concrete are 622 and 1,280 psi, respectively. The friction forces were taken proportional to bearing forces in accordance with the Coulomb friction law; a friction coefficient of unity was assumed. Note that the nonlinearity was limited to the interface gap-friction elements, while the rest of the structure was assumed to behave linearly. Similar to the linear-elastic analyses, nonlinear analyses were performed separately for two different sets of boundary conditions – symmetric and anti-symmetric.

Figure 6. Assumed cracked area beneath the pier

Evaluation Loads for Nonlinear Analysis

The nonlinear analysis was performed for the gravity and hydrostatic loads plus the seismic loads generated by the Lake Hughes acceleration records. The Lake Hughes records were selected as the seismic input because they indicated more damage due to very high spectral accelerations near several periods of the dam, even though their duration is shorter than that of the Temple & Hope records. Since the spillway monolith was not included in the nonlinear model, the spillway-crest acceleration time histories from the linear analysis were used as the seismic input. As expected, the spillway-crest accelerations showed amplification over the ground-surface records applied at the base of the monolith in the linear analysis. In fact, at 0.5g, the maximum spillway-crest acceleration is more than twice the peak bedrock acceleration at the base of the structure. The nonlinear analysis performed in this study may be viewed as a dynamic analysis of the damaged structure during an after-shock event. Except that the input motions used are too severe because the magnitude of a typical after-shock is usually less than that of the main shock. Nevertheless, if the structure is found to be stable for such an intense motion, it can be concluded that it will also remain stable for the less intense after-shocks.

Evaluation of Results

The results of nonlinear analysis clearly show that the tower undergoes bi-directional sliding and rocking, but the amount of sliding and rocking are minimal with no adverse effects on stability of the tower or operation of the gates. The sliding and vertical (or crack opening) displacements of the tower were obtained from relative displacements of the top of the gap-friction elements with respect to the bottom nodes. For symmetric loading, Figure 7 indicates that the amount of sliding and rocking are very small. The maximum sliding displacements reach 0.002 and 0.004 inches respectively in the upstream-downstream and cross-stream directions and the maximum vertical displacement or crack opening at the base of the tower is 0.04 inches. Figure 7 also shows that the sliding is bi-directional in the upstream-downstream direction and uni-directional along the axis of the dam.

The sliding and vertical displacement time histories for the anti-symmetric (cross-stream excitation) loading are displayed in Figure 8. The maximum sliding displacements reach 0.10 and 0.02 inches in the upstream and cross-stream directions, respectively. Although the sliding displacements are much higher for the anti-symmetric than the symmetric excitation, their magnitudes are still very small to have any adverse effects on the stability of the tower. It appears the tower slides continuously in the downstream direction as it moves left and right with respect to axis of the dam. The maximum vertical displacement or the amount of crack opening at the base of the tower is again 0.04 inches, which is so small to cause rotational instability. The results show that despite very intense seismic input the sliding and rocking are minimal with no adverse effects on stability of the tower. The rotational stability of the tower was also examined in accordance with the US Army Corps of Engineer guidelines [6], and was found to remain stable.

The nonlinear analysis with gap-friction elements resulted in significant stress reductions. The maximum vertical tensile stresses near the base of the tower dropped from more than 2,000 psi for the linear analysis to less than 200 psi. This huge stress reduction is not surprising considering that the analysis was conducted for the cracked-base condition of the tower. Vertical tensile stresses near the beam connections at mid-height of the tower were also dropped from 1,750 psi for linear analysis to about 450 psi. The cracked-based analysis therefore not only relieved tensile stresses at the base of the tower, but also reduced tensile stresses at locations above the base. This indicates that the cracking will be confined to the base and will not propagate to other parts of the tower.

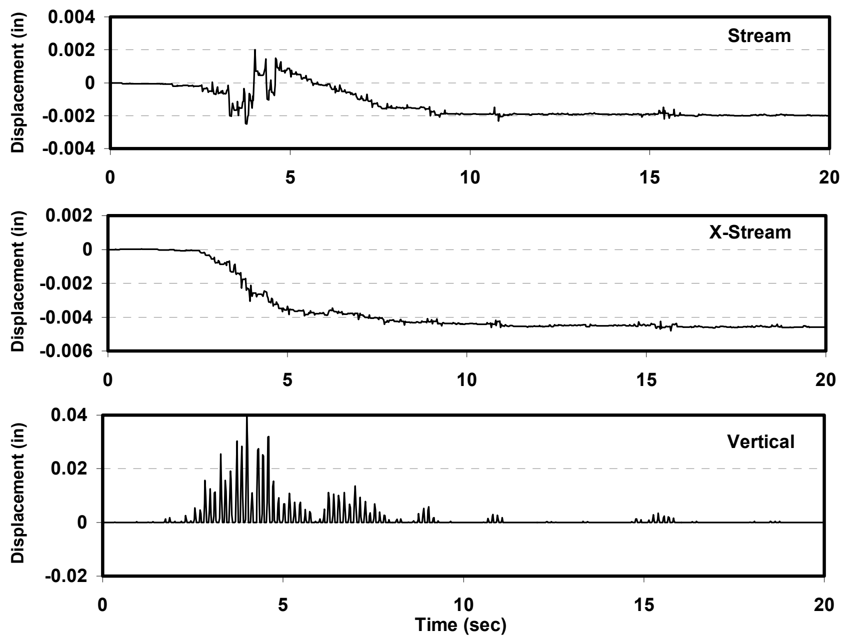


Figure 7. Sliding and crack opening displacement histories of tower for symmetric loading

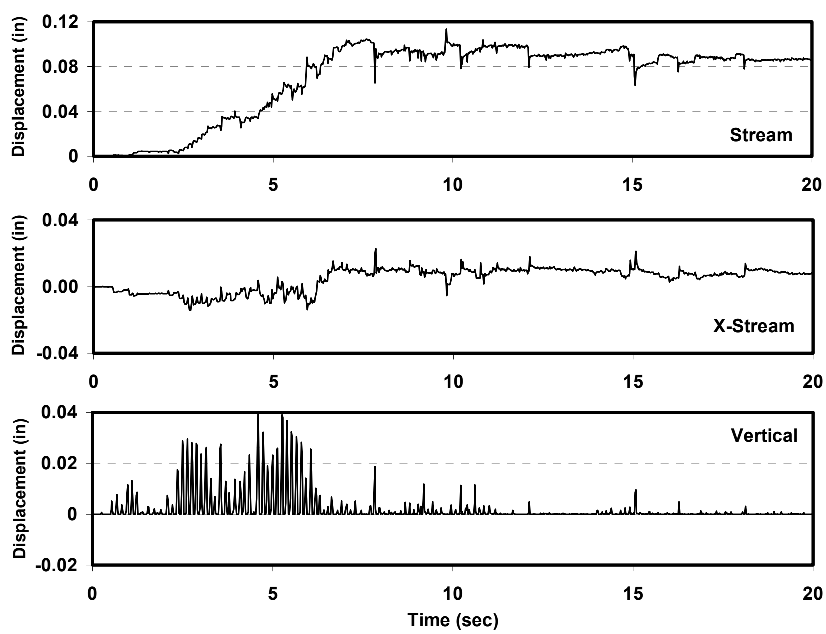


Figure 8. Sliding and vertical displacement time histories of tower for anti-sym. loading

The stress reduction for the pier was also significant. Consequently the assumed cracked region of the pier did not expand either through the thickness or along the length of the pier. As a result, additional nonlinear analyses were not needed to assess further cracking. This finding is also not surprising if the nonlinear interaction between the tower and the pier is considered. For example, as the tower undergoes rocking it can no longer exert tensile forces to the pier, it can only push the pier down. In other words the pier is decoupled from the tower with respect to tension. Consequently, the pier experiences much smaller tensile forces and the cracks do not propagate any further. On this basis it is concluded that in the event of an aftershock the pier would also remain stable and capable of supporting the gates and towers.

CONCLUSIONS

Based on the findings of this study it was concluded that the spillway towers will suffer damage and may undergo sliding and rocking motions, but sliding and rocking would be minimal with little impact on the stability of the towers and operation of the gates. The pier would experience partial cracking but the cracks would not extend to the entire section due to decoupling of the damaged tower from the pier. Therefore, the pier would also remain stable and capable of supporting the gates and towers during and after the postulated MCE ground motions. Furthermore, the steel frame structure and the service bridge were also found to have adequate strength to withstand the MCE event.

This study has demonstrated that a nonlinear analysis capable of capturing dominant nonlinear response mechanisms can be very effective in assessment of the damage and stability condition of a dam. In the case of Claytor Dam, such analysis proved that the dam suffers damage but remains stable, thus prevented unnecessary retrofits.

REFERENCES

1. Schnabel Engineering Associates, "Seismic Analysis of the Concrete Towers atop Claytor Hydroelectric Dam," Report to AEP, June 1999.
2. Kleinschmidt Associates, "Appalachian Power Company, Claytor Hydroelectric Project, Supplement to the Report on Seismic Analyses of the Concrete Dam," May 2001.
3. Quest Structures, "Linear and Nonlinear Finite-Element Analyses of Claytor Dam," Report to AEP, September 2002.
4. Federal Energy Regulatory Commission, "Engineering Guidelines for the Evaluation of Hydropower Projects: Chapters 3 and 11."
5. Schnabel Engineering Associates, "Claytor Hydroelectric Project, FERC 739, Maximum Credible Earthquake Evaluation and Dam Seepage Evaluation," Pulaski County, Virginia. Unpublished report to AEP, 1996.
6. US Army Corps of Engineers, Engineer Manual EM 1110-2-6050, "Response Spectra and Seismic Analysis for Concrete Hydraulic Structures," 30 June 1999.