Dynamic SSI Analyses of the Headworks Reservoir West Using 2.5D Computational Procedure

Martin B. Hudson[1], Michael Mehrain[2], Alek Harounian[3] ,
Liangcai He[1], Marshall Lew[1], William Lai[4], and Mariam Schahmoradi[4]
[1]Amec Foster Wheeler, Los Angeles, California, United States
[2]Mehrain Naeim International, Inc., Irvine, California, United States
[3]GeoPentech, Irvine, California, United States
[4]Los Angeles Department of Water and Power, Los Angeles, California, United States

ABSTRACT
The Headworks Reservoir West (HRW), a 60-million-gallon reinforced concrete box reservoir, will be constructed as part of the water supply system of the City of Los Angeles Department of Water and Power (LADWP), and performance of the water supply system is crucial in the event of an earthquake. Seismic design of the HRW was performed by LADWP engineers using Equivalent Lateral Force (ELF) procedures. Seismic performance of the HRW was evaluated by performing dynamic nonlinear soil-structure interaction (SSI) analyses using computer program FLAC 2D.

Generally, sites with different soil profiles have different responses to earthquake shaking. The thickness of alluvium underneath the HRW site varies significantly from about 30 feet on the east to over 100 feet on the west. As a result, the reservoir structure may have differing response across the site. To capture the three-dimensional effect of the site response on the structure as well as to model the roof and foundation diaphragm action, a quasi-three dimensional model, herein called a 2.5D model, was developed in which seven two-dimensional grids representing parallel sections through the HRW were linked by beam elements to represent the roof and floor diaphragms of the reservoir. The specifics used in linking the deformation of several two dimensional models to represent the three dimensional performance - including torsion - while using a two-dimensional program, is novel and is the reason for presentation of this paper.

Four design ground motions were introduced to the model and response of key areas of the model were monitored and analyzed. Outputs included acceleration time histories and response spectra at key locations, moments and shear forces in the structural elements, deformation of roof and foundation diaphragm beams, and plastic hinge rotations in locations where plastic response was allowed.

Results of the dynamic 2.5D analyses show that the extent of plastic yielding is expected to be small and that damage to the structure is expected to be minor and insignificant in the event of the design ground motions. Further, the analyses indicated that the structure will have considerable reserve capacity for handling subsequent earthquake events, allowing for continuing water service after the earthquake.

1 PROJECT BACKGROUND/INTRODUCTION

The proposed Silver Lake Complex Replacement Project consists of the Headworks Reservoir Complex (East and West Reservoirs), a Hydropower Plant to be constructed on approximately 12 acres within the Headworks Spreading Grounds, and a series of new water conveyance pipelines to and around the existing Silver Lake Reservoir (not at the location of the proposed reservoir site).

The proposed Headworks Reservoir Complex will consist of two structurally-separate reinforced concrete water storage reservoirs surrounded by earthen embankments, and covered with a soil layer about 3 feet thick. These structures are referred to as the HRE and HRW throughout this paper for the Headworks Reservoir East and Headworks Reservoir West, respectively. The proposed reservoir complex including the planned HRE and HRW are shown in Figure 1. The two structures will have approximately the same capacity and the combined capacity will be 110 million gallons. The footprint of the

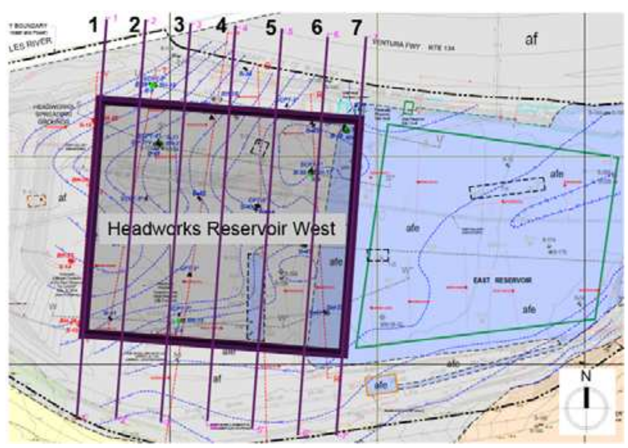

Figure 1. Plan of Headworks Reservoir West Site

combined reservoirs will be trapezoidal-shaped in plan having approximately 1,039 feet and 1,001 feet in length along the south and north ends and about 656 feet and 374 feet in width along the west and east ends, respectively. The height of the reservoirs will be 35 feet. The depth of water in the reservoirs will be approximately 30 feet. This paper addresses the seismic analysis of the HRW. The two-dimensional seismic analysis of the HRE is described in Hudson et al. (2012).

The structure of each of the reservoirs will consist of a mat foundation, perimeter walls, roof slab, and internal columns. The columns will be generally placed on a 30-foot by 30-foot grid. The mat will consist of a 48-inch-thick slab in the bays nearest the exterior walls and will transition to a 36-inch thick slab at a distance of about 32 feet in the North-South direction and 36 feet in the East-West direction measured from the exterior edge of the walls. The 48-inch thick slab is divided into two zones based on the plastic moment capacity. Exterior walls will be 48 inches thick, and the roof slab will be 30 inches thick at the walls and 20 inches thick toward the interior. The columns will be 36 inches in diameter. A cut view through half of the reservoir is shown in Figure 2.

The reservoir is located in a high-seismicity area of southern California and would be subject to high levels of design ground shaking. Seismic design of the reservoir was performed by LADWP engineers using Equivalent Lateral Force (ELF) procedures. After initial design, seismic performance of the reservoir was evaluated by performing dynamic nonlinear soil-structure interaction (SSI) analyses using computer program FLAC 2D (Itasca Consulting Group, Inc., 2011), and results of the SSI were used to make adjustments to the design; the design/analysis sequence is described in more detail in Hudson et al. (2012).

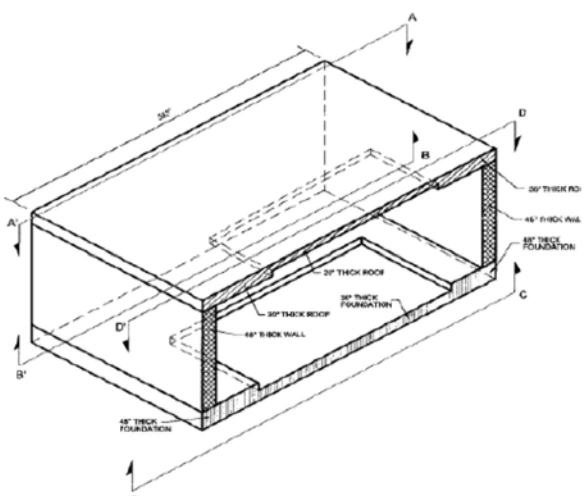

Figure 2. Reservoir Structure

2 MODEL

Generally, sites with different soil profiles have different responses to earthquake shaking. The thickness of alluvium underneath the reservoir site varies significantly from about 30 feet on the east to over 100 feet on the west. As a result, the reservoir structure may have differing response across the site.

Considering the importance of this structure to proper functioning in the event of a large earthquake, the standard simplified approaches for dynamic modeling of buried structures such as this were not considered appropriate.

For the three-dimensional response to be captured at the site, a standard two-dimensional approach was first considered, but the slope geometry of the bedrock beneath the reservoir was considered to likely complicate the response of the reservoir structure.

A full three-dimensional model was considered, but for a dynamic three-dimensional model, the computational effort and therefore duration for computations was considered to be excessive in this case. In addition, most available three-dimensional computer models available do not adequately model both the shear/bending response of diaphragms along with dynamic soil continuum response.

Therefore, to capture the three-dimensional effect of the site response on the structure, to model the dynamic response of the soil continuum, as well as to model the roof and foundation diaphragm action, a quasi-three dimensional model, herein called a 2.5D model, was developed. In the 2.5D approach utilized, seven two-dimensional grids representing parallel sections through the HRW were linked by beam elements to represent the roof and floor diaphragms of the reservoir. The specifics used in linking the deformation of several two-dimensional models to represent the three dimensional performance - including torsion - while using a 2-D program, is novel.

2.1 Site characteristics

The subsurface materials underneath the site of the West Reservoir consist of new engineered fill over alluvium to depths ranging from about 30 to 100 feet below ground surface. The alluvium at the site consisted of materials ranging from silty sand with gravel, poorly- to well-graded sand with gravel, and sandy gravel. The alluvium is underlain by highly to moderately weathered quartz diorite bedrock.

Amec Foster Wheeler previously performed geotechnical investigations for the site of the proposed HRW and presented the results in reports dated May 14, 2014 and April 9, 2015 (AMEC, 2014, Amec Foster Wheeler, 2015). Amec Foster Wheeler also prepared a report of liquefaction analyses and a report of geotechnical recommendations for the proposed HRW and presented the results in reports dated May 26, 2015 and July 9, 2015 (Amec Foster Wheeler, 2015a, 2015b), respectively. A more detailed description of the site, subsurface, and groundwater conditions can be found in the Supplemental Data Report of Geotechnical Investigation (Amec Foster Wheeler, 2015).

As part of the previous investigations, shear wave velocities were measured within properly compacted test fill (in Exploratory Trench T-4), alluvium, and bedrock materials using downhole suspension logging testing within drilled borings. In addition, measurements were performed using spectral analysis of surface waves (SASW) testing to estimate compression (P-) and shear (S-) wave velocities in alluvium and bedrock material. The shear wave velocity profiles were presented in the AMEC (2012) report. Also, as part of the geotechnical investigation for the proposed HRW, suspension logging was performed in Boring B-40 and results were presented in the AMEC (2014) report. The representative shear wave velocities for each material are presented in Table 1. For the permeable gravel layer, a shear wave velocity of 1,000 feet per second was assigned based on published information (Shear Modulus and Damping Relationships for Gravels, by Rollins, Evans, Diehl and Daily, 1998). The shear wave velocities for the earth materials in the model are presented in Table 1.

Poisson's ratio values for each material type were evaluated using P-wave and S-wave velocities measured in the borings with suspension logging testing as described in the AMEC (2012) report. Poisson's ratio values used in the analysis were 0.30 for the engineered fill materials, 0.34 for the alluvium, gravel, and existing fill, and 0.38 for the bedrock.

AMEC (2012) compiled the laboratory direct shear test data from the 2006 to 2011 investigations and evaluated strength properties of the on-site engineered fill, import engineered fill, alluvium, and bedrock. AMEC (2012) developed best-estimate values (design values) of the strength parameters for use in SSI analyses for the HRE. Brief discussions on the development of the design values are presented below. More detailed discussions are presented in the AMEC (2012) report. The same design values were used in the current SSI analyses for the HRW and are presented in Table 2.

Based on the material types and velocities used, considering one-dimensional and two-dimensional results previously obtained at the site, it was decided to utilize an equivalent-linear approach with Mohr-Coulomb strength characteristics to represent the soil materials, and an elastic approach to model the bedrock material. In order to obtain strain-compatible modulus degradation, the entire model was run iteratively until the shear strain across the continuum was reasonably consistent between iterations. Being a dynamic model, the natural hysteretic damping obtained was present, and in addition, a small Rayleigh damping was added to the model. This model was utilized based on the general acceptance of the method, and allowed a more direct comparison with one- and two-dimensional models using the same properties. The materials were found to exhibit reasonable shear strains for application of an equivalent-linear/mohr-coulomb model based on the results of the computations.

The interface between soil and structural elements was accomplished using interface elements, with the interface elements controlled by friction coefficient appropriate for the material against the foundations/side walls.

2.2 2.5D Model

Generally, sites with different soil profiles have different responses to earthquake shaking. The thickness of alluvium underneath the HRW site varies significantly from about 30 feet on the east to over 100 feet on the west. As a result, the reservoir structure, especially the roof, may have different response across the site. In this study, we use a pseudo-3D analysis approach after Mehrain and Naeim (2003) to capture responses of the roof and other components of the HRW structure. This simplified 2.5D approach involves linking, and simultaneously analyzing, seven 2D cross-section models with a roof diaphragm beam and a foundation diaphragm beam. An overview of the 2.5D model is shown in Figure 3.

Table 1. Representative Shear Wave Velocity for Fill, Gravel, Alluvium, and Bedrock.

Material	Material V_s (feet per second)
Engineered Fill (import-for use for sidewall embankment backfill)	800
Engineered Fill (on-site-placed beneath reservoir)	800
Alluvium	600 to 1,300 between elevations 480 and 430 feet, and 1,300 below elevation 430 feet
Colluvium	1,300
Gravel	1,000 (assumed)
Existing Fill	800 (assumed)
Bedrock	2,500

Table 2. Material Strength Properties.

Material	Cohesion (psf)	Friction Angle (Degrees)
Engineered Fill (import-for use for sidewall embankment backfill)	200	32
Engineered Fill (on-site-placed beneath reservoir)	10	35
Alluvium	10	40
Colluvium	500	25
Gravel	10	45
Existing Fill	100	30
Bedrock	NA[1]	NA[1]

[1] NA= not applicable as bedrock is taken to be an elastic material

The seven 2D models shown in Figure 3 are denoted as Cross Sections 1 through 7 from the bottom up in the analysis plane. For convenience of presentation and application of free-field boundary, the seven models are aligned vertically in the 2.5D simulation, and their elevations are shifted accordingly from actual elevations.

In the 2.5D model, each 2D model simulates a north-south slice of the HRW structure and the underlying geologic materials. The geologic cross section at each slice is shown in Figure 4. Relevant material properties and external loads are scaled based on the width of the slice (i.e., tributary width).

In the cross sections, the in-plane behavior of the roof slab and foundation mat was modeled with horizontal beam elements. The upper right and lower left corner beam nodes in each cross section were defined as roof and foundation master nodes, respectively. The horizontal degree of freedom (X-DOF) of all other roof beam nodes in a cross section was slaved to the X-DOF of the roof master node in the cross section, and the X-DOF of all other foundation beam nodes in a cross section was slaved to the X-DOF of the foundation master node in the cross section.

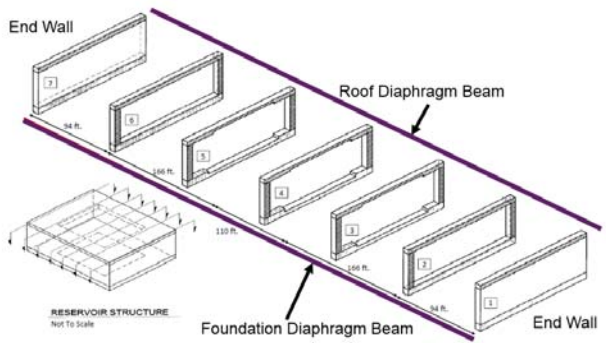

Figure 3. Reservoir Structure Split into Slices

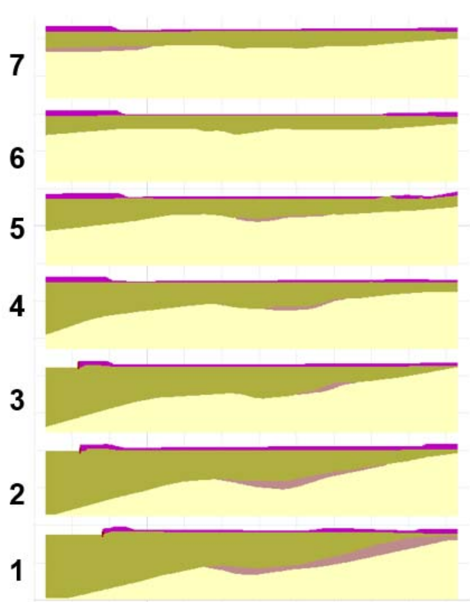

Figure 4. Geologic Cross Sections Used in 2.5D Model

As shown in Figure 5, the vertical beam to the left of the cross sections was used to model the foundation out-of-plane diaphragm action, and the vertical beam to the right of the cross sections was used to model the roof out-of-plane diaphragm action. The X-DOF of each roof diaphragm beam node was slaved to the X-DOF of the corresponding roof master node of that cross section (i.e. the nodes were connected for the x-direction, which is the horizontal direction in the two-dimensional modeling plane shown on Figure 5). Similarly, the X-DOF of each foundation diaphragm beam node was slaved to the X-DOF of the corresponding foundation master node of that cross section.

The 2.5D SSI analysis model included seven unconnected 2D models placed on a common analytical plane, along with two diaphragm beams. Each 2D model simulates a particular cross-section along the HRW structure, and the underlying soils and rock. The diaphragm beams link the seven 2D models. This section describes each component of the 2D models and the diaphragm beams.

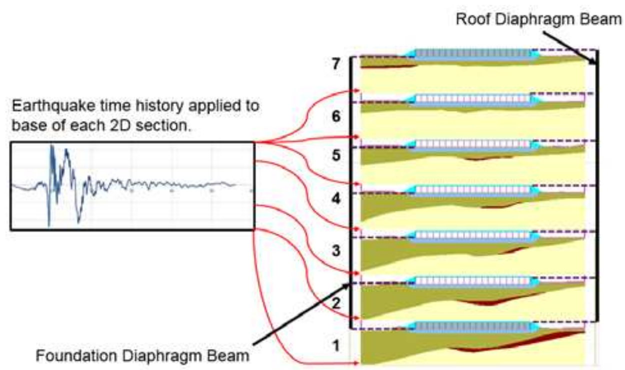

Figure 5. Overview of 2.5D Model

A simple boundary condition was used for static analysis prior to shaking the numerical model. However, the simple boundary conditions applicable to static loading are inappropriate for dynamic analysis. In dynamic analyses, fixed boundary conditions cause reflection of outwardly propagating waves back into the model and do not allow the necessary energy radiation. The use of a larger model can minimize the problem, since material damping will absorb most of the energy of the waves reflected from distant boundaries. However, this solution leads to a large computational burden. The alternative is to use quiet (absorbing) boundaries. Several formulations have been proposed. The viscous boundary developed by Lysmer and Kuhlemeyer (1969) is used in FLAC as a built-in boundary condition model. It is based on the use of independent dashpots in the normal and shear directions at the model boundaries. The method is almost completely effective at absorbing body waves approaching the boundary at angles of incidence greater than 30 degrees. For lower angles of incidence, there is still energy absorption, but it is not as absorbent. However, the scheme has the advantage that it operates in the time domain. Its effectiveness has been demonstrated in both finite-element and finite-difference models. Quiet boundary conditions were applied at the sides and base of the model.

The boundaries at the sides of the model reflect fewer waves if they respond in a similar manner as the free-field motion which would exist in the absence of the structure. An approach to simulate this condition is to "enforce" the free-field motion in such a way that boundaries retain their non-reflecting properties (i.e., outward waves originating from the structure are properly absorbed). This approach was used in the continuum finite-difference code NESSI (Cundall et al., 1980). A technique of this type was developed for FLAC, involving the execution of a one-dimensional free-field calculation in parallel with the main-grid analysis. The lateral boundaries of the main grid are coupled to the free-field grid by viscous dashpots to simulate a quiet boundary, and the unbalanced forces from the free-field grid are applied to the main-grid boundary. In this way, plane waves propagating upward suffer negligible distortion at the boundary because the free field grid supplies conditions that are identical to those in an infinite model. Free field boundary conditions were applied to the side of the model during the seismic shaking. These free-field conditions act independently of one another at the sides of each of the seven cross sections. The free field dashpots were computed utilizing the density and shear wave velocity of the adjacent zones in the finite-difference grid of FLAC.

The free-field boundaries were applied in FLAC to the left and right boundaries using the "apply ffield" command, with the left boundary being the gridpoints with horizontal grid designation i=1 and the right boundary corresponding to the last-encountered non-null zone. Any unconnected sub-grids are not considered when the free-field boundaries are created. In order to use the "apply ffield" command to apply free-field boundary conditions to the 2.5D model, the seven cross sections were aligned vertically in FLAC with the gridpoints on the left boundaries of all cross sections having horizontal grid designation i=1, and the right boundaries of all cross sections at the same, largest, value of i. In addition, the nulled zones vertically separating the 2D cross sections (corresponding to zones designated i=1 and the largest i) were temporarily changed to elastic zones and grouped with a designation of 'Dummy.' These 'Dummy' zones were assigned with very low density and modulus values and then the "apply ffield" command was issued. A schematic representation of the boundary condition applied at the side of the model during dynamic excitation is presented in Figure 6. After application of the side boundary condition in this manner, the 'Dummy' zones were nulled and quiet boundary conditions and a shear-stress history were applied to the base of each of the seven cross sections to excite the 2.5D model.

The model view in the FLAC graphic user interface indicated that the boundary conditions had been applied to all seven cross sections using the method described above. To verify that the free-field boundary conditions in the seven cross sections were indeed correctly modeled, a 2D model for Section 4 only was analyzed using the 1999 Duzce input motion without iteration (see the following section for a description of the ground motions

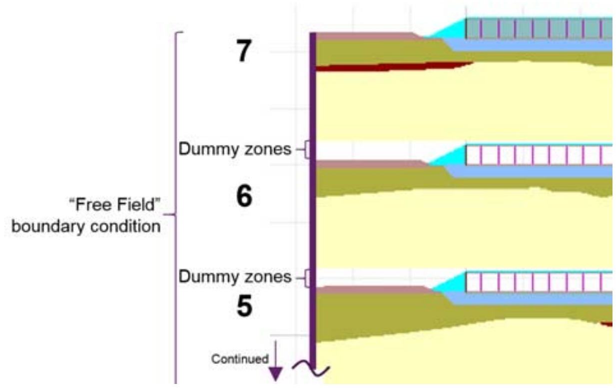

Figure 6. Side Boundary Condition

iteration 1 performed simultaneously on the seven cross sections in the absence of roof and foundation diaphragm beams. The comparison showed that the free-field boundary condition modeling produced the intended side conditions without seismically connecting the seven sections.

2.3 Ground Motions

Amec Foster Wheeler and LADWP previously selected the 84th percentile deterministic ground motion as the design earthquake for the proposed Headworks Reservoir Complex. Amec Foster Wheeler performed a ground motion study and developed rock outcropping motion at the 84th percentile for the site. The ground motion study including deterministic and probabilistic seismic hazard analyses (DSHA and PSHA) to obtain response spectra, and development of ground motion time histories compatible with the response spectra, was presented in MACTEC (2010). Four time histories recorded during past earthquakes were selected and adjusted using the method of Abrahamson (1992) to match the 84th percentile rock outcropping motion for use as the base input motion in the SSI analysis for the HRE (AMEC, 2012). The SSI analyses presented in this report for the HRW used these same ground motion time histories as the base input motion.

The following recorded four time histories were selected to develop base input motion for the SSI analyses:

- 1989 Loma Prieta, UCSC LGPC 000 Component (LGP000)
- 1992 Cape Mendocino, Petrolia 090 Component (PET090)
- 1999 Duzce, Bolu 000 Component (BOL000)
- 1995 Kobe, Kakogawa 090 Component (KAK090)

These recorded time histories were modified by spectrally matching them to the outcropping rock design response spectrum (deterministic 84th percentile). Details of the time histories selection and spectral matching are presented in Mactec (2010). The spectrally matched time histories were used as the input motion at the base of each of the seven cross sections. To save computation time, only a selected duration with the strongest shaking is analyzed for each history. A plot of the adjusted acceleration time history for one of the input motions used as input to the analyses is presented in Figure 7; this input motion is used as the basis for the results shown herein. Note that the portion of the time-history with the most significant shaking was utilized as shown in Figure 7 because the 2.5D model took about one week to obtain results utilizing the multi-core computers available for the computation, and therefore running the portions of the record near the beginning and end of the record were not deemed to be necessary.

Note that the input time histories are for one horizontal component only. Vertical motions were not used in the 2.5D SSI analysis because the vertical motions in the reservoir do not control the shear and moment distributions in the reservoir walls.

To apply ground motion at the base of the model with the presence of quiet boundaries, only the upward-propagating wave motion (one-half of the outcrop motion) was applied at the base of the model (Mejia and Dawson, 2006). This input was applied as a stress history, because the effect of the quiet boundary would be nullified if the input were applied as an acceleration (or velocity) wave.

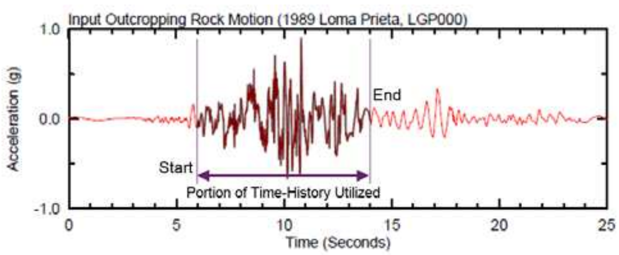

Figure 7. Input Motion – 1989 Loma Prieta Earthquake, LGP000 Record

3 RESULTS

3.1 Output and utilization of results

The 2.5D model with inclusion of roof and foundation diaphragm beams was analyzed using the four input motions discussed above. Many computational outputs can be obtained from the 2.5D modeling. However, for the purpose of this project, only the most important outputs at key areas of the model were utilized in accordance with the needs of the designers. These include acceleration time histories and response spectra at key locations in the middle cross section (Cross Section 4), moment and shear envelopes in the structural elements, deformation of roof and foundation diaphragm beams, and plastic hinge rotation (if any) in locations with allowable plastic hinges.

Various validation excercises were performed to check the functionality of the 2.5D model. These included comparison of free-field motions from one-dimensional models, and running of the 2.5D model with and without the roof diaphragm beam, with the results obtained without a roof diaphragm beam checked against a pure two-dimensional model (Itasca, 2011).

3.2 Results

This study employed the pseudo-3D (2.5D) approach to perform a nonlinear soil-structure interaction (SSI) analysis for the HRW structure to evaluate seismic response of the structure during the design earthquake. Examples of the results obtained using the 2.5D analyses are shown in Figures 8 through 11.

Figure 8 illustrates the shape of the roof diaphragm beam in plan view (representing the deformed shape of the reservoir roof) at various times during the shaking event, including the time with the greatest lateral roof deflection, shown as the fifth shape in the figure..

Figure 9 illustrates the maximum and minimum moment envelope (the maximum and minimum from the entire shaking event) at each point along the roof diaphragm at any time during the shaking. The envelopes are compared to the yield moment of the roof diaphragm, showing that near the center of the reservoir, moment yielding occurred during the shaking event shown.

Figure 10 illustrates the maximum and minimum moment envelope (the maximum and minimum from the entire shaking event) observed at each point vertically along the wall of the reservoir structure. The envelopes are compared to the yield moment of the wall, which indicate that yielding within the wall itself did not occur in the shaking event shown.

Figure 11 shows the plastic hinge rotation that occurs at each of the four corners of the section near the center of the reservoir. This figure shows that mid-way through the shaking, a plastic hinge rotation occurs at the top of the wall-roof joint.

The overall results showed that the maximum moments in the roof diaphragm beam reached the moment capacity of the roof in limited zones within 85 feet of the middle of the reservoir. The maximum shear force in the roof diaphragm beam was computed as being about 142 kips at the ends. Small plastic yielding was found at the upper corners in Cross Sections 3, 4, and 5.

As part of the checking of the performance of the numerical model, the model was run with no interconnecting roof and foundation diaphragm beams. In the absence of the roof and foundation diaphragm beams, responses of Cross Sections 1 and 7 (representing the end walls of the reservoir) are smaller and those of Cross Sections 2 through 6 are larger than those of the 2.5D model with the diaphragm beams, which is an expected result.

Results of the dynamic SSI analyses showed that the extent of plastic yielding is expected to be small and that damage to the roof diaphragm is expected to be minor and insignificant. The analyses results indicated that the proposed structure, which the LADWP engineers indicated had several design elements reflecting conservative assumptions beyond those required by code-based procedures, should experience relatively minor levels of damage (e.g., cracking) in the design event. Further, the analyses results indicated that the structure will have considerable reserve capacity for handling subsequent earthquake events, such that there may be no need to take the reservoir out of service.

4 CONCLUSIONS

The Headworks West Reservoir structure was a very large scale concrete box reservoir with complex three-dimensional geometry of underlying earth materials. Considering the importance of this structure to proper functioning in the event of a large earthquake, the standard simplified approaches for dynamic modeling of the structure were not considered appropriate. Therefore, a 2.5D model was utilized, which represented a novel way to model the three-dimensional response without the additional computation and modeling consequences inherent with three-dimensional models. The results were consistent with those that would be expected with properly-functioning two-dimensional models, as validated in the analyses. Therefore, the reservoir structural demands could be checked, and design modifications made to produce a reservoir adhering to the performance criteria of this important reservoir structure. Limitations of the model used include the lack of ability to handle significantly non-linear soil response, and the inability to handle very complex three-dimensional structural design elements.

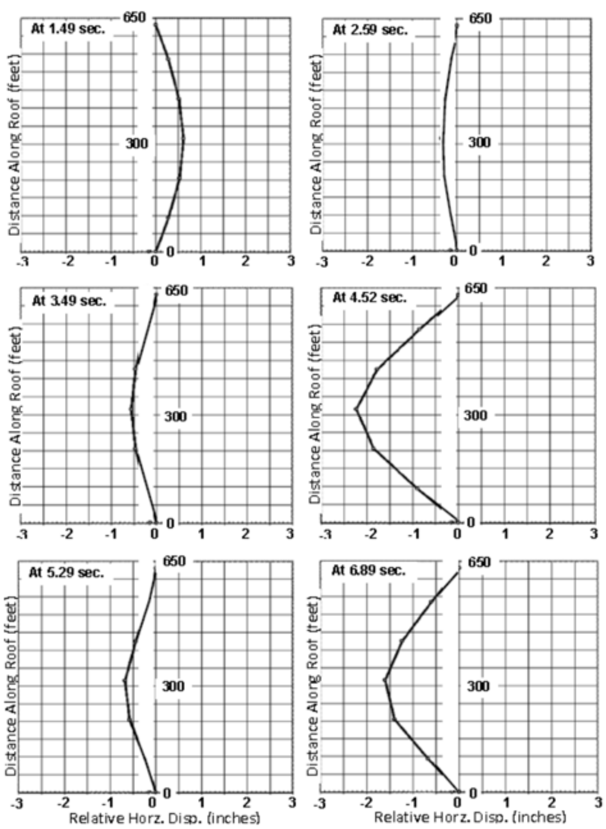

Figure 8. Plan View – Shape of Roof Diaphragm During Shaking

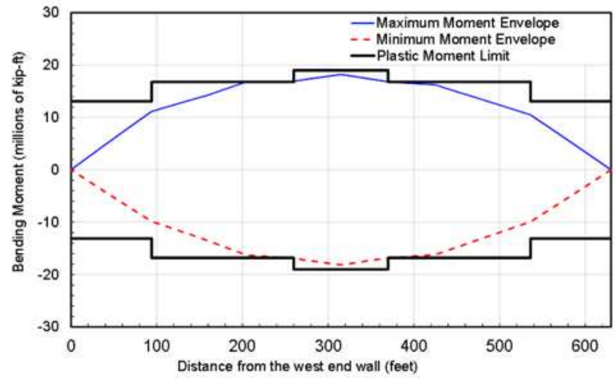

Figure 9. Plan View – Maximum and Minimum Moment Envelopes Along Roof Compared to Plastic Moment Limit

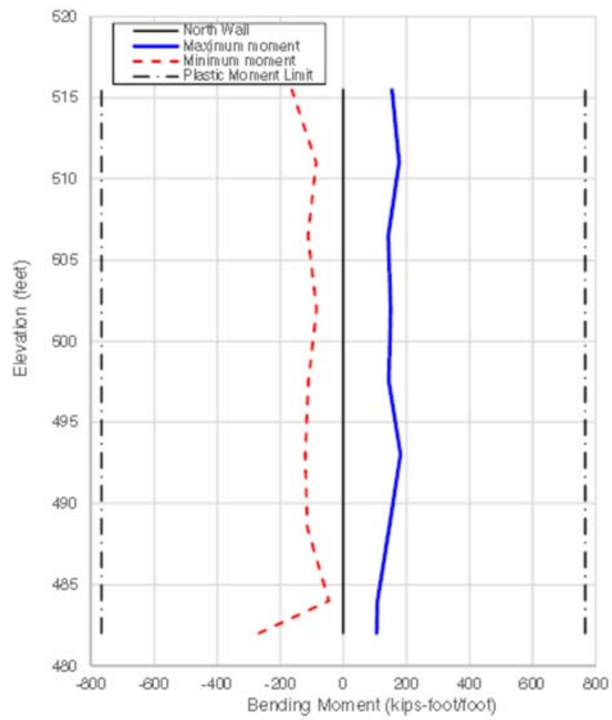

Figure 10. Elevation View – Maximum and Minimum Moment Envelopes Along Wall Compared to Plastic Moment Limit

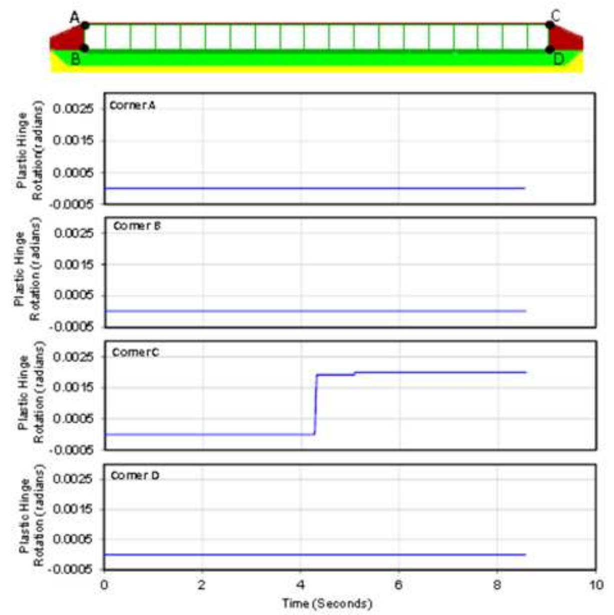

Figure 11. Plastic Hinge Rotation Versus Time at Wall Corners – Cross Section 3 (near Center of Reservoir)

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