Geomechanics and Engineering, *Vol. 17, No. 6 (2019) 573-581*
DOI: https://doi.org/10.12989/gae.2019.17.6.573

Design of initial support required for excavation of underground cavern and shaft from numerical analysis

Joung Oh[1a], Taehyun Moon[2b], Ismet Canbulat[1c] and Joon-Shik Moon[*3]

[1]School of Minerals and Energy Resources Engineering, University of New South Wales, Sydney, NSW 2052, Australia
[2]Geotechnical and Tunneling Division, HNTB, Empire State Building 57th FL, New York, NY 10119, U.S.A.
[3]Department of Civil Engineering, Kyungpook National University, 80 Daehak-ro, Buk-gu, Daegu 41566, Republic of Korea

(Received April 25, 2017, Revised January 29, 2019, Accepted March 20, 2019)

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.

Keywords: underground cavern; kinematic block stability; 3-D discontinuum numerical analysis; 3-D continuum numerical analysis; hard rock mining

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.

*Corresponding author, Associate Professor
E-mail: j.moon@knu.ac.kr
[a]Senior Lecturer
E-mail: joung.oh@unsw.edu.au
[b]Ph.D.
E-mail: tmoon@hntb.com
[c]Professor
E-mail: i.canbulat@unsw.edu.au

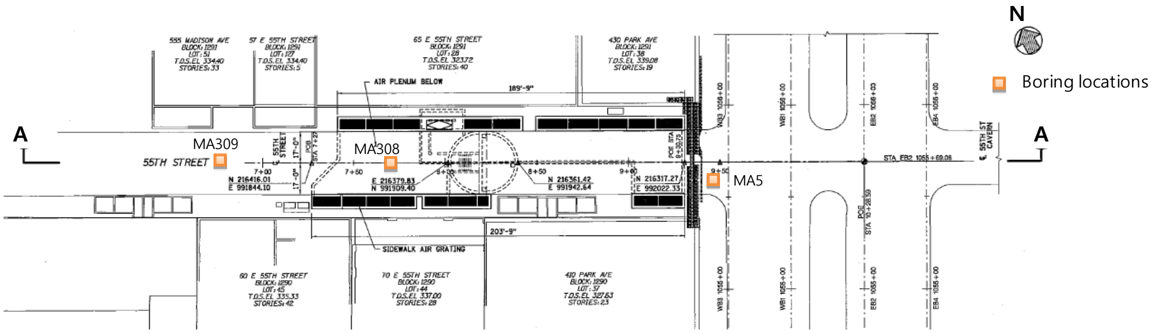
When an excavation is made in a rock mass, stress redistribution occurs in the vicinity of the opening. An excavation also creates a free surface which could allow relief of confinement normal to it and loose blocks or wedges to fall or slide from the surface in a jointed rock mass. Therefore, initial support should be installed shortly after excavation in order to secure a safety of the underground opening. Due to the inherent variability of geologic materials, there is no accepted standard code for the design of initial support. It is usually determined based on a variety of rules; empirical methods such as Rock Mass Rating, RMR (Bieniawski 1989) and Q-system (Barton *et al.* 1974), theoretical or analytical methods such as the convergence-confinement method (Carranza-Torres and Fairhurst 2000, Cui *et al.* 2015), and various types of numerical analyses. Additionally, in order to overcome uncertainties involved in selecting material parameters, reliability-based approaches have been applied to ground support design (Qing *et al.* 2017). This paper describes a case study of initial support design for underground hard rock mining at a relatively shallow depth. Given proposed excavation profiles, special attention is paid to the stability evaluation of the intersection area between shaft and cavern shoulder and crown. 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

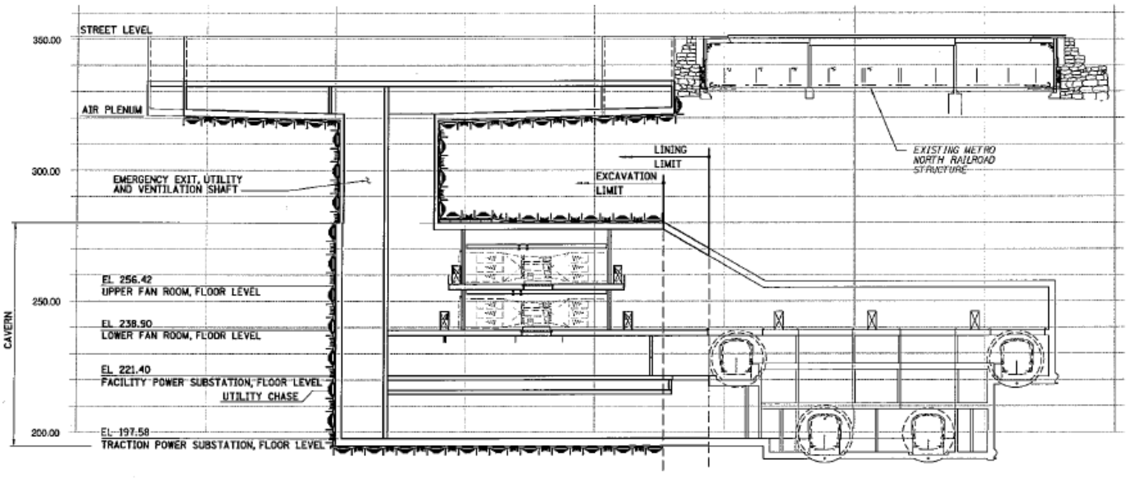
2.1 General geology

The rocks of New York City comprise three

http://www.techno-press.org/?journal=gae&subpage=7
ISSN: 2005-307X (Print), 2092-6219 (Online)



(a) General plan view of the project area



(b) Section view along AA

Fig. 1 General plan view and section view of the project area

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 2005b). 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

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° Mean: 30°	250° to 300° Mean: 279°	40° to 90° Mean: 70°	250° to 300° Mean: 286°
Set 3		Set 4	
Dip	Dip Direction	Dip	Dip Direction
40° to 90° Mean: 55°	90° to 160° Mean: 147°	60° to 90° Mean: 72°	300° to 330° Mean: 311°
Set 5		Set 6	
Dip	Dip Direction	Dip	Dip Direction
45° to 80° Mean: 55°	220° to 250° Mean: 239°	60° to 90° Mean: 80°	20° to 60° Mean: 54°

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 2005a).

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, 2005b).

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).

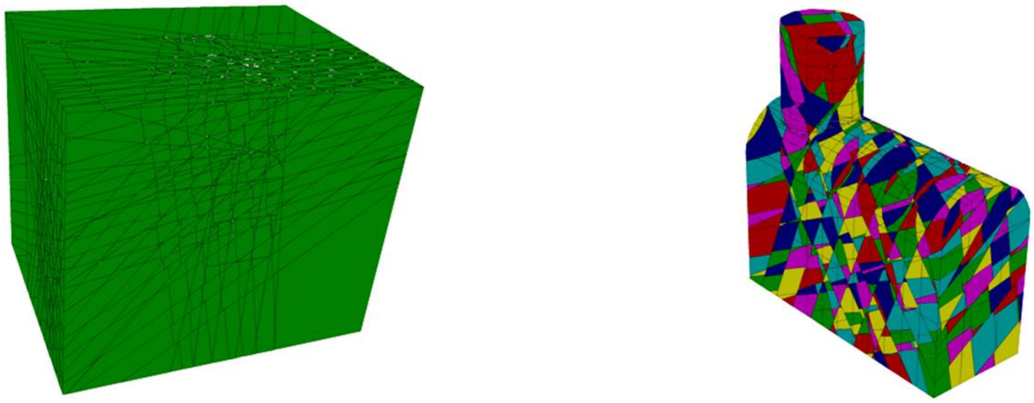
The rocks in the project area are relatively strong and the response of the rock mass to excavation activity is dominated by displacements and failure along discontinuities. Thus the discontinuum approach to assessing the stability of cavern and shaft during excavation is deemed to be suitable (Barla and Barla 2000, Hashash *et al.* 2002, Wang and Huan 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 2005b), and various values were imposed in the model to observe the influence of K0 values on the stability of underground structures.

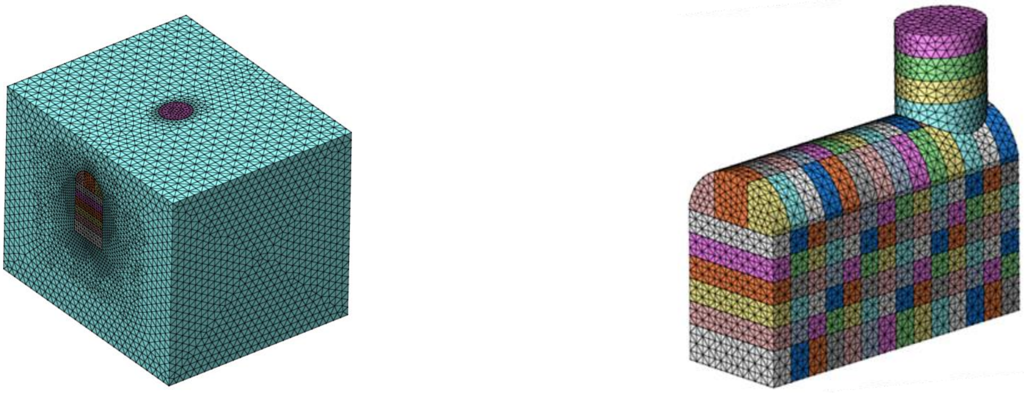
3. Rock mass properties: The topsoil layer is represented by an equivalent overburden pressure. In 3DED, the intact rock is modeled as linear elastic material on the basis of the assumption that displacements occur along discontinuities. On the other hand, MIDAS uses the Mohr-Coulomb failure criterion 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, 2005b) 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



(a) Initial model (b) Cavern and shaft

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)



(a) Initial model (b) Cavern and shaft

Fig. 3 Three dimensional numerical model (MIDAS) of a high cavern and shaft (sequential excavation is represented by different colours)

Table 2 Rock model properties used in the analyses

	K (GPa)	G (GPa)	γ (kN/m^3)	E (GPa)	σ_t (MPa)	c (MPa)	ϕ (degree)
3DEC	18.4	13.8	26.7	-	-	-	-
MIDAS	-	-	-	17.2	2.1	4.4	52

Table 3 Joint model properties used in the analyses

Joint Set	Dip (°)	DD (°)	ϕ (°)	Spacing(m)	k_n(GPa/m)	k_s(GPa/m)
1	30 (dev. 2)	279 (dev. 2)	25	1.8 (dev. 2)	6.9	0.69
2	70 (dev. 2)	286 (dev. 2)	25	3.0 (dev. 2)	6.9	0.69
3	55 (dev. 2)	147 (dev. 2)	25	3.0 (dev. 2)	6.9	0.69
4	72 (dev. 2)	311 (dev. 2)	30	3.0 (dev. 2)	6.9	0.69
5	55 (dev. 2)	239 (dev. 2)	30	3.0 (dev. 2)	6.9	0.69
6	80 (dev. 2)	54 (dev. 2)	30	3.0 (dev. 2)	6.9	0.69

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.

4. Analysis results and initial support design

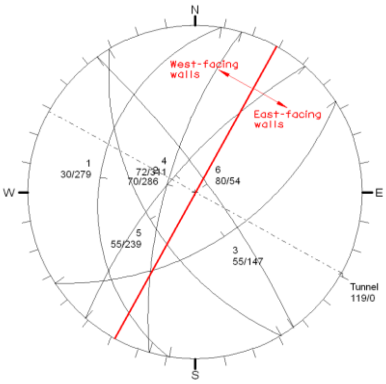
4.1 Kinematic block stability analysis

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.

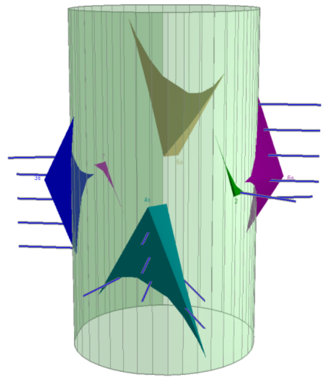
Fig 4 shows kinematic analysis results with typical wedges supported by rock bolts at shaft and cavern. For a minimum factor of safety (FS) of 1.5, the required rock bolt patterns were computed to be 3.6 m (12 ft) long at 1.8 m (6 ft) horizontal and 1.5 m (5 ft) vertical spacing for shaft, and 4.6 m (15 ft) long at 1.5 m (5 ft) horizontal and 1.5 m (5 ft)

Table 4 Rock bolt properties used in the analyses

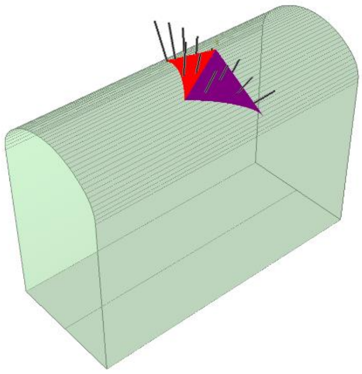
Type	E (GPa)	ν	γ (kg/m^3)	Plate capacity (MN)	Bond strength (MN/m)
#9 rock bolt (Grade 75)	200	0.3	7849	0.1	0.3



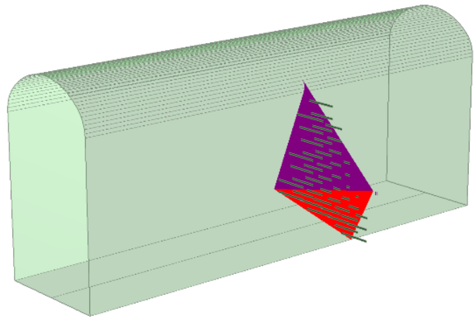
(a) Stereographic plot for discontinuities



(b) Typical wedges at shaft



(c) Typical wedge at cavern crown



(d) Typical wedge at cavern sidewall

Fig. 4 An example of UNSWEDGE analysis results

Fig. 5 An example of 3DEC analysis result for $K_0 = 1.0$ without installation of rock bolts

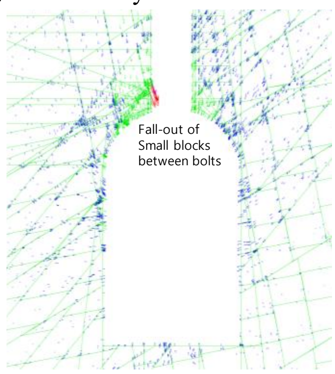
vertical spacing for cavern. It should be noted that the current version of the UNSWEDGE program is not able to model 3-D geometry of intersections accurately. Thus, empirical approaches were considered to determine the required rock bolts at intersections, so that 4.6 m closely spaced long bolts would 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.

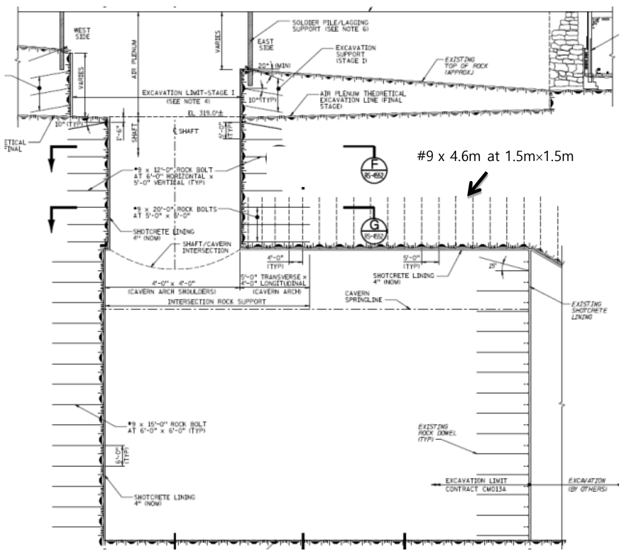
Further analyses were conducted with the installation of the rock bolts designed by the aforementioned kinematic analysis. Fig. 6 shows a 3DEC model of the rock bolt

(a) Rock bolt system installed in 3DEC model

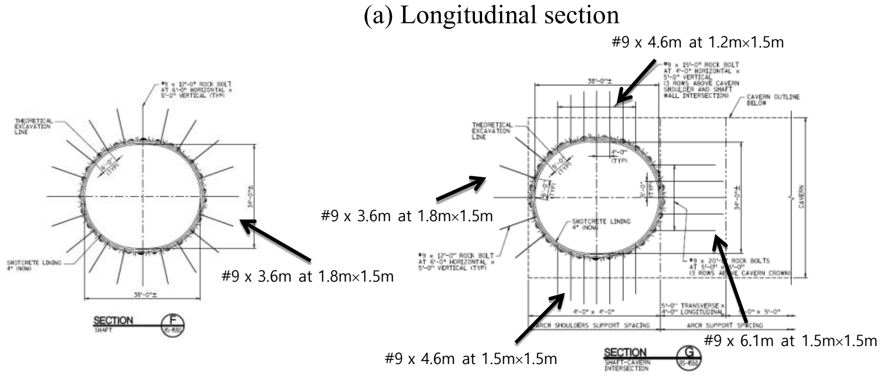


(b) Displacement vectors at cross section along the shaft (K_0 = 1.0)

Fig. 6 3DEC model with rock bolts and an example of simulation results

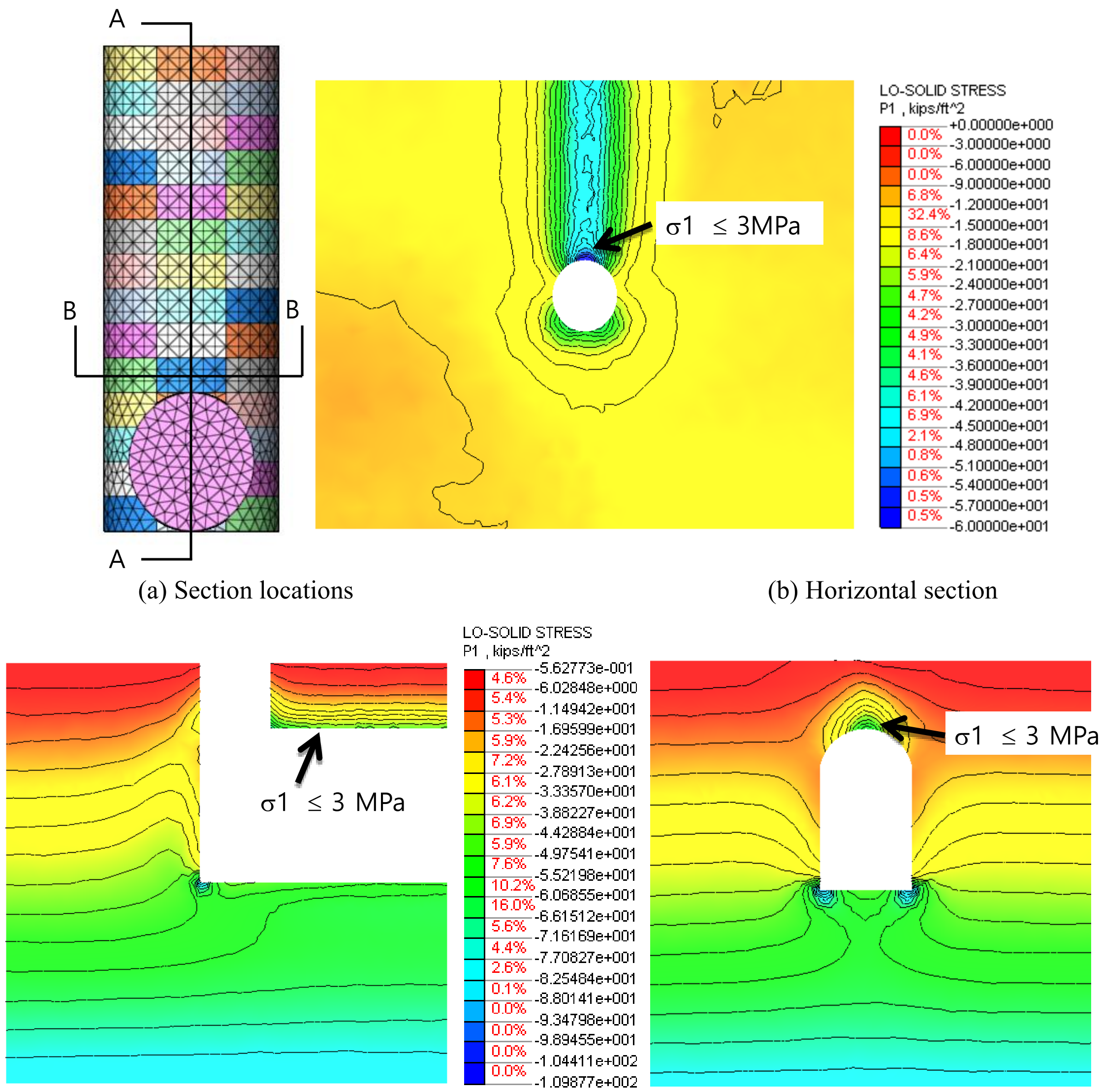


(a) Longitudinal section



(b) Shaft sections

Fig. 7 Proposed rock bolt design at shaft and cavern



(a) Section locations (b) Horizontal section

(c) Section A-A (d) Section B-B

Fig. 8 MIDAS analysis results for K_0 = 2.5 – principal stresses, σ_1 at different sections

system installed at shaft and cavern, and the analysis result for K0 = 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.

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.

5. 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.

One of the challenges tunnel engineers face is how to effectively represent discontinuities or joints in the numerical models as in reality many of discontinuities would not expose. It is particularly important when the excavation is made in a hard rock at shallow depth. Although the current study employs a uniform probability distribution for joint spatial data, more systematic approach would be recommended for taking into account the variability of joint distributions in rock masses. More recently a discrete fracture network (DFN) approach has been developed and used for many geotechnical engineering applications. DFN is a statistic technique that explicitly represents how discontinuities can be spatially distributed in a rock mass on the basis of stochastic analyses to give a range of possible models from field data.

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×Jn) 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

Abdellah, W., Mitri, H, Thibodeau, D., and Moreau-Verlaan, L. (2014), "Stability of mine development intersections-probabilistic analysis approach", *Can. Geotech. J.*, **51**(2), 184-195.

Barla, G. and Barla, M. (2000), "Continuum and discontinuum modelling in tunnel engineering", *Rudarsko Geolosko Naftni Zbornik*, **12**(1), 45-57.

Barla, G. (2016), "Applications of numerical methods", *Proceedings of the EUROCK2016: Tunnelling and Underground Excavations: Recent Trends*, in *Rock Mechanics and Rock Engineering: From the Past to the Future*, Cappadocia, Turkey, August.

Barton, N., Lien, R., and Lunde, J. (1974), "Engineering classification of rock masses for the design of tunnel support", *Rock Mech*., **6**(4), 189-236

Baskerville, C.A. (1994), *Bedrock and Engineering Geologic Maps Of New York County and Parts of Kings and Queens Counties, New York, and Parts of Bergen and Hudson Counties*, New Jersey, U.S. Geological Survey, U.S.A.

Bieniawski, Z.T. (1989), *Engineering Rock Mass Classifications*, Wiley, New York, U.S.A.

Carranza-Torres, C. and Fairhurst, C. (2000), "Application of the convergence-confinement method of tunnel design to rock masses that satisfy the Hoek-Brown failure criterion", *Tunn. Undergr. Sp. Technol*., **15**(2), 187-213.

Chen, M., Yang, S., Zhang, Y. and Zang, C. (2016), "Analysis of the failure mechanism and support technology for the Dongtan deep coal roadway", *Geomech. Eng*., **11**(3), 401-420.

Cui, L., Zheng, J.J., Zhang, R.J. and Lai, H.J. (2015), "A numerical procedure for the fictitious support pressure in the application of the convergence-confinement method for circular tunnel design", *Int. J. Rock Mech. Min. Sci*., **78**, 336-349.

Cording, E.J., Hendron, A.J. and Deere, D.U. (1971), "Rock engineering for underground caverns", *Proceedings of the Symposium on Underground Rock Chambers*, Phoenix, Arizona, U.S.A., January.

Cording, E.J. and Mahar, J.W. (1974), "The effect of natural geologic discontinuities on behavior of rock in tunnels", *Proceedings of the Rapid Excavation and Tunneling Conference*, Las Vegas, Nevada, U.S.A., June.

Fuller, T., Short, L. and Merguerian, C. (1999), "Tracing the St. Nicholas Thrust and Cameron's line through the Bronx NYC", *Proceedings of the Annual Conference on Geology of Long Island and Metropolitan New York*, Stony Brook, New York, U.S.A., April.

Hadjigeorgiou, J. and Karampinos, E. (2017), "Use of predictive numerical models in exploring new reinforcement options for mining drives", *Tunn. Undergr. Sp. Technol*., **67**, 27-38.

Hashash, Y.M.A., Cording, E.J. and Oh, J. (2002), "Analysis of shearing of a rock", *Int. J. Rock Mech. Min. Sci*., **39**(8), 945-957.

Itasca Consulting Group Inc. (2003), *3-Dimensional Distinct Element Code, 3DEC, User's Manual*.

MIDAS Information Technology Co. Ltd. (2007), *MIDAS GTS, User's Manual*.

MTA CC-LIRR (2005), *Geotechnical Baseline Report (GBR), Construction Contract CM0009 Manhattan Tunnels Excavation, ESA Project*.

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

Oh, J., Cording, E.J., and Moon, T. (2015), "A joint shear model incorporating small-scale and large-scale irregularities", *Int. J. Rock Mech. Min. Sci.*, **76**, 78-87.

Lu, Q., Xiao, Z.P., Ji, J. and Zheng, J. (2017), "Reliability based design optimization for a rock tunnel support system with multiple failure modes using response surface method", *Tunn. Undergr. Sp. Technol.*, 70, 1-10.

Rocscience (2011) *UNSWEDGE, User's Manual*.

Sanders, J.E. and Merguerian, C. (1997), "Geologic setting of a cruise from the mouth of the East river to the George Washington Bridge, New York Harbor", *Proceedings of the Field Trip for Participants of the 36th U.S. Rock Mechanics Symposium*, New York, U.S.A., June-July.

Shah, A.N., Wang, J. and Samtani, N.C. (1998), "Geological hazards in the consideration of design and construction activities of the New York area", *Environ. Eng. Geosci.*, **4**(4), 524-533.

U.S. Army Corps of Engineers (1980), *Engineering and Design Rock Reinforcement, EM1110-1-2907*, Department of the Army, U.S. Army Corps of Engineers, Washington, DC 20314-1000, U.S.A.

Wang, T. and Huan, T. (2014), "Anisotropic deformation of a circular tunnel excavated in a rock mass containing sets of ubiquitous joints: theory analysis and numerical modeling", *Rock Mech. Rock Eng.*, **47**(2), 643-657.

Wei, C.H., Zhu, W.C., Yu, Q.L, Xu, T., and Jeon, S. (2015), "Numerical simulation of excavation damaged zone under coupled thermal-mechanical conditions with varying mechanical parameters", *Int. J. Rock Mech. Min. Sci.*, **75**, 169-181.

Yang, X.L. and Li, K.F. (2016), "Roof collapse of shallow tunnel in layered Hoek-Brown rock media", *Geomech. Eng.*, **11**(6), 867-877.

JS