Design of Initial Support Required for Excavation of Underground Cavern and Shaft from Numerical Analysis

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ABSTRACT

Excavation of underground cavern and shaft was proposed for the construction of a ventilation facility in an urban area. A shaft connects the street-level air plenum to an underground cavern, which extends down approximately 46 m below the street surface. At the project site, the rock mass was relatively strong and well-defined joint sets were present. A kinematic block stability analysis was first performed to estimate the required reinforcement system. Then a 3-D discontinuum numerical analysis was conducted to evaluate the capacity of the initial support and the overall stability of the required excavation, followed by a 3-D continuum numerical analysis to complement the calculated result. This paper illustrates the application of detailed numerical analyses to the design of the required initial support system for the stability of underground hard rock mining at a relatively shallow depth.

1. INTRODUCTION

The Long Island Rail Road (LIRR) provides passenger service from 10 branch lines on Long Island through the Amtrak tunnels under the East River to the west side of Manhattan into Penn Station in New York City, NY. The East Side Access (ESA) Project will enable the LIRR to provide direct service to the east side of Manhattan. As part of the ESA project, the construction of a ventilation facility was proposed on East 55th Street. The facility consists of a traction power substation and tunnel ventilation plant beneath East 55th Street on the west side of Park Avenue in New York City. The construction of the street-level air plenum was planned to be performed using cut-and-cover methods to approximately 9.1 m (30 ft) below the stress surface. A 12.2 m (40 ft)
deep by 10.4 m (36 ft) diameter shaft connects the air plenum to the mined cavern. The cavern is 43.6 m (143 ft) long and 25.9 m (85 ft) high at the center of the crown, and extends down to the lower tracks, approximately 45.7 m (150 ft) below the street surface. A layout of the project area and a cross-section are displayed in Fig. 1. Given proposed excavation profiles, special attention is paid to the stability evaluation of the intersection area between shaft and cavern shoulder and crown. This paper describes a case study of initial support design for underground hard rock mining at a relatively shallow depth. The study shows how detailed numerical analyses can be practically utilized to evaluate the overall stability for the excavation of underground cavern and shaft, and to provide the initial support design for the required excavation operations.

2. GEOLOGICAL SETTING

2.1 General geology

The rocks of New York City comprise three lithologically distinct sequences of a metamorphic assemblage of Proterozoic to Lower Paleozoic age consisting of schist, gneiss and marble (Baskerville, 1994; Fuller et al., 1999). The rocks in the project area
belong to the Hartland Formation of Lower Cambrian to Middle Ordovician age and overlie the Manhattan Schist of Lower Cambrian age (Baskerville, 1994; Sanders and Merguerian, 1997). The rocks of Manhattan have a complex structural history due to several superimposed phases of deformation (Shah et al., 1998). The multiple deformation phases have created an intensely folded and locally sheared rock mass with penetrative fabric, total recrystallization and localized partial melting of the rocks. The most prominent fold phase consists of asymmetrical and associated folds that define the regional structure of Manhattan. The axial planes strike N35ºE and generally plunge at low to moderate angles (about 10º to 15º) toward south-southwest. The general style of these folds is a relatively long limb dipping gently toward the east and a shorter limb dipping steeply toward the west. These folds are characterized by flexural-slip surfaces along foliation (Baskerville, 1994; Sanders and Merguerian, 1997).

2.2 Discontinuities

Published information states that at least four major joint sets have generally been recognized in Manhattan Island (Cording and Mahar, 1974). The most prominent joint set, Set No. 1, lies parallel to the plane of weakness formed by foliation and strikes N30º to 35ºE with a 70º to 80ºSE or 60º to 70º NW dip. Set Nos. 2 and 4 generally strike perpendicular to the foliation jointing with dips in the range of 70º to 80ºSW for Set No. 2 and about 75ºNE for Set No. 4. Set No. 3 appears to run parallel to the foliation but dips 60º to 70º in a direction opposite to Set No.1 and has been termed its conjugate. In addition, there exist low-angle joints, essentially striking parallel to Joint Sets 2 and 4 with dips of about 25ºSW. Secondary joints, whose strikes and dips differ slightly from those for the four dominant joint sets, have also been observed. The attitudes of the Joint Set Nos. 2, 3 and 4 appear to change with changes in the attitude of foliation.

The existence of four dominant joint sets in this rock mass have been confirmed by geological mapping of the south wall of the Grand Central Terminal, oriented core borings and joint traces in the borehole walls (MTA CC-LIRR, 2005). However, based on the investigation data, the attitudes of the joint sets occurring along the ESA alignment are different from the published data presented above. The dip angle and direction data at the 55th Street Ventilation Facility vicinity, which were obtained from the geotechnical investigation program undertaken for this project are summarized in Table 1. Foliation shear zones are present throughout the rock mass and are oriented within 35º of North and dip at angles of 40º to 80º in a westward or eastward direction, essentially paralleling Set No. 1 (Cording and Mahar, 1974). Transverse fault zones, cutting across foliation, are present in these rock formations. These zones are well developed and are generally much wider than the foliation or conjugate shear zones. Rock structures within these fault zones are very blocky and seamy, and many surfaces are likely to be sheared (Cording and Mahar, 1974; MTA CC-LIRR, 2005).
Table 1 Observed joint dip angles and dip directions (range and mean values)

| Joint Set Attitudes | Set 1 | | | Set 2 | | |
|---------------------|-------|---|---|-------|---|
|                    | Dip   | Dip Direction | Dip | Dip Direction |
|                     | 15° to 45° | 250° to 300° | 40° to 90° | 250° to 300° |
| Mean: 30°           | Mean: 279° | Mean: 70° | Mean: 286° |
|                     | 40° to 90° | 90° to 160° | 60° to 90° | 300° to 330° |
| Mean: 55°           | Mean: 147° | Mean: 72° | Mean: 311° |
|                     | 45° to 80° | 220° to 250° | 60° to 90° | 20° to 60° |
| Mean: 55°           | Mean: 239° | Mean: 80° | Mean: 54° |

2.3 Geology of the project site

The principal rock types that were encountered during the construction of the Manhattan segment of the ESA Project possess a variety of geological characteristics. Three borings were drilled in the 55th Street Ventilation Facility area. From west to east, these are: MA-309, MA-308, and MA5 (Fig. 1). Boring MA-309 was drilled from street level, about elevation 107.3 m (352 ft), at 55th Street centerline, about 85.3 m (280 ft) west of track WB3. Top of rock (the level at which rock coring began) was at about a depth of 11 m (36 ft). Rock type was primarily gray medium to fine-grained schist, and rock quality throughout the boring was mostly good to excellent with localized exceptions. Boring MA-308 was drilled from street level, about elevation 106.7 m (350 ft), at 55th Street centerline, about 57.9 m (190 ft) west of track WB3. Top of rock was at about a depth of 3 m (10 ft). Rock type and quality throughout the boring were very similar to those of Boring MA-308. Weathering was slight to none, and the rock was entirely unweathered below the depth of 27.4 m (90 ft). Boring MA-5 was drilled from Metro North Tunnel about elevation 99.7 m (327 ft), along track WB3 about 3 m (10 ft) south of centerline 55th Street. Top of the excavated rock surface was at a depth of 0.6 m (2.0 ft). Rock quality generally improved with depth. Within the uppermost 3.7 m (12 ft) of the rock, weathering was greater and fracture spacing was closer than elsewhere in the boring, possibly related to previous excavation at this location. Weathering was slight to none elsewhere in the boring. Detailed reviews of each boring and the coring log of each boring are described in the Geotechnical Data Report (MTA CC-LIRR, 2005).

3. NUMERICAL ANALYSIS FOR INITIAL SUPPORT DESIGN

A comprehensive series of numerical analyses was performed for initial support requirements and the overall stability of the required excavation. The study included kinematic stability calculations to estimate the maximum sizes of kinetically feasible wedges and preliminary rock reinforcement requirements and numerical modelling to assess the stress redistribution associated with the proposed excavation geometry, excavation-induced displacements and potential ground failure due to the discontinuities present in the project area. The geometry of the problem was three-
dimensional as shown in Fig. 1 and was analyzed using three-dimensional numerical codes, 3DEC (Itasca, 2003) and MIDAS (Midas GTS, 2007).

The rocks in the project area are relatively strong and the response of the rock mass to excavation activity is dominated by displacements and failure along discontinuities. Thus the discontinuum approach to assessing the stability of cavern and shaft during excavation is deemed to be suitable (Barla and Barla, 2000; Hashash et al., 2002; Wang et al., 2014). In 3DEC, the discontinuous medium is represented as an assemblage of discrete blocks; and the discontinuities are treated as boundary conditions, thereby allowing the detachment and rotation of blocks (Itasca, 2003). On the other hand, the continuum modelling code, MIDAS, was employed to capture the stress concentration at the intersection between shaft and cavern to complement the discontinuum analysis. The shaft and cavern numerical model has the following features:

1. Model geometry: Fig. 1 shows that there are three caverns at this location, namely high cavern, mid-height cavern, and low cavern. Numerical modelling was conducted for a high cavern only because it is most critical, which is approximately 14.6 m (48 ft) wide and 25.9 m (85 ft) high. The cavern is approximately 43.6 m (143 ft) long and connects to an approximately 12.2 m × 10.4 m (40 ft × 34 ft) diameter shaft. The top of the model is set approximately at the bottom of the air plenum. Fig. 2 shows the initial 3DEC model of which the dimensions are 76.2 m × 61.0 m × 67.1 m (250 ft × 200 ft × 220 ft), and a high cavern and shaft generated as part of numerical simulation. Fig. 3 shows an initial model and a high cavern and shaft, which are constructed from MIDAS for continuum analysis.

2. Initial conditions/stresses: Vertical stress is assumed to be a unit weight of rock multiplied by depth. The in-situ stress ratio, K0 ranges from 1.4 to 3.2 based on 11 hydraulic fracturing tests (MTA CC-LIRR, 2005), and various values were imposed in the model to observe the influence of K0 values on the stability of underground structures.

3. Rock mass properties: The topsoil layer is represented by an equivalent overburden pressure. In 3DED, the intact rock is modeled as linear elastic material on the basis of the assumption that displacements occur along discontinuities. On the other hand, MIDAS uses the Mohr-Coulomb failure criterion to represent the stress-strain relationship of the rock mass. The values of material parameters used in the analyses were obtained in part from laboratory tests (MTA CC-LIRR, 2005) and experiences gained elsewhere, and are listed in Table 2.

4. Rock joint properties: Rock joint behaviour is modeled using the Mohr-Coulomb failure criterion in 3DEC. The shear strength of joint is represented by a friction angle only, and tensile strength and cohesion are conservatively assumed to be zero. In order to be more realistic about joint orientation and spacing, 3DEC modelling utilizes a uniform probability distribution for joint spatial data. The values of joint parameters used in 3DEC simulation are shown in Table 3.
**Fig. 2** Three dimensional numerical model (3DEC) of a high cavern and shaft (rock blocks are defined by joint sets and represented by different colours)

**Fig. 3** Three dimensional numerical model (MIDAS) of a high cavern and shaft (sequential excavation is represented by different colours)
Table 2 Material model properties used in the analyses

<table>
<thead>
<tr>
<th></th>
<th>K (GPa)</th>
<th>G (GPa)</th>
<th>( \gamma ) (kN/m(^3))</th>
<th>E (GPa)</th>
<th>( \sigma_t ) (MPa)</th>
<th>c (MPa)</th>
<th>( \phi ) (degree)</th>
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<tr>
<td>3DEC</td>
<td>18.4</td>
<td>13.8</td>
<td>26.7</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>MIDAS</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>17.2</td>
<td>2.1</td>
<td>4.4</td>
<td>52</td>
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</table>

Table 3 Material model properties used in the analyses

<table>
<thead>
<tr>
<th>Joint Set</th>
<th>Dip (°)</th>
<th>DD (°)</th>
<th>( \phi ) (°)</th>
<th>Spacing (m)</th>
<th>( k_n ) (GPa/m)</th>
<th>( k_s ) (GPa/m)</th>
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</thead>
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<tr>
<td>1</td>
<td>30 (dev. 2)</td>
<td>279 (dev. 2)</td>
<td>25</td>
<td>1.8 (dev. 2)</td>
<td>6.9</td>
<td>0.69</td>
</tr>
<tr>
<td>2</td>
<td>70 (dev. 2)</td>
<td>286 (dev. 2)</td>
<td>25</td>
<td>3.0 (dev. 2)</td>
<td>6.9</td>
<td>0.69</td>
</tr>
<tr>
<td>3</td>
<td>55 (dev. 2)</td>
<td>147 (dev. 2)</td>
<td>25</td>
<td>3.0 (dev. 2)</td>
<td>6.9</td>
<td>0.69</td>
</tr>
<tr>
<td>4</td>
<td>72 (dev. 2)</td>
<td>311 (dev. 2)</td>
<td>30</td>
<td>3.0 (dev. 2)</td>
<td>6.9</td>
<td>0.69</td>
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<tr>
<td>5</td>
<td>55 (dev. 2)</td>
<td>239 (dev. 2)</td>
<td>30</td>
<td>3.0 (dev. 2)</td>
<td>6.9</td>
<td>0.69</td>
</tr>
<tr>
<td>6</td>
<td>80 (dev. 2)</td>
<td>54 (dev. 2)</td>
<td>30</td>
<td>3.0 (dev. 2)</td>
<td>6.9</td>
<td>0.69</td>
</tr>
</tbody>
</table>

4. ANALYSIS RESULTS AND INITIAL SUPPORT DESIGN

4.1 Kinematic block stability analysis

As a preliminary investigation, kinematic analysis was conducted using the UNSWEDGE software (Rocscience, 2011) for all design sections in order to derive a basic initial support design. UNSWEDGE is a 3D stability analysis program based on a limit equilibrium method and generates the largest rock wedges that can be formed for the given geometrical conditions. In this analysis, however, an apex height of 7.6 m (25 ft) was used to scale the wedges down to more realistic sizes based on field observations. The structural discontinuities, such as joints and foliations, included in the analysis are assumed to be planar and continuous, which generally give conservative results. The joint orientations and friction angles shown in Table 3 are also used in the kinematic analysis. Rock bolt properties used in the analysis are shown in Table 4.

Table 4 Rock bolt properties used in the analyses

<table>
<thead>
<tr>
<th>Type</th>
<th>E (GPa)</th>
<th>( \nu )</th>
<th>( \gamma ) (kg/m(^3))</th>
<th>Plate capacity (MN)</th>
<th>Bond strength (MN/m)</th>
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<tr>
<td>#9 rock bolt (Grade 75)</td>
<td>200</td>
<td>0.3</td>
<td>7849</td>
<td>0.1</td>
<td>0.3</td>
</tr>
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</table>

Fig. 4 shows kinematic analysis results with typical wedges supported by rock bolts at shaft and cavern. For a minimum factor of safety (FS) of 1.5, the required rock bolt patterns were computed to be 3.6 m (12 ft) long at 1.8 m (6 ft) horizontal and 1.5 m (5 ft) vertical spacing for shaft, and 4.6 m (15 ft) long at 1.5 m (5 ft) horizontal and 1.5 m (5 ft) vertical spacing for cavern. It should be noted that the current version of the UNSWEDGE program is not able to model 3-D geometry of intersections accurately. Thus, empirical approaches were considered to determine the required rock bolts at intersections, so that 4.6 m closely spaced long bolts would be installed in a staggered way between the shaft and shoulder of the cavern. The design is examined further using 3DEC numerical simulations.
4.2 Discontinuum modelling

A series of numerical simulations was performed using the 3-D distinct element code, 3DEC. 3DEC simulated the sequential excavations with the initial support at each stage. A shaft was first excavated, and followed by the central drift of the heading and the side drifts of the heading. Bench excavations were finally made. K0 values of 1.0, 1.5 and 2.5 were included in the simulations and Fig. 5 shows an analysis result without rock bolts for K0 = 1.0, which resulted in the worst case (most block/wedge fall-outs), presumably due to less clamping stress.
Further analyses were conducted with the installation of the rock bolts designed by the aforementioned kinematic analysis. Fig. 6 shows a 3DEC model of the rock bolt system installed at shaft and cavern, and the analysis result for $K_0 = 1.0$. As indicated in the result, all loose wedges shown in Fig. 5 were stabilized by the proposed reinforcement system; and only small blocks fell out between rock bolts, which should be prevented by scaling after blasting and surface protections such as shotcrete. Therefore, three-dimensional discontinuum numerical analyses confirmed that the required excavation with the proposed reinforcement system could be carried out without potentially affecting the stability of adjacent buildings and structures in an urban area. Fig. 7 depicts the proposed rock bolt lengths and patterns at different sections of the shaft and cavern.
Fig. 6 3DEC model with rock bolts and an example of simulation results.

(a) rock bolt system installed in 3DEC model

(b) displacement vectors at cross section along the shaft ($K_0 = 1.0$)

Fall-out of Small blocks between bolts
4.3 Continuum modelling

Continuum numerical analyses were conducted using MIDAS software to evaluate the overall stability of the excavation and stress redistribution around the intersection of the cavern and shaft. As 3DEC did, a MIDAS modelling sequence followed the excavation sequence: shaft, cavern heading and cavern bench. K0 values
of 1.0, 1.5 and 2.5 were also simulated and Fig. 8 presents analysis results for K₀ = 2.5, which provided more stress concentration at the intersection areas than the other two cases. As shown in Fig. 8, the maximum calculated stress at the intersection area is approximately 3 MPa. This range of stress is far below the value of rock strength in the project area and implies that heavy support, such as steel rib, would not be required. However, if discontinuities are present in the intersection area, this magnitude of stress can possibly trigger wedge failures by crushing and shearing off irregularities of discontinuities (Cording et al., 1971; Oh et al., 2015); and thus a proper reinforcement system is required.

Fig. 8 MIDAS analysis results for K₀ = 2.5 – principal stresses, σ₁ at different sections
5. DISCUSSION AND CONCLUSIONS

Current practice in the design of underground structures tends to be based on precedent. The requirement of a support system can thus be determined from the knowledge obtained from previous projects on similar ground or experience gained elsewhere. The construction of underground structures under conditions not previously encountered necessitates other design methods, and a numerical method can be employed as one of the methods.

The fast advance of numerical modelling techniques has enabled their versatile applications to underground construction problems such as (but not limited to) stability analysis, parametric studies, comparative design, back analysis, etc. (e.g. Wei et al., 2015; Barla, 2016; Chen et al., 2016; Yang and Li, 2016; Hadjigeorgiou and Karampinos, 2017). However, it should be mentioned that the predictions of numerical modelling are about as reliable as input values used in the analysis. The determination of input values that properly represent the field behaviour of rock masses is challenging. It usually involves a certain degree of assumption in addition to those made when defining material constitutive relations or failure criteria. Therefore, observations during construction are essential to confirm the suitability of the selected input parameters and assumptions made in the modelling.

There are no universally accepted methods in designing reinforcement systems at intersections, but some empirical methods address the design issues in the case of tunnel intersections. For instance, Q systems (Barton et al., 1974) use the factor of three when calculating joint set number ($3 \times J_n$) and Engineer Manual, EM1110-1-2907 (U.S. Army Corps of Engineers, 1980) multiplies two when determining the confining pressure at intersections. More recently a probabilistic approach was employed to analyses stability of mine development intersections, thereby recommending the installation of additional or secondary rock supports (Abdellah et al., 2014). Along the same lines as those methods, the study in this project proposed the rock reinforcement at narrow spacing in staggered pattern installed from both shaft and cavern, as described in Fig. 7.

This paper presents numerical analyses of initial support design for the excavation of underground shaft and cavern at shallow depth. At the project site, the rock mass is relatively strong and several well-defined joint sets are present. The most common types of failure given the geologic conditions are those involving wedges or blocks falling out from shaft and cavern during the excavation. Therefore, kinematic block analysis and discontinuum analysis were employed primarily to determine the required reinforcement, followed by continuum analysis to complement the proposed initial support design. The numerical analyses conducted in this study demonstrate the feasibility of the required excavation with the proposed reinforcement system in urban area construction.
REFERENCES


MTA CC-LIRR (2005), *Geotechnical Data Report (GDR), Construction contract CM0009 Manhattan Tunnels Excavation, ESA Project*.


