Wappapello Dam: A Seismic Assessment Case History for Embankment Deformation

by

Gregory L. Hempen[1]
[1] Geophysicist, PhD, PE, RG, St. Louis District, Corps of Engineers, St. Louis, MO 63103, USA

ABSTRACT

Wappapello Dam was designed in the 1930s and completed in 1941. The structure was defensively designed for its time, but its seismic resistance has been questioned because of its proximity to the New Madrid Seismic Zone. Several analyses of the structure's seismic resistance have been undertaken since the late 1970s. More extensive and careful fieldwork and advances in the state-of-the-practice have allowed the seismic risk assessments to develop better understanding of the structure. Original seismic studies were simplistic. The advances in both determining the dynamic properties of the structure and analyzing the potential deformation have improved the risk appraisal.
The fieldwork and risk studies have been completed in distinct phases for this particular embankment dam. Continued, but episodic, progress has resulted in understanding: the foundation feature that could liquefy, dynamic properties of the foundation and embankment, potential ground motions and causative earthquakes, failure modes for the structure, and deformation of the Wappapello embankment on its foundation. The most recent work, yet to be finalized, indicates that the deformation is not significant for the properties of the embankment.

KEYWORDS: deformation; earthquake; embankment dam; ground motion; Wappapello Dam.

1.0 INTRODUCTION

The dam was constructed between 1938 and 1941, for flood-control and other purposes. It has a maximum height of approximately 22 meters (m) with a crest elevation (EL) of 127.9 m (National Geographic Vertical Datum) and a crest width of 9.0 m. The total length of the dam is approximately 825 m. The dam is founded on approximately 38 m of alluvium underlain by dolomite bedrock. The slopes of the dam are protected by rip-rap and a toe drain is installed at the downstream toe.
The structure was designed in the 1930s. The designers recognized the earthquake risk of the area. Several defensive features were incorporated into the design even though analytical procedures had not been developed. The main stabilizing measures were the use of relatively flat embankment slopes of 2.5H:1V, 4.5H:1V and 8H:1V, and the large freeboard of the dam (typically about 18 m) for the normal-operation water level at EL 110 m. The maximum water level reached during the life of the structure is EL 121.6 m (1.3 m above the emergency overflow spillway). The water level has been below EL 113 m during 94% of the life of the structure.
The properties of loose sandy soils over part of the shallow foundation were not modified during construction. Later appreciation of potential earthquake shaking at this site and understanding of the loose sandy soil in part of the foundation initiated seismic assessments of the embankment dam and outlet works. This paper concerns the deformation studies of the embankment dam.

2.0 PRIOR STUDIES

Wappapello Dam was reviewed for seismic stability in the late 1970s by Memphis District. The Waterways Experiment Station [WES, now named Engineering Research and Development Center (ERDC), still the research arm of the Corps of Engineers] aided Memphis District in review of the structure. In the early 1980s the responsibility for Wappapello Dam's operation was transferred to the St. Louis District.

St. Louis District funded and conducted most of the study assessments under the review of the Mississippi Valley Division and Corps of Engineers' Headquarters. The studies built upon the base of fieldwork by Memphis and St. Louis Districts. WES performed some valuable fieldwork that has been utilized to the present.

2.1 WES Appraisal

The WES conducted a seismic safety evaluation of Wappapello Dam at the request of the Memphis District. The results of the investigation are summarized in the report entitled "*One Dimensional Dynamic Analysis of the Liquefaction Potential of the Foundation Materials of Wappapello Dam,"* by Wahl and Deer (1983).

The main conclusion was that liquefaction could be triggered within the Young Point Bar deposit by a large earthquake. Wahl and Deer provided seismic evaluations and crosshole shear-wave velocities of the dam's embankment and foundations.

2.2 Comprehensive Probabilistic Study

The St. Louis District undertook additional investigations. This mid-1980's evaluation utilized additional borings, newly appraised ground motions applicable for the New Madrid Seismic Zone, circular-arc slope failure, realistic pool-release modes, and economic analyses. This comprehensive evaluation attempted to resolve the probabilistic potential of a catastrophic release and of economic loss.

St. Louis District (1988) concluded.

- The sand/silt materials comprising the Young Point Bar deposit are potentially liquefiable in a crescent-shaped zone along a 300-m reach of the dam axis.
- Slope failures are likely if the average residual strength in the Young Point Bar deposit is low. The predicted deformations, however, will likely be minimal as the post-earthquake limit equilibrium factors of safety are just marginally less than 1 and the slopes are very flat.
- The possibility is low that an earthquake-induced embankment slide would cause a pool release. Yet this is not the only, or the most likely, failure mode.
- Realistic inundation analyses indicated that loss of life and great economic damage were a remote possibility.
- The cost/benefit analysis for mitigating the embankment related earthquake-induced pool-release indicates that modification of the dam and foundation is not warranted with the determined information.

2.3 First Deformation Assessment

Dr. W.D. Liam Finn conducted the appraisal of the Wappapello embankment with data provided by St. Louis District and WES. Finn made a field visit with Mississippi Valley Division and WES personnel. This group reviewed the prior actions and recommended future work.

Deformation of the Wappapello embankment and foundation system was appraised by TARA-3 (Finn et al, 1986 & 1989). Finn (1991) provided the initial appraisal. St. Louis District conducted added fieldwork to appraise some critical factors. Finn (1995) made the following conclusions.

- Assuming sliding along well-defined slip surfaces may not always assess the stability of structures within zones of liquefiable materials.
- Under both (500- and 800-year) earthquakes, all of the Young Point Bar deposit liquefied with the exception of a limited area in the lower two-thirds of the deposit beneath the dam. These elements had strengths greater than the residual and provided considerable restraint on both the vertical and horizontal deformations.
- The peak horizontal displacement is about 3.6 m for the conservative EL 119 m pool, and the deformations are negligible with the pool at EL 110 m. The vertical deformations are negligible for all pool levels. The levels of deformation were essentially

independent of whether the input motions were from either the 500- or 800-year return periods, because there was only a slight difference in the extent of liquefaction under the two earthquakes.

- As resolved in the previous investigation by Finn (1991), important factors contributing to the downstream displacements of the dam are the low $(N_1)_{60}$ and residual strength values in the free field beyond the toe of the dam.

2.4 Actions to reduce seismic impacts

St. Louis District conducted other actions to reduce losses and improve the management of seismic risk. The existing Water Control Plan was assessed as optimal. The outlet structure should resist a large earthquake. The Outlet Tunnel was in question until its reinforcement was determined significantly stronger than originally appraised.

The gatehouse internal machinery and emergency gates were secured so that seismic loading would be unlikely to cause a loss. An emergency program was developed to release water from the reservoir due to any adverse impact on the outlet system.

3.0 THE RECENT DEFORMATION ASSESSMENT

3.1 Wappapello Pool Levels

Higher pool levels cause greater driving loads on the dam. Corps of Engineers' policy has established the 10%-Duration Pool as the critical event for seismic design. This pool level is specified as only having been exceeded 10% during the life of the structure (or 90% of the time the pool was below this elevation). For Wappapello the 10%-Duration Pool is EL 112 m.

3.2 Ground Motions For Design Events

Ground motions were developed for Memphis District studies of the early 1980's. St. Louis District resolved earthquake events and ground motions for study events that were reviewed by Dr. Otto Nuttli, St. Louis University. These motions were noted in St. Louis District (1988).

Mr. Don Yule, ERDC, was contracted recently by St. Louis District to provide ground motions for the Wappapello site. Yule has completed the in-house report in cooperation with St. Louis District that provides ground-motion details.

Potential earthquake locations and types of events were considered for the New Madrid Seismic Zone. Both the Maximum Credible Earthquake (MCE) and Operating Basis Earthquake (OBE) were resolved for Wappapello studies. The controlling event and location has been determined as an MCE of 7.8 M_W at 67 km southeast of Wappapello. The causative event may be a reverse fault on the Reel-foot Limb or a strike-slip movement on the East Prairie Limb of the New Madrid Fault. The two faults happen to converge at this minimum distance to the site.

Two records from the Chi-Chi Taiwan Earthquake of 21 September 1999 were appraised for the reverse fault. One record from the 28 June 1992 Landers Earthquake was assessed for the strike-slip event. All three records were spectrally matched to attenuation models for the Central U.S. The Joshua Tree Fire Station record for the Landers Earthquake was the most energetic of the three modified accelerograms. That record was spectrally matched, amplitude scaled, and had a repeated time segment to lengthen the record. This most energetic record for the MCE had: 0.36 gravity, peak ground acceleration; 60 cm/second, peak particle velocity; 41 seconds, bracketed duration; and, 182 cm/second, Arias Intensity. The modified record is provided in Figure 1.

3.3 Deformation of Wappapello's Embankment under present appraisal

Mr. Richard Ledbetter, Ledbetter & Associates, was contracted to conduct the input review and deformation analysis of the Wappapello embankment. Ledbetter had been a contributor to assessments of Wappapello, while a WES employee in the late 1980s and early 1990s. The report remains an in-house draft at this point.

Material properties were retained from the earlier deformation studies. The $(N_1)_{60}$ values were used to determine the maximum-potential-residual - excess-porewater-pressure-potentials (MPP) that can be generated by each MCE and OBE time history. A version of TARA, that is coupled with Seed's updated empirical approach (Youd and Idriss, 2001) and relations for residual excess porewater pressure, was used to calculate the MPP in each element in the Young Point Bar deposit for the design accelerograms. This is then set as the limiting pore-water pressure.

Ledbetter conducted TARA-3 analyses of the Wappapello embankment for varied pool levels, including the 10%-Duration Pool of EL 112 m, and each of the modified MCE and OBE records. The dam's and Young Point Bar deposit's horizontal displacements are estimated to be negligible, between 0.6 to 2. m downstream at the top of the structure, with no settlement (vertical movement). This deformation is insignificant relative to the properties of the embankment.

4.0 CONCLUSIONS

Varied phases of the seismic risk analyses for Wappapello Dam's embankment have been undertaken since the late 1970s. Early phases were more simplistic assessments to gage the potential risk. Later phases incorporated improved, state-of-the-practice information to properly assess the liquefiable feature, the ground motions for the site, realistic failure modes of the foundation-embankment system, dynamic properties of the system, and the deformation of the embankment and foundation. The present analysis seems to indicate that the structure will tolerate vigorous shaking with little deformation.

5.0 REFERENCES

Finn, W.D. Liam (1991). "Analysis of Potential Post-Liquefaction Flow Deformations in Wappapello Dam", PEACS, Vancouver, B.C., Canada.

Finn, W.D. Liam (1995). "Analysis of Potential Post-Liquefaction Flow Deformations in Wappapello Dam Using 1992 Updated Soils Data", PEACS, Vancouver, B.C., Canada.

Finn, W.D. Liam and M. Yogendrakumar (1989). "TARA-3FL: Program for Analysis of Liquefaction Induced Flow Deformations", Department of Civil Engineering, University of British Columbia, Vancouver, B.C., Canada.

Finn, W.D. Liam, M. Yogendrakumar, N. Yoshida, and H. Yoshida (1986). "TARA-3: A Program for Nonlinear Static and Dynamic Effective Stress Analysis", Soil Dynamics Group, University of British Columbia, Vancouver, B.C., Canada.

U.S. Army Engineer District, St. Louis (1988). "Earthquake Study for Wappapello Dam", U.S. Army Engineer District, St. Louis.

Wahl, R.E. and G.W. Deer (1983). "One Dimensional Dynamic Analysis of the Liquefaction Potential of the Foundation Materials of Wappapello Dam", U.S. Army Engineer Waterways Experiment Station, (currently ERDC Vicksburg) Vicksburg, Mississippi.

Youd, T.L. and I.M. Idriss (2001), "Liquefaction Resistance of Soils: Summary Report From The 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils", Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 4, April.

Figure 1. Modified Joshua Tree Fire Station record for the 28 June 1992 Landers Earthquake, the most energetic of the MCE records.

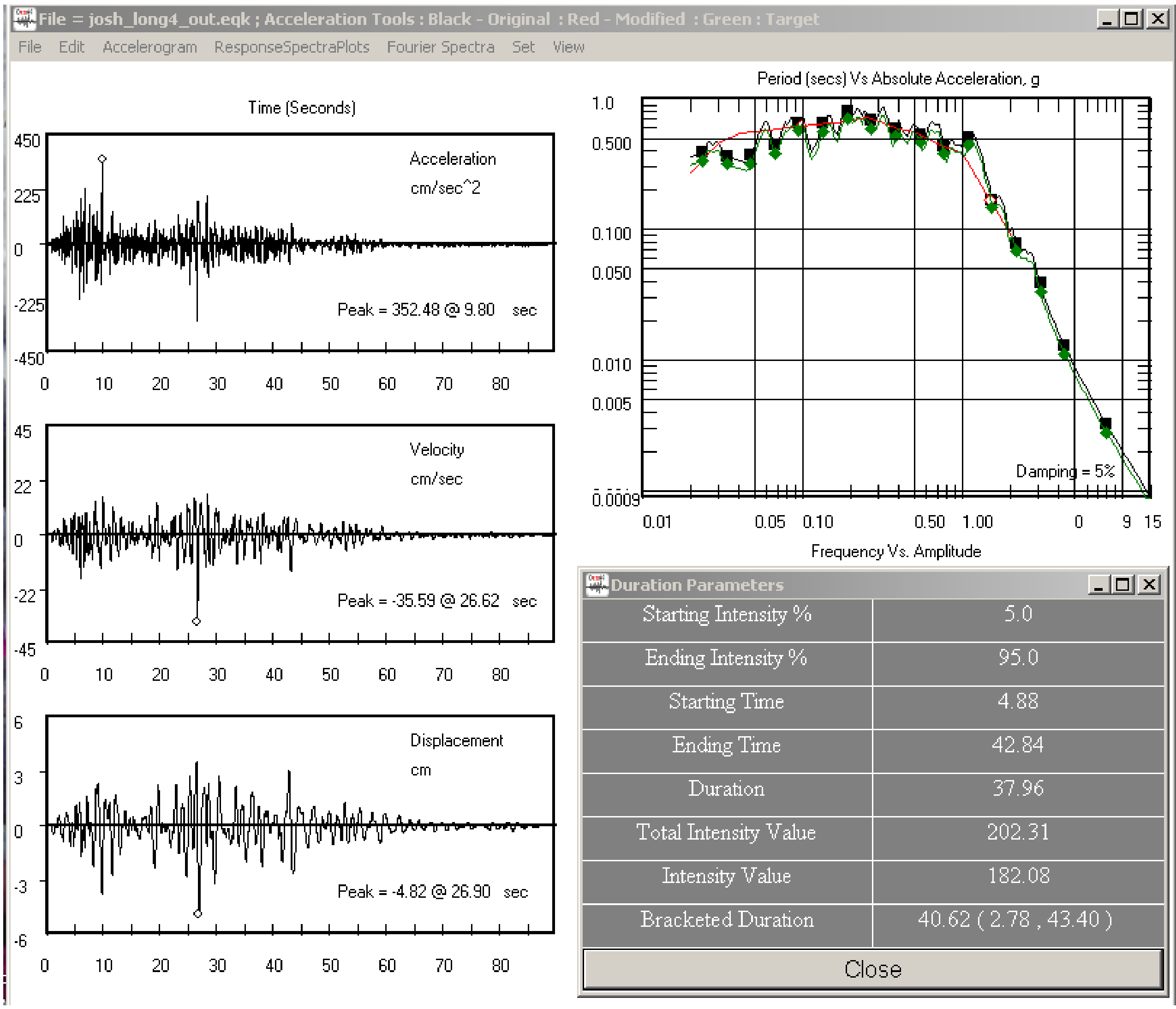

Figure 2. Wappapello deformation from the most energetic MCE record.