Title: Finite Element Calculations of the MIGHTY NORTH Event

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The MIGHTY NORTH event was a precision high explosive test performed in jointed Salem limestone for a modeling verification and validation program sponsored by DTRA. The test bed was subjected to a cylindrical shock front, making the response applicable for comparison to 2-D plane strain computations. While other investigators modeled the rock response with various elastic-plastic failure criteria, we demonstrate that simple elastic-perfectly brittle response with a tensile failure criterion replicates the experiment quite well. This paper provides comparisons between results of numerical simulations of the test event and the published test bed response.

INTRODUCTION

Recent work at Los Alamos National Laboratory has included studies of the response of tunnels in hard, brittle rocks to explosive-induced ground shock. In order to have confidence in the results of these studies, modeling attributes need to be validated against relevant test data.

The Defense Threat Reduction Agency (DTRA) (formerly the Defense Science Weapons Agency (DSWA)) sponsored an extensive computational and experimental program to verify and validate constitutive models and codes used for numerical simulation of response of tunnels in rock [1]. The program focused on the response of Salem limestone, an Indiana formation with very consistent, well-characterized material behavior. Studies included theoretical calculations, and numerical simulations of static and dynamic tests of intact rock -- including rock specimens with small-scale tunnels -- and an explosive event, MIGHTY NORTH, on a jointed rock test article with a lined tunnel.

The investigators were concerned with relatively high amplitude loading for this material and focused their studies on ductile response and development of elastic-plastic failure models. However, our prior studies, combined with observations of the MIGHTY NORTH damage and limited review of Salem limestone laboratory test data, suggest that the rock in this test responded in a brittle manner. This is consistent with the observation in Reference 2 that “the primary deformation mechanisms appear to be fracturing of the intact limestone and slip. No rock [plastic behavior] is apparent.” Consequently, we determined that it would be of interest to simulate the experiment using a brittle failure model to represent the Salem limestone. The results of these calculations are reported herein.

PURPOSE OF STUDY

The dynamic response of tunnel complexes to ground shock in deep, hard-rock formations is poorly understood. Current tunnel failure criteria within the ground shock community assume plastic deformation of the tunnel walls under static load conditions to predict tunnel failure. This approach is derived from the semi-theoretical method of Reference 3, and has previously been applied to tunnels in ductile rock. Additionally, finite element calculations of this problem generally rely on the application of various forms of elastic-perfectly plastic failure models as have been developed for studies of dynamic response of geologic media.

However, because brittle-to-ductile transition confinement stress for igneous rocks is of the order 100 MPa or greater [e.g., 4] a brittle failure model would be more appropriate for describing the response of tunnels in these media. While little data exist for validating brittle response, we believe that the MIGHTY NORTH test results
provide a significant opportunity to contribute to the understanding of the response of a lined tunnel in a jointed brittle rock medium.

MIGHTY NORTH EXPERIMENT

The MIGHTY NORTH experiment (Figure 1) consisted of milled limestone bars, stacked around an aluminum-lined “tunnel.” A concrete test bed surrounded the test article. The test article was exercised with an explosively-driven cylindrical shock wave. In addition to active data, precision measurements of individual “brick” dimensions were made both during construction of the test article and during post-test decomposition.

Analysis of active gage data verified uniform response throughout the central portion of the test article along the tunnel axis, confirming two-dimensional response in that section [2, 5]. The data also verified symmetrical response about the vertical plane running along the tunnel axis. Additionally, the measurements and inspection of individual bricks indicated that no plastic deformation occurred in the rock. On the contrary, deformation was fully attributed to development of vertical rock fractures in individual bricks, forming an apparent “chimney,” combined with liner deformation and accompanying slip along the chimney fractures [2]. Figure 2 displays the qualitative description of the brick failures, showing the chimney formed above and below the tunnel. (Vertical displacements were exaggerated by a factor of two for this plot [5].)
Reference 2 also describes the liner deformation. A fit of Fourier coefficients to the liner radial deformation yielded the response mode plot in Figure 3. Test liner deformation was dominated by ovalling (mode 2) with significant breathing (mode 0) contribution. Contributions of other modes were observed to be insignificant.

NUMERICAL SIMULATIONS

We performed a series of calculations to study the MIGHTY NORTH experiment. Four ABAQUS/Explicit [6] finite element calculations will be described.

The basic set-up was similar among the calculations. Despite the symmetry about the vertical tunnel axis, the entire test bed was simulated. We determined that it would be more appropriate to calculate the full model rather than include a more complex symmetry boundary imposed by the imbricate stacking of the individual rock prisms (i.e., one-half of the bricks straddle the center while the other half have an edge on that plane). The liner was modeled by four rows of continuum elements defined in a circular pattern having the 40.6-cm outer diameter and 1.25-cm (1/2-inch) thickness of the actual aluminum tube. The Johnson-Cook strain rate-hardening model with aluminum properties was applied to these elements. The medium surrounding the jointed rock was included to avoid unwanted reflections from boundaries within the simulation time and was assumed to have similar material properties as the limestone (described below) but without joints.

The 2.13-m by 2.13-m rock cross-section was modeled by continuum elements. The exact meshing of the rock depended upon the details of the simulation. The four simulations presented here include:

1. model the entire rock portion as intact material,
2. model each brick as having full planar contact with each of its neighboring bricks,
3. model with prescribed gaps separating individual bricks from their neighbors, and
4. same as case 2, but substituting a reinforced concrete liner for the aluminum liner.

Case 1 provides a baseline case and contributes to the validation of the model relative to the test response as will be shown later. Cases 2 and 3 are both performed as they are somewhat bounding variations of the actual joint conditions where the joint surfaces are not perfectly planar and thus do not stack in a perfectly “square” fashion. The gaps are derived from the precision measurements made during construction, which showed that the final test article section was greater, both horizontally and vertically, than the sum of the individual bricks (nominally 50.71-mm square in section). The measurements suggest that an average gap of 0.163 mm existed between rows of bricks, while average side-by-side gaps were estimated to be 0.188 mm. Case 4 was performed to estimate the response of a more realistic liner condition.

The mesh (Figure 4) was designed so that each square brick consisted of a 4-element by 4-element mesh while each longer end brick, in alternating rows, included 4-element by 6-element meshes of elements the same-size as in the square bricks. Meshes in odd-shaped bricks around the tunnel were meshed individually to fill the available area and maintain reasonable aspect ratios. For options 2 and 3, each brick’s mesh perimeter was defined by contact surfaces to interact with adjacent bricks. In case 2, starting node locations for adjacent surfaces were identical, thus describing full planar contact, while in case 3 the surfaces were separated by the gap widths defined above. The case 1 mesh had identical element sizes to case 2, but was modeled as one contiguous mesh throughout the rock area with no contact surfaces. Case 4 was similar to case 2 except for the liner material model.

The rock was modeled with the brittle failure model described in Reference 6. The brittle failure model was selected based on the following considerations. The Salem limestone brittle-ductile transition confining pressure is 34 MPa [1]. The design loading level for MIGHTY NORTH was 100 MPa at the top of the test article. Other investigators assumed that the confining stress provided by the loading wave would cause the material to respond in ductile failure and selected various forms of elastic-plastic Mohr-Coulomb type failure criteria to model the rock response [1]. However, review of laboratory data (Figure 5) indicates that even at a lower confinement of 50 MPa, a loading of 100 MPa would still be in the elastic response range of Salem limestone. Our prior calculations indicate that when a free field stress environment is within the medium’s elastic range, the environment about the tunnel, particularly the development of hoop tension, appears to dominate local response. Consequently, we assumed that the elastic-brittle response with tensile failure should allow a good estimate of the rock response.
Figure 4. MIGHTY NORTH test article mesh.

- a) mesh.
- b) mesh detail.
- c) contact surfaces.
The brittle cracking option of the ABAQUS model defines Mode I cracking parameters including definition of a tension failure limit for initiation of cracking and a tension-softening curve to define the total cracking strain at which all tensile strength is lost. The brittle shear option defines Mode II cracking, and is a shear-softening model activated after the initiation of Mode I fracture. Prior to failure, the brittle model responds elastically in compression and in tension.

No direct data were available for the parameters of this numerical model, and so rules of thumb were applied to the existing data. It is generally considered that tension strength in brittle materials is 7% to 10% of unconfined compressive strength [6], so the tensile strength of the intact rock case was estimated to be 10% of the unconfined compressive strength (average of 53 MPa) published in Reference 2. However, specimens with small aspect ratios (e.g., square as in this test) are shown in laboratory tests to have higher strength by factors of 50% or more and so the tensile strength for the bricks was estimated to be 15% of the published unconfined compressive strength. Full tensile strength loss was based on the recommendation of Reference 6 which prescribes that data suggest full tension strength to be lost at a crack strain of approximately ten times the strain at crack initiation ($\varepsilon_{0,ck}$), with $\varepsilon_{0,ck}$ estimated from the tensile strength and the elastic modulus, 30 GPa, published in Reference 2.

The mesh was driven by a velocity boundary on the uppermost nodes, including both the test article and the surrounding medium. The boundary histories were determined from velocity history “3V” (Figure 6), derived from accelerometer AV3 at the top center of the test article. Distance to each individual node from the center of the loading wave (defined in Figure 1) was applied to determine nodal amplitude (using 1/r attenuation) and arrival times.

Contact interactions were modeled alternately with Coulomb friction coefficients of 0. (i.e., frictionless) and 0.2.
COMPUTATION RESULTS

The results of various calculations are presented in the following paragraphs. The discussion is generally qualitative in nature, providing figures allowing for comparison of the damage pattern and extent between the model and the calculation. The calculated damage is presented in terms of cracking strain. On the accompanying grayscale plots, white is no cracking while black is full loss of tensile strength (i.e., fully cracked). Liner deformation is also addressed.

Figure 7 provides the computed cracking pattern assuming a test bed with gaps between bricks and no contact surface friction. This computation reproduced the test article’s vertical chimney cracks above and below the tunnel springlines. However, the extent of these cracks varies from the test in that they extend to a greater depth below the tunnel but to a lesser extent above the tunnel. Further, the calculation contrasts from the test by predicting low-strain horizontal cracks within the “chimney” area.

Computed cracking damage for the intact case is illustrated in Figure 8a. This case estimated the chimney cracks to occur as well. However, additional damage occurs in triangular regions both above and below the opening. The lines superimposed onto the contour plot qualitatively define the full region of damage for the section. That is, all damage occurs between the lines above the crown and between the lines below the invert with no damage estimated for the areas to the sides of the springlines between these lines.

These lines were reproduced, and in Figure 8b were superimposed onto a reproduction of the test article damage shown earlier in Figure 2. Figure 8b reveals that the areas encompassing all damage in the intact case also encompass all damage in the test article, including fractured bricks outside the chimney area. We feel that this lends to the validation of the model because while prior authors [1, 2] limited their discussions to the chimney damage, it is apparent that the “off-chimney” cracking is also a part of the opening-caused rock damage response.
The next comparison with the test data involves a second calculation with gaps between bricks. However, for this calculation the brick contact surfaces included Coulomb friction. The calculated damage pattern for this case (Figure 9) is nearly identical to the test article damage seen earlier. The calculation predicts well-defined vertical chimney cracks extending a similar distance both above and below the tunnel. Crack damage is nearly non-existent within the chimney above the tunnel, while some limited cracking is evident within the chimney below the tunnel. Additionally, there is a limited amount of cracking evident within the triangular regions above and below the lined hole. (Crack strains in these regions were determined to be relatively low; brick outlines were not included in Figure 8 so as to render these low-strain cracks more readily apparent in the black and white format of this publication.)

Figure 9. Calculated crack pattern for gaps between bricks with friction on contacts.

Figure 10 displays a deformed mesh plot for this last calculation; as in Reference 5 displacements are exaggerated by a factor of two. Recall that the test article was described as having deformed through slippage along the chimney fractures and deformation of the liner with no observed plastic deformation of the bricks. The continuum nature of the individual bricks does not allow explicit fracturing, thus slippage, along the crack lines. However, in the calculation the bricks within the chimney undergo a general downward displacement with the mesh outside the chimney left undistorted. Considering the limitations of the continua, this mesh response is a reasonable match to the test article.

Figure 10. Deformed mesh for gaps between bricks with no friction on contacts.

Figure 11 provides the results of a modal analysis of the computed liner deformation as was provided for the test article in Figure 3. As with the test liner, the simulated modal response was dominated by modes 2 and 0. However, the modal amplitudes are higher than those shown in Figure 3, suggesting greater distortion. Absolute diameter changes confirm this: crown/invert diameter reduction of 11.4 mm in the test vs. 21.4 mm in the computation; springline diameter increase of 3.5 mm in the test vs. 5.8 mm in the computation. Further, there is greater contribution of higher mode response in the calculation, such as is evidenced by the flattening of the liner at the crown seen in Figure 10.

Next, we present the case with bricks in full planar contact with Coulomb friction on the contact surfaces. Figure 12 shows the damage pattern in this calculation. The crack damage is limited to the well-
defined vertical chimney cracks, accompanied by a small amount of hoop cracking within the chimney area. The gapped model is definitely more appropriate for estimating rock damage around the tunnel than is this case. However, inspection of the Fourier modal analysis of the liner deformation for this case (Figure 13) and comparison to the test liner deformation (Figure 3) reveals that they are nearly identical in magnitude and relative contribution from the various modes. Moreover, computed diameter changes are nearly identical: 12.9 mm for the crown/invert diameter (compared to 11.4 mm) and 3.5 mm for the springline diameter (identical to the previously reported value).

Finally, a calculation was accomplished substituting a simulated reinforced concrete liner for the actual test aluminum liner. A brittle concrete liner with reinforcing was modeled using standard ABAQUS models. All other aspects of the calculation were identical to Case 2 defined earlier; that is, bricks in full planar contact with Coulomb friction.

Figure 14 illustrates that the concrete liner in this simulation fails, and a significantly different response of the tunnel and surrounding rock occurs. While the continuum model is not adequate to fully address this large-displacement response, the simulation results suggest important response characteristics. First, the potential for liner collapse could at the least create a condition of “functional” failure within the tunnel regardless of rock response. Second, the simulation predicts that full rock blocks (bricks) could encroach into the tunnel not only from collapse of the crown but also from upheaval at the invert. (This is confirmed by the large-displacement discrete element calculations presented in a companion paper of this conference [7].) Finally, loss of confinement at the tunnel interface due to liner collapse promotes much more extensive rock damage.

**CONCLUSIONS**

Results from a series of calculations were presented and compared to damage response measured in the MIGHTY NORTH test. The calculations assumed a brittle failure model for Salem limestone under the test conditions. Various representations were made of the “jointed” rock test article and liner.
Simulation of the experiment assuming gaps between adjacent bricks did an excellent job of replicating cracking damage patterns in the bricks but overestimated liner deformation. An alternate case assuming full planar contact between bricks underestimated the extent of rock damage but did an excellent job of replicating the liner response. The authors propose that a more realistic calculation with random contacts between bricks to better model the apparent effect of asperities and non-planarity of brick surfaces may provide a better match to all aspects of the experiment. Nevertheless, we believe that the results presented herein are better matches to the test than those provided by elastic-plastic model computations described in References 1 and 8, and that the study provides encouragement toward the validation of the brittle failure model for simulating response of tunnels sited in brittle rock materials.

The joints greatly reduce the extent of rock damage relative to the intact case. Slippage along the joints appears to dissipate hoop stresses, and damage occurs in the form of vertical chimney cracks concentrated above and below the springline. Further, different joint details cause differences in rock and tunnel response and damage.

Additionally, liner character appears to be an important factor in tunnel response. The ductile liner used in MIGHTY NORTH was a necessary choice to provide data for code validation and verification. Having thus verified the brittle material model, a computation including a brittle concrete liner was performed. For the liner modeled, the applied loading caused this simulated liner to buckle. The calculation resulted in a prediction of significantly greater damage extent than was estimated for an otherwise identical computation that included the ductile, non-failed aluminum liner. Block intrusion into the tunnel from both the crown and the invert is predicted.

We believe that the calculations compare very well to the MIGHTY NORTH data. This match validates the use of brittle failure constitutive models in those cases where brittle rock response is thought to govern. We encourage the testing community to provide material properties consistent with the requirements of the brittle constitutive model so that more accurate simulations may be performed. Moreover, more precise models of the response of brittle materials to dynamic loading should be developed for conditions where tunnels exist in brittle media.

REFERENCES

8. Goering, K., July 2000, from briefing presented at DTRA, Alexandria, VA.