THERMAL AND SEISMIC IMPACTS ON THE NORTH RAMP AT YUCCA MOUNTAIN

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ABSTRACT

The impacts of thermal and seismic loads on the stability of the Exploratory Studies Facility North Ramp at Yucca Mountain were assessed using both empirical and analytical approaches. This paper presents the methods and results of the analyses. Thermal loads were first calculated using the computer code STRES3D. This code calculates the conductive heat transfer through a semi-infinite elastic, isotropic, homogeneous solid and the resulting thermally-induced stresses. The calculated thermal loads, combined with simulated earthquake motion, were then modeled using UDEC and DYNA3D, numerical codes with dynamic simulation capabilities. The thermal- and seismic-induced yield zones were post-processed and presented for assessment of damage. Uncoupled bolt stress analysis was also conducted to evaluate the seismic impact on the ground support components.

II. THERMAL LOADS AND EARTHQUAKE CONTROL MOTION

The three-dimensional layout and time-sequential emplacement of the heat-generating high-level nuclear waste in Yucca Mountain was modeled assuming conductive heat transfer through an elastic, isotropic, homogeneous rock mass. The waste was modeled with an areal power density of 100 kW/acre and an assumed out-of-reactor age of 30 years. Waste emplacement was assumed to start in the northern portion of the repository area as shown in the simplified repository isometric drawing in Figure 1. This repository configuration was selected in the ESF Alternative Study.2 Thermal stresses generated along the North Ramp were calculated using the computer code STRES3D. Stresses at different times, up to 300 years after waste emplacement within the preclosure period, were computed and the highest stress at specific ramp locations were selected as thermal loads to be combined with in situ and seismic loads. Figure 2 presents the temperature profile calculated along the alignment of the North Ramp. The thermal loads are insignificant for most of the length of the North Ramp but did contribute to the loading in the lower portion of the ramp near the repository. The thermal loads at three locations studied are listed in detail in Table 1. The locations selected were at the starter tunnel in the TCW at a depth of 150 ft, midway down the ramp in the TSW1 at a depth of 610 ft, and at the base of the ramp in the TSW2 at a depth of 815 ft below the surface.

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Table 1 Calculated Thermal and Seismic Stresses (MPa)

<table>
<thead>
<tr>
<th>Thermo-Mechanical Units</th>
<th>Thermal Stress</th>
<th>Seismic Loads (0.4 g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SIGX</td>
<td>SIGY</td>
</tr>
<tr>
<td>TCW</td>
<td>150</td>
<td>0.36</td>
</tr>
<tr>
<td>TSW1</td>
<td>610</td>
<td>-0.52</td>
</tr>
<tr>
<td>TSW2</td>
<td>815</td>
<td>-1.28</td>
</tr>
</tbody>
</table>

NOTE: X direction as in-plane horizontal direction
Y direction as in-plane vertical direction
Z direction as out-of-plane direction, along axis of ramp
Tension negative

Estimation of quasi-static seismic loads for the North Ramp analysis was based on the ESF Seismic Design Basis adopted by the project. The subsurface repository category of 0.4 g was used to define the earthquake control motion as listed below:

- Maximum ground acceleration: 0.4 g
- Maximum horizontal component of velocity: 40 cm/sec
- Maximum vertical component of velocity: 27 cm/sec

The maximum horizontal velocity was calculated from the maximum acceleration and the empirical relation between the maximum acceleration and maximum horizontal velocity. The empirical relation, based on the recorded seismic data presented in Campbell, Joyner and Fumal, and Mohraz, expressed that the maximum horizontal velocity in cm/sec is 100 times the maximum ground acceleration expressed in g. The maximum vertical velocity was then computed based on the recommendation of Newmark and Hall that the maximum vertical velocity can be taken as 2/3 of the maximum horizontal velocity.

The Working Group Report for the Exploratory Shaft Seismic Design Basis recommends that the performance of the exploratory shaft and underground structures be confirmed using best-estimated conditions when subject to ground motions that are a factor of 1.67 times the proposed control motions. With a factor of 1.67, applied earthquake collapse load becomes:

- Maximum ground acceleration: 0.67 g
- Maximum horizontal component of velocity: 67 cm/sec
- Maximum vertical component of velocity: 45 cm/sec

Both the design base load, 0.4 g, and the collapse load, 0.67 g, were used in the assessment of North Ramp stability. The free-field quasi-static seismic loads, defined as the maximum seismic stresses for wave propagation in an infinite elastic medium, were calculated using the following equations for body wave from Jaeger and Cook:

\[
\sigma = V_p \times E \times (1 - \nu) / [(1 + \nu) (1 - 2\nu) C_p]
\]
\[
\tau = V_s \times G / C_p
\]

where \(\sigma\) and \(\tau\) are the induced free-field stresses, \(V_p\) and \(V_s\) are the particle velocities, \(\nu\) is Poisson’s ratio, \(E\) is Young’s modulus, \(G\) is the shear modulus, \(C_p\) is the propagation velocity of the \(P\) wave, and \(C_s\) is the propagation velocity of the \(S\) wave. These stresses were applied as loads to plane-strain models of the North Ramp opening, with boundary conditions resulting in a free-field principal stress state for \(P\) waves of

\[
\sigma_1 = \sigma
\]
\[
\sigma_2 = \sigma \times v / (1 - v)
\]

oriented with the load application axis; for \(S\) waves, a pure-shear condition results in:

\[
\sigma_1 = \tau
\]
\[
\sigma_2 = -\tau
\]

with the principal axis oriented at 45° to the load axis.

The calculated free-field quasi-static seismic loads at the three locations of interest are listed in Table 1.

III. ANALYSES METHODS

It is anticipated that the quasi-static and dynamic analyses would yield similar results for a deep tunnel with the assumption of infinite surrounding medium. However, for a shallow tunnel with a free surface boundary, the effect of wave reflection from the free surface cannot be assessed using only the quasi-static analysis. Both quasi-static and dynamic analyses were therefore conducted for the cross sections of the ramp using the distinct-element code UDEC with deformable finite-difference zones, and the finite-element program DYNA3D. These two codes were selected for this study for several reasons. First, they both possess a Mohr-Coulomb constitutive model with non-associated flow rule for application of geotechnical material modeling. Second, they both have the capability of solving static problems and using a computed stress state as initial conditions for dynamic analyses. Third, non-reflecting boundary conditions, available in both
codes, allow a very large layer to be approximated by a finite system. Both codes were used for the two-dimensional cross-section modeling of a deep tunnel and their results were compared to ensure the correctness of the analyses.

A Mohr-Coulomb plasticity model with a non-associated flow rule was used for simulating the material constitutive behavior and for predicting the failure area. The rock mass cohesion, angle of internal friction, and dilation angle required as input for the model are presented in Table 2. These properties were obtained from a study that estimated the rock mass mechanical properties at Yucca Mountain by Lin et al. The rock mass surrounding the opening is assumed to be a continuum, i.e., individual rock blocks were not considered.

Table 2. Strength Parameters and Dilation Angles for the Mohr-Coulomb Criterion

<table>
<thead>
<tr>
<th>Thermo-Mechanical Unit</th>
<th>Cohesion (MPa)</th>
<th>Angle of Internal Friction</th>
<th>Dilation Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>TCw</td>
<td>2.7</td>
<td>49</td>
<td>25</td>
</tr>
<tr>
<td>TSw1</td>
<td>0.5</td>
<td>32</td>
<td>16</td>
</tr>
<tr>
<td>TSw2</td>
<td>2.2</td>
<td>49</td>
<td>24</td>
</tr>
</tbody>
</table>

Dynamic analyses were conducted when equilibrium had been reached for excavation with the in situ and thermal loadings applied. Sinusoidal excitation for compressional (P) and shear (S) waves with a range of frequencies was imposed at the lower boundary in the dynamic analyses as the earthquake motion. Three frequencies, 1 Hz, 10 Hz, and 33 Hz, were used to cover the range of excitation of concern from the representative earthquake event. Non-reflecting viscous boundaries were placed on the lateral and upper boundaries to simulate the surrounding infinite medium. The analysis for the P and S waves were conducted separately due to the differences of wave speed and imposed boundary conditions for modeling P and S wave propagation. Mass-proportional damping was used in the dynamic analyses. The fraction of critical damping was set as 5% to simulate the damping level for geological material.

Uncoupled bolt stress analysis was conducted to estimate the bolt stress history during dynamic loading. Uncoupled analysis is a technique used to assess the loads in the rock support components based upon the unsupported deformation projected in the rock. Details for the uncoupled analysis can be found in Hardy and Bauer. Stress and displacement histories at the projected rockbolt locations were recorded for calculation of bolt stress histories. A concrete liner was explicitly modeled for the TCW cross-section analysis.

The boundary conditions for the quasi-static analysis were fully constrained to simulate the surrounding rock mass as an infinite medium. The combined in-situ, thermal, and seismic loads were imposed on the model. Figure 3 shows the UDEC finite difference mesh used in both quasi-static and dynamic analyses.

IV. ANALYSES RESULTS

It was found that the rock would yield in the free field subject to the combination of in situ, thermal, and seismic loads for the TSw1 unit. The numerical analyses, therefore, only considered the cross sections of the TSw2 and TCW units.

The results of UDEC and DYNA3D were compared for both quasi-static and dynamic analyses. Good agreement was found in both cases. Quantitative comparisons of stresses close to the opening indicate that DYNA3D predicts a slightly lower stress level near the excavation face than UDEC. The minor discrepancy can be attributed to the element type used in each analysis. DYNA3D uses single-point integration hexahedral elements, whereas UDEC uses triangular finite difference zone.

The TSw2 results are presented first followed by TCW results. The compressional (P) and shear (S) waves were arc presented sequentially. The rock mass failure zones determined by the Mohr-Coulomb failure criterion for both quasi-static and dynamic analyses results are presented to estimate the degree of damage.

A. Cross Section Analysis for the Ramp in the TSw2 Unit

1. P wave results. The yield zones for quasi-static and dynamic analyses at a frequency of 10 Hz subjected to the 0.67 g load are presented in Figure 4 and 5 respectively. Both analyses show a limited yield zone around the opening. The results for analyses with applied design base load of 0.4 g are similar to those of the collapse load.

2. S wave results. The yield zone for quasi-static analysis with the 0.67 g load is presented in Figure 6. The butterfly-shaped yield zone extends approximately 24 meters from the opening. For the dynamic analysis at a frequency of 10 Hz and subjected to the 0.67 g load, the result presented in Figure 7 shows extensive yield across the modeling region. The result for
dynamic analysis with applied design base load is presented in Figure 8. Without the factor of 1.67, the yield zone is much smaller (approximately 5 meters from the opening).

3. Uncoupled bolt stress analysis. The relative displacement and stresses at the locations of three hypothetical 10 ft point-anchored roofbolts positioned on the center of roof and 45° away from the center were calculated in the S wave dynamic analysis. The bolt stresses were well below the strength of typical grade 60 rockbolt during the complete two cycles of sinusoidal excitation.

B. Cross Section Analysis for the Ramp in the TCw Unit

A horse-shoe shaped tunnel with a concrete liner of 1.5 ft thickness was modeled as the cross section for the ramp in the TCw unit. The top boundary of the model used the stress-free boundary conditions for simulating the free surface. Free-field (non-reflecting) boundaries are imposed on the lateral boundaries to absorb the reflecting wave from the free surface for the dynamic analysis.

1. P wave results. No yield zone developed for the quasi-static analysis with the 0.67 g load. However, the result for the dynamic analysis with applied collapse load, shown in Figure 9, indicates extensive yielding occurs along the wall. The wave reflection from the free surface and the tensile phase of the dynamic loading, which could not be simulated in the quasi-static analysis, are the causes of the difference between the quasi-static and dynamic analyses results. While the dynamic analysis for the design base load was performed, no yield zone was observed.

2. S wave results. Extensive plastic flow was observed for the quasi-static analysis with the 0.67 g load. Due to the extensive failure, no converged result was obtained. The result for the dynamic analysis with the applied 0.67 g load also indicated extensive yielding, with the yield zone expanded from the perimeter of the opening to the lateral and lower boundaries. The dynamic analysis for the design peak ground motion of 0.4 g shows that the yielding extends laterally from the wall to the side boundaries.

3. Concrete liner and uncoupled bolt stress results. Stress histories for several locations in the concrete liner were recorded during the cyclic loading. The peak stress observed was nearly half of the concrete strength of 5000 psi during a complete cycle of sinusoidal excitation. Uncoupled bolt stresses were also calculated from the S wave dynamic analysis with the 0.67 g load. A high tensile stress of approximately 250 MPa was predicted for the roof bolt. The high stress is attributed to the relatively large deformation between the anchor and head of the roof bolt. This level of tensile stress, however, is still within the strength of typical grade 60 rockbolt.

V. CONCLUSION

The results of the analysis, in general, suggest that the collapse load causes extensive failure around the opening whereas the design base load causes only minor damage. The observation agrees with the currently collected empirical data base of seismic damage for underground structures presented in Hardy and Bauer.¹ The predicted extensive yield zones occur most frequently in the shear wave analyses with applied collapse load. The Mohr-Coulomb constitutive model is used mainly to identify the shear failure but does not address the complicated post-failure behavior for rock mass under cyclic loading. In order to better understand the post-failure behavior for the seismic induced yield zone, a more sophisticated model is required.

The dynamic analyses identified deficiencies in application of the simplified quasi-static approach and potential ground support concerns in the openings in the TCw unit where in situ stresses are low. Further review of the potential seismic events and their transmission mechanisms and attenuation through unstrusted, near-surface jointed rock is recommended. Combination of P and S wave motions in the analysis was not considered in this study. Modeling of randomly phased earthquake components with probabilistic combination is recommended for future study.

The ground support system currently designed for the TCw near-surface portion of the tunnel includes 16-ft rockbolts on 1.5 m centers and 6 inches of shotcrete. This rockbolt and shotcrete design should be sufficient to resist fall-out of the rock loosened by a seismic event. The bolt stresses predicted in the uncoupled rockbolt analysis are all below the yield strength of rockbolt steel.
REFERENCES


Figure 1 Isometric View of Potential Repository

Figure 2 Temperature Profile for the North Ramp
Figure 3 Finite Difference Mesh for UDEC Analysis of TSw2 Unit Scale (m)

Figure 4 Yield Zone of P Wave Quasi-Static Analysis with 0.67 g Load for TSw2 Unit

Figure 5 Yield Zone of P Wave Dynamic Analysis with 0.67 g Load for TSw2 Unit
Figure 6 Yield Zone of S Wave Quasi-Static Analysis with 0.67 g Load for TSw2 Unit

Figure 7 Yield Zone of S Wave Dynamic Analysis with 0.67 g Load for TSw2 Unit

LEGEND
- elastic (.)
- at yield surface (+)
- yielded in past (x)
- tensile failure (T)

Figure 8 Yield Zone of S Wave Dynamic Analysis with 0.4 g Load for TSw2 Unit

Figure 9 Yield Zone of P Wave Dynamic Analysis with 0.67 g Load for TCw Unit
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