Observations Made During Construction of a Shallow Rock Cavern in New York – A Study of the 7 Line Extension Project

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1. Project Description

The 7 Line Extension project is a 2.4 km long two track subway expansion extending service from the existing Times Square Station at 41st Street and Seventh Avenue out to the west side of Manhattan and terminating at a new station to be located at 34th Street and 11th Avenue.

The centerpiece of the project and main focus of this paper is the 300 m long mined station cavern at 34th Street (Figure 1). The station consists of a two level public area (200 m long) and lower interlocking caverns on either end for track crossovers . Among the numerous technical challenges for construction of the cavern were the following:

- Urban setting
- Less than one span rock cover (typical 14 m for 21 m span)
- Close proximity to active rail lines and historic buildings (min. 8 m to Amtrak tunnel)
- Lack of precedent with regards to cavern construction experience in NYC

A more specific construction challenge was the need to form 6 large penetrations of the cavern side wall that would be left as stub tunnels for future entrance connections or utility adits. The fully excavated cavern has a 21 m span and is18 m high. The construction sequence was a staggered, multiple drift top heading (3 drifts x 50 m^2 each) excavated through the full length of the main cavern, followed by benching and interlock cavern excavation, also using the multiple drift approach.

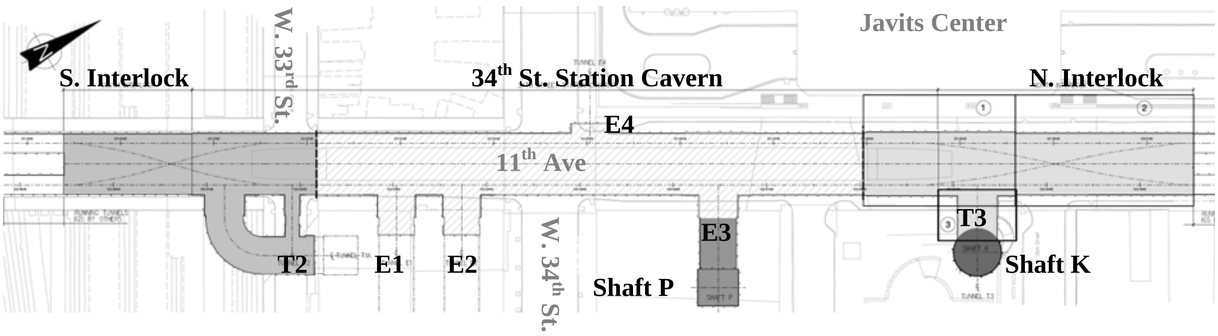

Figure 1: Plan view of 34th Street Station cavern along 11th Avenue showing the the penetrations for future entrance development. Primary construction access was through Shaft P.

2. Rock Mass Characterization

2.1 Geological Overview

Two different rock types were expected to be present along the cavern alignment. A central intrusion of granitoid rock, approximately described as euhedral, crystalline, acidic, and mica deficient with mica schist in a central depression of the intrusion and at the ends of the main cavern with minor pegmatite. The mica schist has distinct, well-defined schistosity that is generally planar and smooth, but can be crenulated or folded. One of the prominent joint sets in the schist is along the foliation planes, striking north and dipping 50° to 80° west.

The contact between the granitic rock and the schist was generally intact to moderately weathered. The southern limb of the intrusion was at the transition from the station to the interlocks, where rock grades back into mica schist. The northern limb was characterized by a faulted contact between the granitic rock and mica schist. The contact was approximately 1 m thick and contained decomposed rock and breccia in a matrix of green, low plasticity clay. The schist at the contact was faulted and sheared with sub-vertical to vertical foliation fractures and seams. The fault was actually sub parallel en echelon features striking obliquely across the excavation bounded by higher quality schist, similar to that found in the southern end of the cavern. The total length of cavern in the poor quality zone was 100 m.

Figure 2: Sub vertical foliation joints of Manhattan Schist as observed in Shaft P.

Figure 3: Persistent granitic joint dipping through right side heading.

2.2 Structural Geology

Two distinct forms of schist were observed during excavation. The first type was present at the TBM starter tunnels and assembly chamber and was highly micaceous with a dominant schistosity and very close foliation joints. Conversely, the schist in the cavern was more blocky, dominated by intersecting discontinuities, containing both discernable foliation joints and wider spaced sub-vertical cross foliation joints, (meeting the term “blocky” ground in the Geological Strength Index (GSI) which was applied to derive geotechnical design parameters). Stability was anticipated to be controlled by horizontal cooling joints in granite and foliation joints in schist. Weathering was intensified along these features by groundwater seepage. During excavation, a vertical joint set striking NE-SW that was identified as a “minor” joint set in the Geotechnical Baseline Report, dipping 70° proved equally important in defining structural instability. This set was found to be much more common in the excavation than indicated in the geotechnical investigation due to the inherent bias created when only using vertical boreholes. With spacings ranging from approximately 25-600mm, this joint set played a significant role in rock mass behavior, in both the granite and schist rock groups. Therefore, if there is any sign of vertical or sub-

vertical joints, it is recommended that a geotechnical investigation should include inclined boreholes with acoustic televiewer data to accurately characterize the vertical joint prevalence, condition and spacing.
Jointing in the granitic rock was generally orthogonal with a minor sub-vertical joint set cross cutting through the mass. Horizontal joints are typically open and clay filled (up to 15 mm). These joints also produce the majority of water inflow into the cavern with estimated flows of up to 10 l/min where infill was washed out.

Despite the more prevelent distribution of vertical jointing found in the rock mass, the systematic classification carried out during excavation found a satisfactory match between the expected and encountered conditions. Consequently, no alterations were required to the designed initial support types.

2.3 Geological Mapping

Assumptions about the condition of a rock mass are inevitable and necessary during the design stage, often because the source of data is limited to 50 mm diameter rock core and lab test results. This is especially true in urban construction where rock outcrops are rare, and those that do exist have been physically weathered and/or chemically altered and disturbed for decades.

If the rock mass classification is based on rock core, the parameters contained within the Q system [1] can typically be derived with higher confidence than those associated with the RMR [2], which requires strength and large scale (persistence) parameters as inputs. There are general correlations between the two classification systems which allows conversion between the two during preliminary empirical design, but the most accurate correlation will be one derived from site specific data. This was the case for this project because geological mapping produced over 250 such classifications, recorded by the same limited number of engineers and geologists which limits subjective variability. The site specific correlations for granitic rock and Manhattan Schist are shown in Figure 4. A case can be made that these correlations are valid based on the fact that the trend line for all the data is approximately equal to the general correlation in Eq. (1) given by Bieniawski [2] which is commonly applied in rock mass classification:

$$RMR = 9\ln Q + 44 \qquad \text{Eq. (1)}$$

It is an unfortunate reality that it is a difficult, if not impossible task to accurately characterize large scale joint properties from borings. If outcrops are unavailable for scanline observation, an educated guess must be made for these parameters, which in most cases leads to a conservative design. However, properties that can only be quantified by mapping, such as joint persistence, large scale waviness, and roughness, are all key input parameters into discontinuum modeling and key block analyses. For example, there will be a drastic difference in modeling results (support requirements) between a model with planar, continuous joints versus one with low persistence (discontinuous) joints, which can only be mobilized by shearing through the intact rock. Likewise, if the amplitude of large scale waviness of the joint is great, dilation is inhibited.

Therefore, improved quantification of joint parameters during the design stage can lead to an optimized, cost effective support scheme through more realistic numerical models. In order to verify the design, joints were characterized whenever possible by observation and profile gauge. Large scale waviness was calculated by taking the ratio of maximum amplitude to wavelength over the visible trace length. The Joint Roughness Coefficient (JRC), as determined from profile gauge measurement and compared to widely published charts [3], was used to estimate joint roughness (Jr) from tables prepared by Barton [4].

Over 120 joints comprise the database. Tables 1 through 3 give statistics for the schist and granite joints. The schist joints have been subdivided into foliation and cross foliation types. All of the measured characteristics can be used to improve the joint shear strength models and produce more realistic representations of structural geology to be used in discontinuum numerical models. These values, when combined with direct shear test data, can give a detailed description of rock joint characteristics for future design of tunnels in New York.

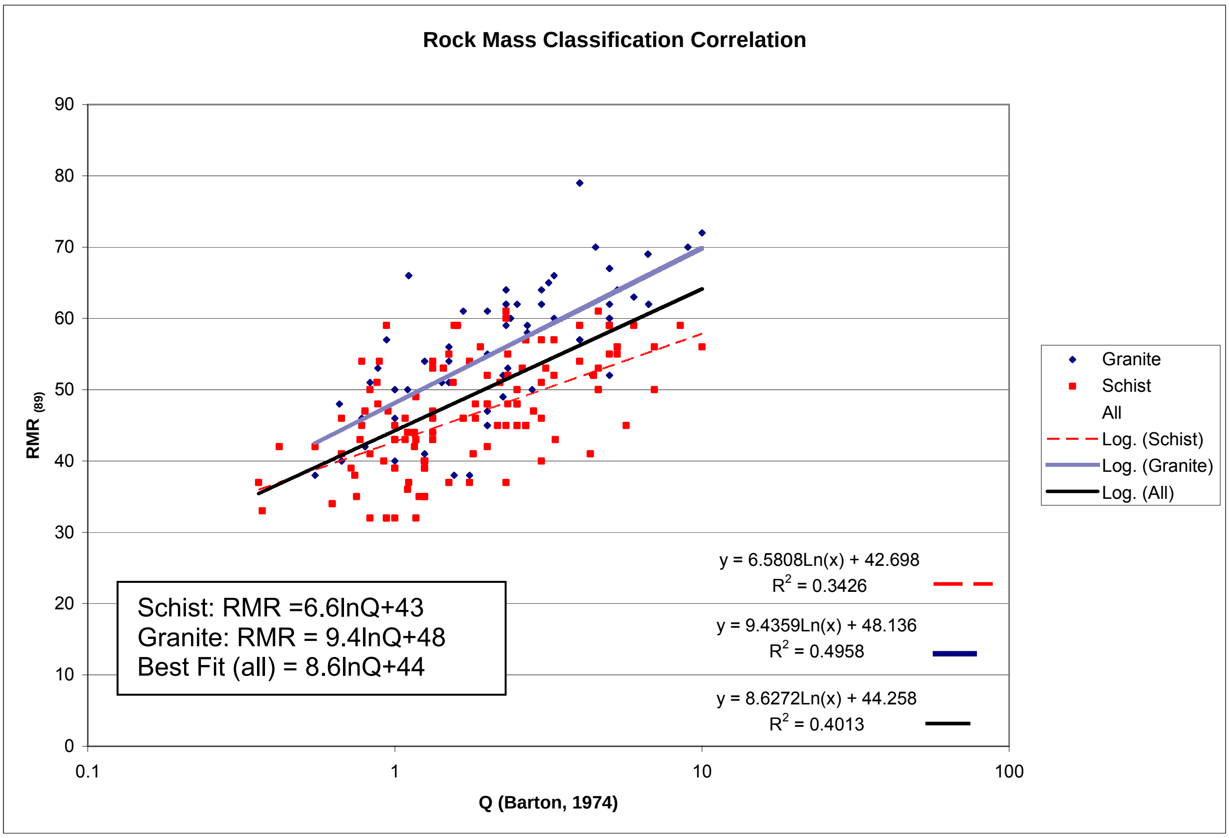

Figure 4: Site specific rock mass classification correlation for 7 Line Project.

Table 1: Summary of Manhattan Schist foliation joint characteristics

Persistence of joints (m)	< 1	1-3	3-6	6-9	>9	
% per total surveyed	0	27	59	14	0	
JRC	2-4	4-6	6-8	8-10	10-12	>12
%	6	28	25	22	14	6
Jr	0.5	1	1.5	2	3	4
%	0	3	75	11	11	0
i°	0-2	2-4	4-6	6-8	8-10	>10
%	17	31	28	19	3	3
Nature of infill	none	surface staining	non-cohesive	clay		
%	96	0	4	0		
Large scale waviness	none	<.01	.01-.02	>.02		
%	42	0	0	58		

Table 2: Summary of Manhattan Schist cross-foliation joint characteristics

Persistence of joints (m)	< 1	1-3	3-6	6-9	>9	
% per total surveyed	0	9	82	9	0	
JRC	2-4	4-6	6-8	8-10	10-12	>12
%	0	17	25	17	33	8
Jr	0.5	1	1.5	2	3	4
%	0	0	50	42	8	0

i°	0-2	2-4	4-6	6-8	8-10	>10
%	0	42	17	17	0	25
Nature of infill	none	surface staining	non-cohesive	clay		
%	97	0	3	0		
Large scale waviness	none	<.01	.01-.02	>.02		
%	30	0	10	60		

Table 3: Summary of granitic rock joint characteristics

Persistence of joints (m)	< 1	1-3	3-6	6-9	>9	
% per total surveyed	0	10	22	51	17	
Joint Type	sub-horz	sub-vert	vert			
%	17	35	48			
JRC	2-4	4-6	6-8	8-10	10-12	>12
%	2	17	25	25	22	8
Jr	0.5	1	1.5	2	3	4
%	0	2	63	27	7	2
i°	0-2	2-4	4-6	6-8	8-10	>10
%	17	39	27	7	7	3
Nature of infill	none	surface stain.	non-cohesive	clay	mineralization	
%	32	42	2	19	5	
Large scale waviness	none	<.01	.01-.02	>.02		
%	28	15	18	38		

3. Rock Mass Behavior

3.1 Longitudinal Deformation

The concept of longitudinal deformation, or relaxation, and subsequent stress redistribution caused by formation of a plastic zone ahead of an advancing tunnel face is well documented [5]. Since then, numerous papers have been written on the empirical and hypothetical shape of the longitudinal displacement profile for an advancing circular tunnel at a constant rate, i.e. TBM tunneling. These profiles are then applied in 2D numerical models to account for the 3D face effects through methods such as convergence-confinement. The relaxation factor, λ, is defined in Eq. (2) by the ratio of the amount of radial deformation that takes place in the ground prior to arrival of the face to the total amount of radial deformation that takes place well behind the face:

$$\lambda = \mu_r(o)/\ \mu_r(\infty) \qquad \text{Eq. (2)}$$

For TBM tunneling, which takes place at a more or less constant rate, a smooth longitudinal deformation curve is assumed as the stress redistribution reaches a quasi steady state during excavation. This is a valid assumption for continuous excavation, but not for cyclic drill and blast tunneling. This aspect was explored by observing ground relaxation in the 3 top heading drifts, as well as their interaction with each other. The method of excavation is directly responsible for the stress path that the ground is subjected to. The stress path in turn will determine the extent of the plastic zone around the face and hence the deformation. For the 7 Line project, the situation is further complicated by having a staggered, multiple drift construction sequence in which the plastic yield zone ahead of and around the drifts will interact with each other and cause further deformations.

Multi-point extensometer arrays installed from the surface in advance of construction were monitored by a real time data acquisition system which allowed movement to be correlated with the blasting cycle. The general layout of an array is shown in Figure 5. For the central heading, the relaxation factor was calculated by taking the ratio of movement occurring in the instrument up to the point of face arrival to the subsequent movement recorded up to the point when Drift 2 arrives at the instrument. This value is not quite "pure" relaxation in the sense that bolting is typically completed 1D behind the face, resulting in slightly higher λ values (i.e. less movement measured due to support installation). Nevertheless, the values obtained give a good indication of the range and magnitude of relaxation that could be applied in a staged numerical model. Similar observations were made for Drifts 2 and 3. A summary of the findings is presented in Table 4. The average distance of the instrument from tunnel face at first response was also recorded for each drift to give an indication of the extent of the plastic zone ahead of the face.

The general trend of the data suggests that the proportion of movement that occurs ahead of the face is over half of the final value reached in each of the drifts, i.e. greater than 50% relaxation. This is because the sudden strain release caused by blasting produces more plastic damage in the rock mass than mechanical excavation methods, which minimize the disturbed zone. Applied in a model, this would lead to larger deformations but smaller loads on the support. The difference in increasing the bolt pattern spacing by even 0.3 m could have significant cost savings over the length of a 300 m cavern.

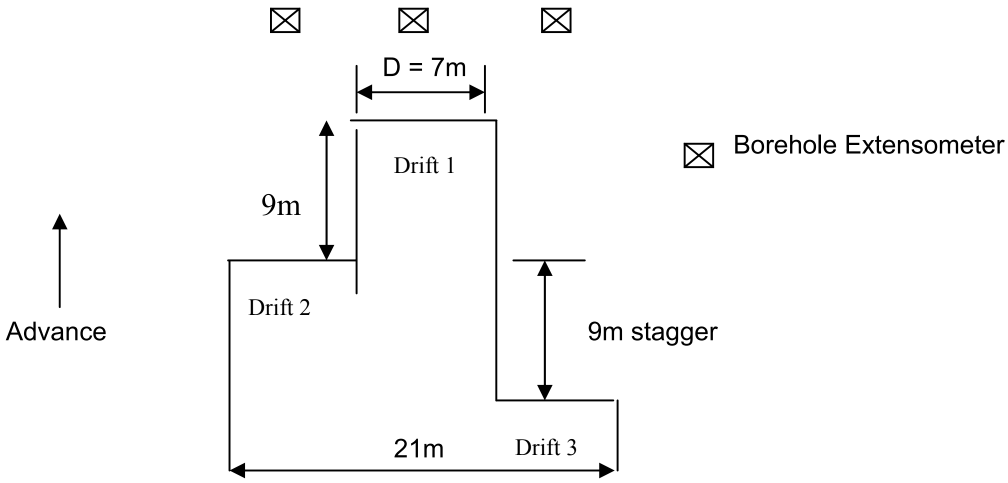

Figure 5: Plan of cavern top heading construction sequence and instrumentation arrangement.

Table 4: Summary of drift relaxation in cavern top heading

Drift	No. of Measurements	Ave. First Response [D = 7 m]	Ave. Relaxation Ratio [λ ± S.D.]
1	5	1.2D	0.70 ± 0.28
2	2	1.0D	0.71 ± 0.17
3	4	0.7D	0.66 ± 0.07

3.2 Junction Formation

Junction design is commonly carried out in several ways. The first is an empirical approach using the Q system (namely J_n x 3). The increase in joint number is to account for the addition of a third dimension, formed by the intersection, along which the potential for kinematic wedge failure is increased. The second way is to utilize a structural beam-spring model to design the thickness of the shotcrete. This requires an estimate of rock load on the lining and does not account for any rock-structure interaction (i.e. no arching effects). Shotcrete capacity is usually designed to keep combinations of moment and thrust within the elastic envelope, neglecting the post cracking benefits of steel fibers. Both methods are typically

Figure 6: T3 junction with main cavern and north interlocking cavern being advanced in the background.

conservative. Complex 3D models can be useful, but are time consuming to build and often difficult to interpret. Another question is how far to extend the additional reinforcement around either side of the penetration. One adit diameter is a typical rule of thumb value used during design.

Ground movements around three of the penetrations with similar size and rock mass classification excavated through the cavern sidewall were studied. Note that the magnitude of cavern crown deformations recorded were on the order of 5-10mm . Table 5 summarizes the results.

Table 5: Rock mass behavior in junction areas of 7 Line main cavern

Adit	Span Ratio [D_{Adit} / D_{Cavern}]	Rock Mass Classification [Q / RMR_{89}]		$\Delta\delta_m/\delta_{mo}$ [%]	$\Delta\delta_a/\delta_{ao}$ [%]
		Cavern	Adit		
E1	0.6	1.3 / 47	1.3 / 44	N/A	172
E2	0.6	1.0 / 39	2.7 / 49	8	280
T3	0.6	1.0 / 43	0.7 / 46	13	93

where : $\Delta\delta_m$ = additional roof settlement of main cavern due to excavation of adit
δ_{mo} = roof settlement of main cavern prior to adit excavation
$\Delta\delta_a$ = additional roof settlement over junction point due to excavation of adit
δ_{ao} = settlement over junction point prior to adit excavation

The adits were blasted only after the main cavern top heading had been fully excavated and supported. Consistent roof movement in the cavern of less than 15% additional strain (compared to cavern movement prior to junction excavation) was observed in E2 and T3 junction construction, both of which were in Manhattan Schist. The E1 extensometer was damaged during blasting, so no reading was possible.

The extent of the deformation, or influence, zone around the junctions was smaller than assumed during design. Extensometers located 4.5m offset from the edge of the adit showed no response during excavation. This corresponds to a zone of influence of less than $0.5D_{Adit}$ either side of the penetration. In addition, extensometers located on the far side of the cavern (opposite the adit) showed no response to adit construction. Therefore in typical Manhattan rock mass conditions, the increase in shotcrete thickness can be restricted to one or two meters around the penetration corners for local containment of wedges without needing to be extended across the entire cavern span, keeping the design cost effective.

The effect of pre-support, which was installed around the T3 adit prior to breaking into Shaft K, was also investigated. This junction was located in the faulted schist characteristic of the northern end of the cavern. The pre-support consisted of horizontal 4.6m long Ø32mm tensioned Dywidag threadbar bolts spaced at 450mm centers, 120° around the periphery of the adit. The purpose of the bolts was twofold. First, they acted as traditional spiles in supporting and promoting arching within the blocky and disturbed rock mass. Secondly, the 9 tonnes of tension imparted to the bolts served to stiffen the collar area by providing confining stress along the minimum principal stress direction, which enhanced rock mass strength.

The positive effect of the pre-support bolts is evident in the final column of Table 5. Movement at the junction was restricted to nearly half of what was observed above the other two adits, which were both located in higher quality rock masses. Therefore, pre-support should be considered for junction construction whenever the rock mass is classified as seamy or disturbed, and especially when limiting ground movement is a key design criterion.

4. Conclusion

The successful construction of the 7 Line project has provided a unique opportunity to verify critical design assumptions and carry out rock mass behavior observations to refine shallow cavern design methodology in New York City. A site specific rock mass classification correlation has been proposed based on geological mapping to improve estimation of supoprt during the design stages. Rock mass deformations around junctions have been shown to be localized and not require heavy additional initial support that is typically specified. The work also proves the effectivenss that pre-supoprt measures have in controlling deformations. Pre-support should be considered for junction construction whenever the rock mass is classified as seamy or disturbed, and especially when limiting ground movement is a key design criterion.

With several caverns in the planning, design and bid stages in similar geological conditions, the information gained during construction can be used for design of cost effective construction sequences and intitial support that can be tailored to the unique combination of rock mass and in situ stress conditions found beneath New York and adapted to similar environments elsewhere.

5. References

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