ABSTRACT: The Citybanan commuter train tunnel will comprise a 6 km long, double-track train tunnel to be constructed in rock under Stockholm City. In this paper, analysis of the intersections between the planned train tunnel and so-called "energy tunnels" is described. The objective of this analysis was twofold: (i) to predict deformations and stability of the energy tunnels caused by excavation of the train tunnel, and (ii) to determine required rock reinforcement for the train tunnel. Analysis was conducted primarily using the threedimensional finite difference program $FLAC^{3D}$. The analysis results showed that only minor deformations can be expected in the energy tunnels due to the excavation of the train tunnel, and that the overall stability of the tunnels was satisfactory. Furthermore, the results could be used to confirm a proposed rock reinforcement scheme for the train tunnels, thus securing the stability of the load-bearing structure for the rock tunnel.
2 Reinforcement design and verification of the load-carrying capacity for the train tunnel.

The analysis work comprised the use of several different methods, including empirical methods, analytical calculations, limit equilibrium methods, and numerical analysis (with and without installed reinforcement). The use of different design methods served to complement each other.

The first task involved analysis of the energy tunnel using numerical modeling (without installed reinforcement). Both the energy tunnel and the train tunnel were included. Analyses were only conducted for a worst-case scenario in terms of deformations, corresponding to finalized tunneling (through the intersection area). Block analysis was used as a complement to assess the likelihood of fall-outs and/or sliding of rock blocks formed by intersecting joints. The above analyses were evaluated with respect to local and global stability, as well as induced deformations in the energy tunnel.

The second task included empirical and analytical design calculations, as well as block analysis, to arrive at a preliminary reinforcement design. Results from numerical modeling without reinforcement...
were also utilized in this process. These initial analyses were followed by numerical modeling with installed reinforcement in order to verify the preliminary design. The numerical analysis was conducted for several excavation stages, corresponding to the assumed excavation scheme for the intersection area. These analyses were evaluated with respect to the load-carrying capacity of the installed reinforcement and the rock in interaction.

Figure 2. Example of geometry of intersection (N1) between existing energy tunnel and the planned Citybanan train tunnel. (Ground surface elevation not to scale.)

2.2 Choice of analysis method

The complex geometry dictated the use of three-dimensional modeling to correctly simulate the rock and tunnel behavior in a numerical model. A continuum approximation was judged acceptable since no large-scale, dominant, geological structures could be identified near any of the tunnel intersections (see also Section 2.3). The possible effect of pre-existing discontinuities in the jointed rock mass (Fig. 3) was included through the use of equivalent rock mass properties, and through supplementary block analysis.

Numerical modeling was conducted using the three-dimensional finite difference program FLAC (Itasca, 2004). For the analysis of the energy tunnel, calculations were carried out for selected parameter combinations in order to identify critical cases, in terms of induced deformations, as well as to determine the sensitivity of the results. For the analysis of the train tunnel, only one set of parameter values was used due to time constraints.

Simplified two-dimensional analyses were conducted using the two-dimensional finite difference code FLAC (Itasca, 2001). The purpose of these was to assess the required model size and study which parameter combinations that appeared to be critical. This served as important input to the three-dimensional modeling.

Block analysis (wedge failures) was conducted using the Unwedge limit equilibrium code (Rockscience, 2004a). Safety factors (against sliding and/or falling) for delineated blocks were calculated. In addition, this program allows assessment of the clamping effects of rock stresses (through increased normal forces on wedge planes thus increasing the factor of safety). Furthermore, the variability of joint properties could be included through a "combination analyzer", in which the program calculates all possible blocks formed by combinations of the given joint orientations.

Figure 3. Example of continuum and discontinuum approximations for rock masses.

2.3 Pre-requisites and input data

Only preliminary site investigations were completed by the time of the analysis. This included geological and rock mechanics mapping of nearby underground excavations, supplementary mapping of the energy tunnels, and data from a few core drilled boreholes. No laboratory test results were available in time for inclusion into the input data to the analysis. More comprehensive site investigations are on-going, which will result in a detailed geological and rock mechanics prognosis for the tunnel route. These data will be incorporated into later, and possibly revised, design calculations.

For the present work, existing data were used to establish reasonable, and, as far as possible, representative input data to the analysis. For each of the tunnel intersections, rock mass quality and strength, and dominant joint orientations, were determined. An estimated variability (minimum and maximum values) was assigned for each parameter, including a typical (most likely) value.

The mechanical properties for the rock mass, as input to the numerical modeling, was determined using the empirical Hoek-Brown failure criterion cou-
plied with rock mass characterization using the GSI system (Hoek et al. 2002, Rocscience 2004b). Values on the uniaxial compressive strength was estimated based on experience. Linear regression of the curved Hoek-Brown envelope was conducted to arrive at equivalent values on cohesion and friction angle for the rock mass assuming a Mohr-Coulomb material model. The linear regression analysis of the Hoek-Brown envelope was conducted over a range for the minor principal stress, $\sigma_3$. In this case, the maximum value for the $\sigma_3$-range was determined based on two-dimensional linear-elastic stress analysis of typical cross-sections.

Individual values were thus obtained for the different tunnel intersections. In general, the rock mass is of fair quality with typical GSI-values in the range of 50 to 60. The corresponding rock mass cohesion and friction angles were between 1.9 and 2.8 MPa and 59 and 64°, respectively. The estimated typical values on Young's modulus for the rock mass were in the range of 10 to 18 GPa for all intersections.

In situ (virgin) rock stresses were estimated based on previous overcoring stress measurements in the Stockholm City area. The following stress profiles were used:

$$\sigma_{Hv} = 4.5 + 0.075z \quad (\pm 50\%)$$  
$$\sigma_{hv} = 3.0 + 0.0375z \quad (\pm 50\%)$$  
$$\sigma_{v} = 0.027z \quad (\pm 20\%)$$  

where $\sigma_{Hv}$ and $\sigma_{hv}$ are the maximum and minimum horizontal stress, respectively, $\sigma_{v}$ is the vertical stress (all in MPa), and $z$ is the depth below the ground surface (m). The numbers in the parentheses represent the estimated variability of the stress components. The trend of the maximum horizontal stress was estimated to 160° relative to geographic north (no variability assumed). These stresses were assumed to be valid for all analyzed tunnel intersections.

The analyzed geometry for the different tunnel intersections was taken from existing tunnel sections (energy tunnels), and the planned route and dimensions of the Citybanan train tunnel. The latter was assumed to be fixed—i.e. only one geometrical alternative was analyzed.

3 ANALYSIS OF ENERGY TUNNELS

3.1 Numerical analysis

3.1.1 Model setup
A three-dimensional numerical model was setup for each of the four tunnel intersections using FLAC$^{3D}$. The model size was chosen based on experience, results from the simplified two-dimensional model, and trial-and-error, in order to reduce possible boundary effects. The minimum model size was 120 x 120 x 90 m (width x length x height).

The problem geometry was simplified as follows: (i) all tunnels were straight-lines (no curvature in plan or height), (ii) rock cover was assumed constant over the entire model, (iii) cross-cuts between the train tunnel and the service tunnel, and niches, etc., in the service tunnel, were not included. The models were oriented so that the model axes were parallel or perpendicular to the orientation of the maximum horizontal virgin stress. This facilitated the application of stresses in the model, as no shear stresses were required on the model boundaries. Roller boundary conditions were used on the vertical and bottom boundary of the models, whereas the top boundary (rock surface) was simulated as a free surface. Existing buildings and soil cover were included as external, distributed normal loads acting on the rock surface (top boundary)

The minimum size of the finite difference elements was chosen to 0.5 m, to achieve good resolution close to the tunnel boundaries. The zone size was increased gradually toward the model boundaries. Two examples of tunnel geometry, with different complexity, are shown in Figures 4 and 5.

The actual calculations involved first running each model to an equilibrium pre-excavation state (with all stresses, external loads, and boundary conditions applied). The energy tunnel was then excavated and the model run to a new equilibrium state. Finally, the train- and service tunnels were excavated (simultaneously) and the model run to a final equilibrium state.

Both linear-elastic and perfectly-plastic constitutive models were used. For the plastic models, the excavation stages were performed in two steps. Excavation was first conducted using high material strength values (to inhibit any yielding). The material strengths were then reduced to their correct values and the model allowed to come to final equilibrium state. This sequence was employed to minimize inertial effects (due to sudden geometrical changes), which can lead to unrealistically large yielding in the models.

3.1.2 Parametric study

The uncertainty in input data was handled through a parametric study. A set of cases (with different combinations of parameter values) was selected (partly based on the simplified two-dimensional modeling), which were thought to collectively represent the most critical cases with respect to both deformations and stability in the energy tunnels. Five different cases were defined, as shown in Table 1. It should be noted that to achieve minimum and maximum values on the vertical stress, the density of the rock mass had to be changed (since gravity is acting in the models).
3.1.3 Results

The modeling results were primarily evaluated with respect to deformations and stability conditions around the energy tunnels, as a function of excavation of the train tunnel. Of particular interest were the deformations arising in the roof and the floor of the energy tunnels, and at the brace structures (for the critical installations). An example of calculated roof and floor deformations along one of the energy tunnels is shown in Figure 6.

For this example, relatively small deformations developed as the train and service tunnel were excavated—typically less than 1 mm settlement of both roof and floor. This was, in fact, the case for all tunnel intersections and for Cases 0 and 1, i.e. typical values on all strengths and stresses. For Cases 2, 3, and 4, larger deformations were generally obtained for all intersections. The maximum calculated deformations are summarized in Table 2. It should be noted that maximum deformation in roof and floor does not necessarily occur at the same coordinate; hence, the maximum differential deformations cannot be determined from the values given in Table 2.

It can be seen that Cases 2 and 4 results in significant deformations, in particular for intersections N1 and N2. However, these deformations may be exaggerated, as will be further discussed below.

The differential deformations were specifically calculated at the location of the brace structures in each of the tunnels. An example is shown in Figure 7. The maximum convergence (roof-floor) is between 1 and 3 mm, and the maximum divergence is between 1 and 6 mm for all intersections. However, considering only Cases 0 and 1, the maximum differential deformation is less than 1 mm for all intersections.

The differential settlements along the tunnel were also assessed. For all intersections, the deformations varied smoothly along the tunnel length, with no abrupt changes in settlement or heaving. For intersection N2, a calculated differential settlement of up to 1 ‰ (18 mm deformation over a length of 16 m) could be inferred. For all the other intersections, differential settlements were less than 0.4 ‰.

The overall stability of the energy tunnels may be assessed from modeling using a plastic constitutive model, i.e. Cases 1 through 4 in Table 1. The state occurring after excavation of the energy tunnel corresponds to the present condition in the tunnel. Any yielded zones at this stage can be said to have been handled through the installed reinforcement in the tunnels. However, large areas of yielded ground in the models are probably an exaggeration caused by either too low strengths or too high stresses being input to the models, as the tunnels are stable today, with no signs of global instability.

That said, it was interesting to find that Cases 2 and 4 generally resulted in very large areas of yielding around the energy tunnels (before excavation of
the train tunnel). This was judged to be unreasonable—hence, the minimum strengths and/or the maximum stresses (cf. Table 1) are not thought to accurately reflect the rock mass conditions for any of the intersections. Consequently, the calculated deformations for these cases are also considered highly exaggerated (as discussed above).

![Diagram of tunnel intersections](image)

Figure 6. Calculated vertical deformations in roof and floor along the energy tunnel due to excavation of the train- and service tunnel for intersection N1 and Case 1 (cf. Table 1). Length coordinates and location of of intersection of the energy tunnel is shown in the top figure.

Table 2. Calculated maximum deformations in roof and floor of the energy tunnels as a result of excavation of the train- and service tunnels for each of the tunnel intersections (positive values denote heaving).

<table>
<thead>
<tr>
<th>Intersection no.</th>
<th>Case no.</th>
<th>Roof [mm]</th>
<th>Floor [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>N1</td>
<td>0</td>
<td>-0.58</td>
<td>-0.76</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>-0.59</td>
<td>-0.78</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>11.11</td>
<td>14.4</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>-1.87</td>
<td>-3.01</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>18.75</td>
<td>20.82</td>
</tr>
<tr>
<td>N2</td>
<td>0</td>
<td>1.02</td>
<td>0.09</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>1.08</td>
<td>0.10</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>-6.89</td>
<td>-1.12</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>2.36</td>
<td>0.57</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>14.03</td>
<td>4.93</td>
</tr>
<tr>
<td>N4</td>
<td>0</td>
<td>0.03</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>0.03</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>-0.53</td>
<td>-0.84</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>-1.31</td>
<td>-2.21</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.60</td>
<td>0.26</td>
</tr>
<tr>
<td>N5</td>
<td>0</td>
<td>-0.86</td>
<td>-1.15</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>-0.86</td>
<td>-1.17</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>-2.29</td>
<td>-2.99</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>-2.42</td>
<td>-2.79</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>-2.46</td>
<td>-3.02</td>
</tr>
</tbody>
</table>

With the main focus of this study being the impact of the excavation of the train and service tunnel, it is of interest to study the differences in yielded area occurring from excavation of the train tunnel. The modeling results showed very little additional yielding due to excavation of the train and service tunnel for all intersections. The largest effect was noted for intersection N2, with the smallest rock cover between the tunnels, and the energy tunnel being located below the train tunnel (cf. Figure 5). In this case, additional yielding in the tunnel walls could be noted (primarily due to tensile failure). However, the model was still at equilibrium, i.e. indicating stable conditions. Total collapse and loss of the load-bearing capacity does not occur in any of the intersections or for any of the Cases in Table 1.

To summarize, the analysis showed that no global stability problems can be expected in the energy tunnels for any of the analyzed intersections. Local instabilities may be anticipated for intersection N2, but the extent of these is judged to be minor. The deformations induced due to extraction of the train and service tunnel are generally small (1-2 mm) for the cases that were judged to be realistic. Likewise, differential deformations are small (less than 1 mm). Taken together, it is not likely that these deformations should cause any problems with respect to the function of the energy tunnels.

3.2 Block analysis

Potential wedge failures can impact on the function of the energy tunnels, as falling rock may damage power lines, heating tubes etc, in the tunnels. Since these tunnels are already in existence, blocks formed by pre-existing discontinuities in the rock mass are likely to have slipped/fallen out—either during excavation or through subsequent scaling. Alternatively, existing rock wedges may be securely supported through existing rock reinforcement in the energy tunnels.
The excavation of the Citybanan train tunnel will not affect the block structure around the energy tunnels. However, the stress distribution will be altered, which may facilitate new block failures. This effect was analyzed using the Unwedge program (Rocscience, 2004a). Analyses were conducted for a combination of the minimum, typical, and maximum values of the major joint set orientations. Two sets of stress states were analyzed, corresponding to the stress state before and after excavation of the train tunnel. Stresses were taken from the three-dimensional modeling results, described above. Hence, the safety factors for block failures before and after excavation of the train tunnel could be compared. Finally, safety factors were calculated after installation of reinforcement corresponding to the estimated (from observations) actual rock reinforcement in the energy tunnels. These analyses showed that (for all intersections):

- More than half of the wedges formed were of insignificant volume (less than 0.005 m$^3$).
- A change in stress state due to excavation of the train tunnel has negligible effect on the safety factor for wedge failures (less than 1% change).
- Installed reinforcement results in all wedges having a safety factor larger than 2.

It can thus be concluded that the excavation of the train tunnel does not affect block stability around the energy tunnels.

4 ANALYSIS OF TRAIN TUNNEL

4.1 Methodology

The design of the train tunnel—in particular the load-bearing structure—was conducted according to a common methodology for the Citybanan project. This methodology is essentially the same one as the proposed guidelines by Banverket (the Swedish National Railroad Administration). These guidelines are described in more detail in Sjöberg et al. (2006).

4.2 Preliminary reinforcement design

The first step in the design methodology is the identification of possible failure mechanisms for each design situation (in this case each tunnel intersection). This assessment was based on the available data on the rock mass conditions, results from numerical modeling without installed reinforcement, and experience.

Since the tunnels are fairly shallow, block failures caused by intersecting discontinuities is considered a plausible failure mechanism. The rock quality is fair, which, coupled with the moderate depth, point at few global stability concerns. The numerical modeling described above (for the energy tunnel), i.e. finalized excavation of the train and service tunnel representing a worst-case scenario, also indicated only local instabilities. Local shear and tensile failure can be expected, primarily in the walls of the train tunnel. Local yielding at the tunnel boundary may, however, result in loosening of rock between installed rock bolts, which must be accounted for in the design.

A preliminary reinforcement design was made based on empirical and analytical calculations. The NGI index ($Q$-system) was used to assess the reinforcement requirements based on rock mass classification (NGI 2005, Barton 2002). This was done for minimum and typical rock condition values and for the different spans of the train tunnel, in each intersection, respectively.

Following this, shotcrete thickness was calculated analytically for cases of good and poor adhesion between rock and shotcrete. Adhesion failure, bending resistance and punching failure through the shotcrete were considered, and the required shotcrete thickness as a function of bolt spacing was determined.

Block analysis using Unwedge (Rocscience, 2004a) was conducted to assess the safety against structurally controlled block failures. In this case, a combined analysis with all joint orientations (estimated variability) was performed and for a worst-case scenario in terms of stresses and joint shear strengths. A limited sensitivity study was conducted. Subsequently, reinforcement according to the preliminary design (empirical assessment) was installed and the resulting factors of safety evaluated.

Using all information from the above described analyses, a reinforcement scheme was proposed. This involved systematic rock bolting using grouted rebars of 4 m length at bolt spacings of 1.2 to 1.4 m, coupled with fiber-reinforced shotcrete with 120-200 mm thickness for the different intersections, respectively. The extraction sequence for the train tunnel (pilot tunnel, stoping, maximum lengths, etc) was also reviewed and necessary adjustments made to achieve a practical and safe extraction sequence.

4.3 Numerical analysis

4.3.1 Model setup

The three-dimensional models used for the analysis of the energy tunnels were modified to incorporate rock reinforcement and sequential excavation, mimicking the actual extraction sequence for the train and service tunnel. The model geometry and boundary conditions remain unchanged.

The extraction sequence through the intersection area involved excavation of a pilot tunnel (in increments) and for a maximum length of 12 m. Roofs and walls were then excavated (to full width) in rounds of maximum 2.5 m. Reinforcement was installed up to tunnel front. Stoping was continued until the full tunnel width was excavated to the end of the pilot tunnel. The pilot tunnel was then excavated in another round, and the sequence repeated. This sequence was also simulated in the model. An ex-
ample is shown in Figure 7. Subsequently, the service tunnel was excavated (in one step). The pilot tunnel and the service tunnel were unsupported.

For intersection N2, stoping of roofs and walls was supplemented with a bottom bench (with horizontal blast holes) to reduce blast influence on the underlying energy tunnel. For the intersection in which two parallel, single-track tunnels were planned, excavation of these was done simultaneously. In reality, one of these may be excavated ahead of the other depending on actual conditions. Apart from this, the above described sequence, including excavation of the service tunnel last, is believed to reflect a worst-case scenario in terms of loads on the installed rock reinforcement in the train tunnel.

Only a perfectly-plastic constitutive model was used, and calculations were only conducted for typical material properties and stresses, corresponding to Case 1 in Table 1. This limitation (no sensitivity study) was necessary due to time constraints, and the fact that the sequential excavation for each tunnel intersection was very time-consuming.

Installed rock reinforcement corresponded to the preliminary reinforcement scheme described above. Rock bolts were simulated using the built-in rock bolt model in FLAC3D. This model can simulate both tensile and shear loading of the rock. Rock bolt capacity can be expressed in both maximum tensile force and maximum extension (%). Shotcrete was simulated as a linear-elastic shell element (no plastic material model available for this structural element).

![Figure 8. Simulated tunnel geometry and installed rock reinforcement for excavation of pilot tunnel and subsequent stopping in the three-dimensional numerical model for intersection N1.](image)

4.3.2 Results
The calculated deformations in the train tunnel were small, ranging between 2 and 6 mm horizontal displacement of the tunnel walls, for the analyzed intersections. The calculated induced stress in the tunnel roof was between 8 and 12 MPa. These stresses are sufficient to provide some confinement to rock blocks in the roof, thus increasing their stability. At the same time, these stresses are too low to cause any significant stress-induced fracturing.

The tunnel stability can be further assessed by studying the extent of yielding in the plastic models. Yielding is very moderate for all analyzed intersections, with primarily local tensile failure developing in the tunnel walls. An example is shown in Figure 9. It is unlikely that this minor amount of yielding in the rock mass would cause any significant stability problems. It should further be noted that the models (for all intersections) are at equilibrium, i.e. indicating stable conditions.

In evaluating the performance of the installed rock reinforcement, particular emphasis was placed on strains in rock bolts, and outer fiber stresses in the shotcrete. The use of bolt strains as criteria for bolt performance is motivated by the fact that grouted rebar bolts can sustain very large strains, even if the tensile load is close to the yield strength of the steel. In this case, a limiting strain of 5% was used (common for all Citybanan design). For the shotcrete, the outer fiber stress was evaluated and compared to the compressive and tensile design strengths for the chosen shotcrete.

The numerical models showed relatively moderate straining of the rock bolts. The maximum strain in any bolt segment was less than 0.2%. Each segment is 0.5 m (total bolt length is 4 m). An example of calculated strains in each bolt segment is shown in Figure 10. The maximum bolt forces are around 50 kN, i.e. about 20% of the tensile strength of 246 kN for these types of bolts. An exception is for intersection N2, in which one double-track tunnel is split into two single-track tunnels. Bolt loads of up to 210 kN are indicated in the transfer area. However, this is still below the tensile strength of the bolts; thus, no bolt failures are anticipated in any of the intersections.

For the installed shotcrete, the numerical modeling indicated that no compressive failure was to occur. The tensile stresses were, however, close to and, in some cases, above the tensile design strength of the shotcrete (in this case 4.0 MPa bending tensile strength). This would indicate that local cracking may occur. However, this is also an effect of the shotcrete being installed right up to the tunnel front in each excavation sequence. The developed tensile stresses may thus be exaggerated, and less of a problem in actual construction. No collapse of the shotcrete is indicated. Nevertheless, this should be studied further through parametric studies varying shotcrete properties and installation sequence, to better verify the behavior and capacity of the shotcrete reinforcement.

To summarize, the numerical modeling results indicated stable tunnel conditions during the excava-
tion of the Citybanan train tunnel. Installed reinforcement should be able to handle any local instabilities (loosened rock, etc). The rock bolt reinforcement did not exhibit any failure. Local failure of the shotcrete may be inferred, but this effect is judged to be smaller during actual construction. Since the global stability is satisfactory, it was judged that installed shotcrete is adequate for this design situation.

Based on these results, the preliminary reinforcement design were reviewed and, in some cases, adjusted. Bolt lengths and spacings were adjusted to conform to standard bolt patterns determined for the entire Citybanan project. The final proposed reinforcement scheme for the train tunnel is shown in Table 3. Reinforcement should be installed in both the roof and walls of the train tunnel. All bolts are 25 mm steel rebars with a tensile strength of 246 kN.

Table 3. Final reinforcement design for the train tunnel in the intersection area.

<table>
<thead>
<tr>
<th>Intersection no.</th>
<th>Bolting</th>
<th>Shotcrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>N1</td>
<td>Systematic, grouted rebars, 4.0 m length, 1.4 m spacing</td>
<td>Fiber reinforced 100 mm thickness</td>
</tr>
<tr>
<td>N2, double-track tunnel</td>
<td>Systematic, grouted rebars, 4.0 m length, 1.4 m spacing</td>
<td>Fiber reinforced 100 mm thickness</td>
</tr>
<tr>
<td>N2, single-track tunnel</td>
<td>Systematic, grouted rebars, 4.0 m length, 1.7 m spacing</td>
<td>Fiber reinforced 75/50 mm thickness (roof/wall)</td>
</tr>
<tr>
<td>N4</td>
<td>Systematic, grouted rebars, 4.0 m length, 1.7 m spacing</td>
<td>Fiber reinforced 100 mm thickness</td>
</tr>
<tr>
<td>N5</td>
<td>Systematic, grouted rebars, 4.0 m length, 1.4 m spacing</td>
<td>Fiber reinforced 150 mm thickness</td>
</tr>
</tbody>
</table>

5 DISCUSSION AND CONCLUSIONS

This case study has clearly showed the benefits, and necessity, of using advanced numerical modeling to assist with the design of critical aspects of shallow infrastructure tunnels. For the Citybanan tunnel intersections, three-dimensional modeling was necessary, as governed by the complex geometry. Furthermore, the criticality of the intersections as such (thin rock cover, fairly shallow tunnels, large public exposure), strongly motivated the use of sophisticated analysis methods.

At the same time, simpler and easy-to-use methods such as empirical rock mass classification design and block analysis, were important adjuncts to the numerical modeling, in e.g. preliminary assessment of reinforcement requirements. The use of different methods coupled with sound engineering judgment constitutes a success factor in these types of complex and demanding urban tunnel design and construction. An important aspect of the numerical modeling was the use of parametric studies to quantify the sensitivity in the results, and to gain knowledge of the important factors governing the behavior of the rock surrounding the tunnels.

For the existing energy tunnels close to the planned Citybanan train tunnel, the analysis showed that no significant deformations are to be expected in the energy tunnels due to the excavation of the train tunnel. The induced deformations are predicted to be less than 2 mm (for any intersection) with differential deformations less than 1 mm. No global stability problems are anticipated, and existing reinforcement of the energy tunnels is more than sufficient with respect to possible block failures in the tunnel.
The analysis of the Citybanan train tunnel was successful in confirming the proposed rock reinforcement design for the train tunnels, thus securing the stability of the load-bearing structure for the rock tunnel. Based on the results of the numerical modeling, no global stability problems are expected. Local yielding and/or block failure may occur, but this is controlled through installed rock reinforcement. The analysis verified that the installed rock bolts are only loaded to a small percentage of their ultimate capacity. Shotcrete is subject to higher tensile stresses, but this may be an effect of installation- and excavation sequence in the model. This effect is judged to be less prominent during actual construction.

It is recommended that additional sensitivity studies are conducted for the excavation of the train tunnel. These studies should also utilize the collected data from the detailed site investigations currently ongoing. The site investigations should, in turn, focus on reducing the estimated variability of the material properties, since this has been shown to have significant impact on calculated deformations and inferred stability conditions around the tunnels.

During construction, it is essential that a monitoring program is in effect. This should include (i) geological mapping, (ii) visual inspections of tunnel and reinforcement behavior, (iii) deformation measurements in the energy and train tunnels, (iv) control of reinforcement elements, and, if necessary, (v) supplementary analysis. The results from the present study can be used to plan the deformation monitoring program well ahead of actual construction start. Thus, it is possible to refine the design of the Citybanan train tunnel through observations and measurements during construction, leading to a more optimal final product.

ACKNOWLEDGEMENTS

The work presented in this paper is part of the design work for the Citybanan project, commissioned by Banverket. The work presented in this paper was, in turn, subcontracted to SwedPower by WSP. The permission by Banverket and WSP to publish this work is hereby acknowledged.

REFERENCES


