



FINAL DRAFT Technical Memorandum #06 - Rev A Cavern Final Lining Loads

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Rail Alignment (D2 Subway)

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1 EXECUTIVE SUMMARY

This technical memorandum summarizes the results of an analysis of preliminary rock load estimates required for the final lining design of station caverns for the DART D2 subway project. The project general objective is to construct the stations using mining methods that are less disruptive to the dense urban environment of Downtown Dallas than the cut and cover methodology. The purpose of this memorandum is to develop a preliminary assessment of rock loads that would likely be imparted on the station final linings permanently supporting the underground cavern excavations of the current 10% South of Swiss Alignment of March 8, 2019 for use during the 20% design development, using industry-accepted methodologies to estimate rock support loads.

This analysis has been carried out based on the following published methods of rock load assessment: Karl Terzaghi, (1946); Deere, (1970); Rose (1982); Cording, (1972); Bieniawski, (1973); Barton, (1974); and Carter, (1992). The current alignment profile has been evaluated with respect to the construction of arched center platform stations, binocular stations, and low profile arched crossover caverns. *It is important to emphasize that the anticipated rock cover at CBD East station cavern and Metro Center station cavern (on the previous LPA alignment) was insufficient to allow practical application of the underground mining methods; the top of rail elevation and the station orientation dictates that construction be by cut and cover methods —this fact has a significant impact on results of this analysis.* This impact includes the following considerations:

- For the original LPA alignment, arched station caverns with a mezzanine level over their train rooms and platforms were considered, but subsequently were found to be impractical due to limited cover. Shallower binocular shaped stations are more appropriate for the given ground conditions and track alignment; the cavern lining loads, however, would still approach the full overburden loads from ground surface to the station crown due to generally shallow profile of the LPA.

- Even with application of binocular cavern configuration, the crowns of the CBD East station cavern and the Metro Center station cavern will be near the top of rock elevations and therefore would lack sufficient rock cover for practical application of Sequential Excavation Method (SEM) construction techniques in rock.
- The construction of mined caverns at CBD East station and the Metro Center station would require sizable pre-support and possibly ground improvement application to maintain stability of the opening and manage impacts of SEM mining on the overlying streets, utilities, traffic, and adjacent and overlying structures; such construction would imply larger costs and risks and would likely require relocation of a seven-foot diameter storm sewer tunnel.

For the LPA alignment, the Commerce station cavern and crossover cavern would have been constructed in rock, but the crown stability would have been difficult to maintain and would introduce significant risks during the construction; this is because the crown rock cover requires substantially more thickness than was available, based on the anticipated rock quality and the cavern geometry. Consequently, the LPA alignment was lowered to increase rock cover and this resulted in a modified 10% South of Swiss Alignment.

Therefore, due to the previous elevation of the original LPA alignment, and the resulting extremely shallow cover conditions, *the rock loads for these caverns must approach full overburden loads—a very inefficient and impractical approach for implementation of underground mining methods.* This subject is further explained in TM #08 Assessment of Minimum Rock Cover Over Station Crowns, *which also concludes that the original LPA alignment should have been lowered if application of mining methods and reduction of construction impacts on streets, utilities, traffic, overlying and adjacent structures, communities and businesses, are the main project objectives.* Alternatives to lowering the LPA alignment include construction of shallower cut and cover stations, geometries of which were conducive to the present profile of the current LPA.

Consequently, the 10% South of Swiss Alignment, March 8, 2019, included a lower elevation of the Commerce Street Station to mitigate the problems discussed in this memorandum.

2 INTRODUCTION

This technical memorandum summarizes the results of an analysis of preliminary rock load estimates for the design of final linings of station caverns for the DART D2 subway project.

Rock load estimates must be derived from expected geotechnical rock conditions and related historical cavern excavation rock load data. Three methods available for rock engineers to assess ground loads are empirical, finite and discrete numerical modelling, and stress strain relationships. For this report, empirical methods and historical cavern lining load data are employed.

The product of this analysis is a set of expected minimum and maximum cavern loads to be used for the design of the underground structures of the alignment for the DART D2 project. One of the conclusions of this analysis was that the original LPA, due to its shallow nature, required that the previous track elevation at Commerce Station be lowered. Consequently, the elevation of Commerce Station was lowered in the 10% South of Swiss Alignment, March 6, 2019. This subject is further explained in TM #08 Assessment of Minimum Rock Cover Over Station Crowns.

2.1 Assumptions

This memorandum has been prepared using the following assumptions and inputs:

- The project alignment is as provided on March 8, 2019 (an updated alignment will be issued by the end of July 2019)
- The project alignment includes consideration of 9 existing adjacent buildings and their foundations (as of July 2019 the effort to identify affected subsurface structures and foundations along the alignment corridor is still undergoing)
- Ground conditions are based on data presented in the February 28, 2019 Draft Geotechnical Data Report prepared by Alliance Geotechnical Group (as of July 15, 2019, the Final Geotechnical Data Report is still pending)
- Commerce Station location is between STA 71+13.15 and STA 77+38.15 (in July 2019 it is expected that the station location will be adjusted to the west by approximately 350 feet as part of an updated alignment that would be issued by the end of July 2019).

3 METHODOLOGY

3.1 Definition of Rock Load

Rock load is the amount of weight from the overlying and lateral rock mass exerted upon the lining system for a given cavern excavation. Rock load is required as an input parameter in the design of the shotcrete initial linings (Desai, et. al., 2007) and for cavern final linings. In shallow excavations that require stiff support and lack a sufficient crown pillar stability to develop arching action in the rock mass, the rock load can be expected to approach 80% to 100% of the in-place vertical stress under the most severe geotechnical conditions. The goal of the design process is to verify the chosen ground support system, which is based on empirical rules and is subject to subsequent verification with numerical and structural calculations along the cavern alignment.

3.2 Available Methods of Rock Load Estimation

Four currently available rock mechanics-based approaches for estimating rock loads for station caverns were considered.

Good engineering practice dictates that design of a cavern excavation should ideally be based upon multiple methods to establish rock loads. Cavern design features represented by large, shallow, non-circular openings, jointed rock masses, random shear zones, and variable rock covers required a robust design procedure to define the broad range of ground support conditions that might be encountered. (Desai, et. al., 2005) During preliminary design, such a methodology may provide a safety margin to overcome the lack of available geotechnical information by incorporating multiple paths of analysis;

- Empirical Design Methods
- Numerical Modeling (2D and 3D, Continuum and Discontinuum)
- Close Form Solutions
- Rock Wedge Analysis

Each method is used independently and given equal importance during the design process although they differ in the range of their applicability. None of the methods is considered an inferior or superior analytical tool, but the methods are applied in appropriate circumstances. (Desai, et. al., 2007)

While this memorandum focuses on empirical design methods, the three other analysis methods can be considered in conjunction with the empirical method during final design to evaluate tunnel loads. Acknowledging the differences in these methods, they can still be used for comparative assessments of the reasonableness of the results provided by empirical methods.

3.3 Empirical Design Methods

There are a number of empirical design methodologies that have been developed over the years for use in design of rock tunnels and caverns. As will be described herein, these empirical methods form the basis for the current rock load estimation analysis.

Empirical design methodologies are based on descriptions of the anticipated geotechnical conditions of the rock mass and tunnel geometry. They are easy to use and are portable, meaning they can be used to compare underground conditions between one project and another. Several of these, including the methods of Terzaghi, Bieniawski, and Barton have been used over many years to compare thousands of individual tunneling reaches in widely varying geotechnical environments. The portability and ease of these systems are well-suited for estimating rock loads during the conceptual stage of a cavern excavation project, such as the current phase of the DART D2 project.

3.4 Numerical Methods (2D and 3D, Continuum and Discontinuum)

The numerical analysis method is one of the most useful tools to evaluate ground-structure interaction precisely and estimate rock loads. Complex geometry and multi-stage construction sequences can be practically modelled considering various types of material models and support types, etc. The available numerical methods include Finite Element Method (FEM) and Finite Difference Method (FDM), which are continuum-based models as shown in **Figure 3-1**. The Discrete Element Method (DEM) is based on dis-continuum analysis, which is able to approximate the effects of discontinuities within the rock mass directly, as shown in **Figure 3-2**.

FIGURE 3-1. THREE-DIMENSIONAL FEM ANALYSIS

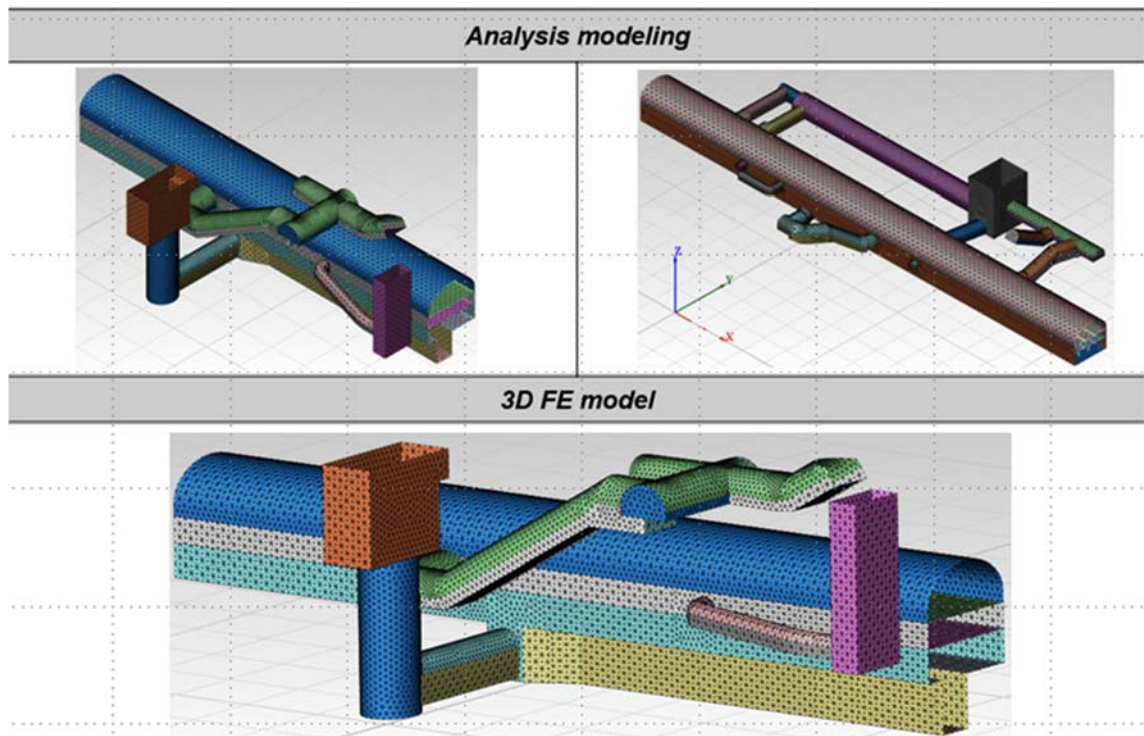
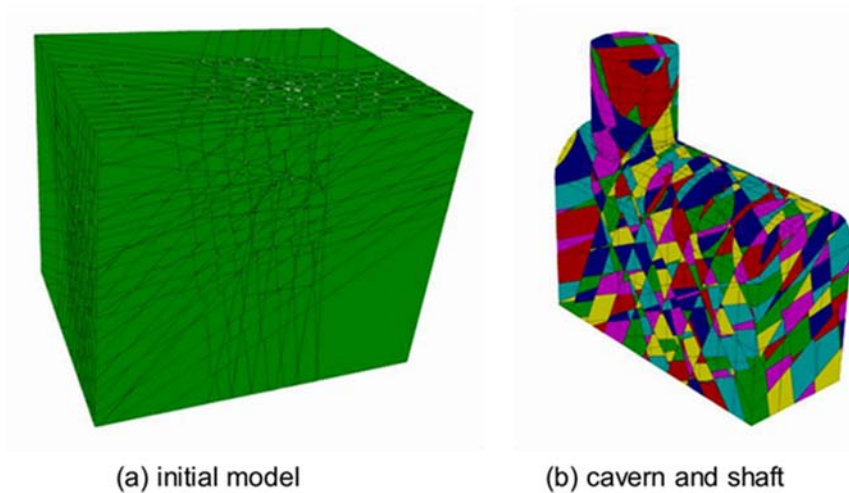


FIGURE 3-2. THREE-DIMENSIONAL DEM ANALYSIS

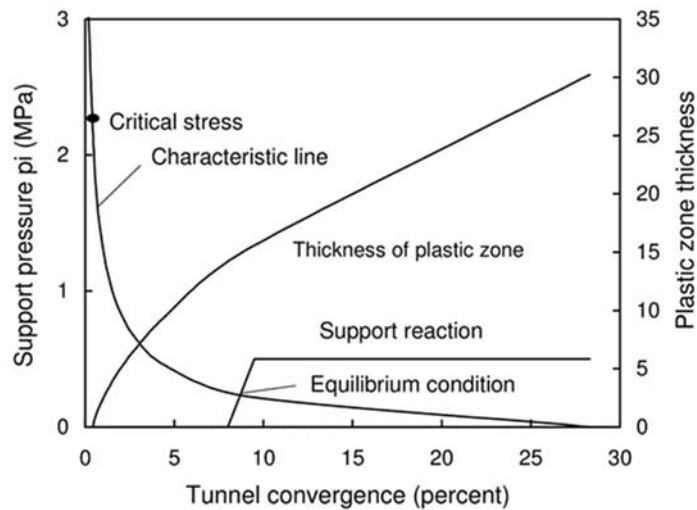
If the geotechnical conditions are clearly known to be homogeneous, the FEM/FDM analysis methods, where the domain is assumed to be a homogeneous medium, can be used extensively for evaluation of underground excavation design problems. To account for the presence of known discontinuities such as systematic joints or fault zones, the rock mass properties and joint properties are determined from measurements on intact core samples. The available material constitutive models include conventional Mohr-Coulomb, Hoek and Brown, and others. One key challenge of the numerical method is to determine ground movement before applying temporary support or permanent structure. To address this, the numerical model should be calibrated by means of ground relaxation scheme based on comparison with the empirical and/or closed form solutions.

Typically, numerical modeling via a modeling program such as 3-DEC has been performed during the final engineering phase to verify ground support systems around penetrations upon further clarification and/or confirmation of geotechnical conditions.

3.5 Closed Form Solutions

The state of stress due to tunnel excavation can be calculated from analytic closed-form solutions. The interaction between support and surrounding ground is described by the ground reaction curve as shown in **Figure 3-3**, which relates internal support pressure to tunnel convergence (Hoek et al., 1995).

FIGURE 3-3. GROUND REACTION CURVES (INTERACTION BETWEEN SUPPORT PRESSURE AND LINING DISPLACEMENT) (HOEK, ET. AL., 1995)



The available analytic solutions for support stiffness and pressure, and analytic solution for ground-liner interaction are summarized in **Table 3-1** and **Table 3-2**, respectively. The closed-form solution is restricted to simple geometries and material models, and therefore often has limited practical value. However, the solution is considered as an appropriate tool for providing a reference check of the results obtained from the numerical analysis.

TABLE 3-1. ANALYTIC SOLUTIONS FOR SUPPORT STIFFNESS AND MAXIMUM SUPPORT PRESSURE FOR VARIOUS SUPPORT SYSTEMS (BRADY & BROWN, 1985)

Support System	Support stiffness (K) and maximum support pressure (P_{max})
Concrete /Shotcrete lining	$K = \frac{E_c [r_i^2 - (r_i - t_c)^2]}{(1 + \nu_c) [(1 - 2\nu_c)r_i^2 + (r_i - t_c)^2]}$ $P_{max} = \frac{\sigma_{cc}}{2} \left[1 - \frac{(r_i - t_c)^2}{r_i^2} \right]$
Blocked steel sets	$\frac{1}{K} = \frac{Sr_i}{E_s A_s} + \frac{Sr_i^3}{E_s I_s} \left[\frac{\theta(\theta + \sin \theta \cos \theta)}{2 \sin^2 \theta} \right] + \frac{2S\theta_B}{E_B W^2}$ $P_{max} = \frac{3A_s I_s \sigma_{ys}}{2Sr_i \theta \{3I_s + XA_s [r_i - (t_B + 0.5X)](1 - \cos \theta)\}}$
UngROUTED mechanically or chemically anchored rock bolts or cables	$\frac{1}{K} = \frac{s_c s_l}{r_i} \left(\frac{4l}{\pi d_b^2 E_b} + Q \right)$ $P_{max} = \frac{T_{bf}}{s_c s_l}$

NOTATION: K = support stiffness; P_{max} = maximum support pressure; E_c = Young's modulus of concrete; t_c = lining thickness; r_i = internal tunnel radius; σ_{cc} = uniaxial compressive strength of concrete or shotcrete; W = flange width of steel set and side length of square block; X = depth of section of steel set; A_s = cross section area of steel set; I_s = second moment of area of steel set; E_s = Young's modulus of steel; σ_{ys} = yield strength of steel; S = steel set spacing along the tunnel axis; θ = half angle between blocking points in radians; t_B = thickness of block; E_B = Young's modulus of block material; l = free bolt or cable length; d_b = bolt diameter or equivalent cable diameter; E_b = Young's modulus of bolt or cable; T_{bf} = ultimate failure load in pull-out test; s_c = circumferential bolt spacing; s_l = longitudinal bolt spacing; Q = load-deformation constant for anchor and head.

TABLE 3-2. ANALYTIC CLOSED-FORM SOLUTION FOR GROUND-LINER INTERACTION

Analytical Solutions	Thrust	Moment
Excavation full slip	Crown = $[0.5 \cdot (1+k_0) \cdot (1-\alpha_0) - (0.5 \cdot (1-k_0) \cdot (1-2 \cdot \alpha_2))] \cdot (\gamma_m \cdot h \cdot d/2)$	Moment at Springline/Crown = $\pm 0.5 \cdot (1-k_0) \cdot (1-2 \cdot \alpha_2) \cdot (\gamma_m \cdot h \cdot (d/2)^2)$
	Springline = $[0.5 \cdot (1+k_0) \cdot (1-\alpha_0) + (0.5 \cdot (1-k_0) \cdot (1-2 \cdot \alpha_2))] \cdot (\gamma_m \cdot h \cdot d/2)$	
Excavation no slip	Crown = $[0.5 \cdot (1+k_0) \cdot (1-\alpha_0) - (0.5 \cdot (1-k_0) \cdot (1+2 \cdot \alpha_4))] \cdot (\gamma_m \cdot h \cdot d/2)$	Moment at Springline/Crown = $\pm [0.25 \cdot (1-k_0) \cdot (1-2 \cdot h+2 \cdot b_2) \cdot (\gamma_m \cdot h \cdot (d/2)^2)]$
	Springline = $[0.5 \cdot (1+k_0) \cdot (1-\alpha_0) + (0.5 \cdot (1-k_0) \cdot (1+2 \cdot \alpha_4))] \cdot (\gamma_m \cdot h \cdot d/2)$	
Einstein & Schwartz (1979)	$\alpha_0 = C \cdot F \cdot (1-\nu_m) / (F+C+(C \cdot F \cdot (1-\nu_m)))$	$b_2 = [C \cdot (1-\nu_m)] / [2 \cdot (C \cdot (1-\nu_m) + 4 \cdot \nu_m - 6 \cdot \beta - 3 \cdot \beta \cdot C \cdot (1-\nu_m))]$
	$\alpha_2 = (F+6) \cdot (1-\nu_m) / (2 \cdot F \cdot (1-\nu_m) + 6 \cdot (5-6 \cdot \nu_m))$	$C = [(d/2) \cdot E_m \cdot (1-\nu_L^2)] / [E_L \cdot (A_L/w_L) \cdot (1-\nu_m^2)]$
	$\alpha_4 = \beta \cdot b_2$	$F = (d/2)^3 \cdot E_m \cdot (1-\nu_L^2) / (E_L \cdot I \cdot (1-\nu_m^2))$
	$\beta = ((F+6) \cdot (1-\nu_m) \cdot C + (2 \cdot F \cdot \nu_m)) / (3 \cdot F + 3 \cdot C + 2 \cdot C \cdot F \cdot (1-\nu_m))$	
Muir-Wood (1975)	Thrust at Crown = $1/3 \cdot (\sigma_v - \sigma_H) \cdot (d/2) + (4/3) \cdot \lambda \cdot (\text{deflection}) / (d/2) + (\sigma_v - \sigma_H) \cdot (d/2)$	Moment at Springline/Crown = $\pm (\sigma_v - \sigma_H) / 6^2 \cdot (d/2)^2 \cdot \eta^2 \cdot R_s / (1+R_s)$
	Thrust at Springline = $2/3 \cdot k_0 \cdot (\sigma_v - \sigma_H) \cdot (d/2) + (2/3) \cdot \lambda \cdot (\text{deflection}) \cdot (d/2) + (\sigma_H) \cdot (d/2)$	
	$k = (1-\nu_m) / ((1-2 \cdot \nu_m) \cdot (1+\nu_m))$	$R_s = (9 \cdot EI) / (\lambda \cdot \eta^2 \cdot (d/2)^4)$
	$\beta = (E_m/E_L) \cdot (d/2 \cdot \eta) / (A_L/w_L)$	$\sigma_H = \sigma_v \cdot k_0 + h_w \cdot \gamma_w \cdot (1-k_0)$
Excavation Full Slip	$\eta = (\phi_D + d) / (d/2)$	$\sigma_v = \gamma_m \cdot h + S$
	$\phi_D = (d/2 - t_L) \cdot 2$	

NOTATION:

 ν_m : Poisson's ration for ground ν_L : Poisson's ration for Liner E_m : Young's Modulus for ground E_L : Young's Modulus for Liner t_L : Thickness of Liner w_L : Width of Liner A_L : Cross-Sectional Area of Liner γ_m : Ground Unit Weight γ_w : Water Unit Weight d : Diameter of Tunnel I : Moment Inertia of Liner per Unit Length C : Compressibility Ratio (measure of the extensional stiffness of the medium relative to the liner) F : Flexibility Ratio (measure of the flexural stiffness of the medium relative to the liner) k_0 : Coefficient of Lat. Earth Pressure h : Depth to Springline h_w : Depth from Water Table to Springline R_s : Stiffness Factor S : Surcharge σ_H : Horizontal Stress σ_v : Vertical Stress

3.6 Rock Wedge Analysis

One of the major concerns for tunnels and caverns in jointed rock masses is the kinematic failure of discrete blocks in the excavation area, i.e., rock blocks become loose and fall out of the rock matrix, destabilizing the rock mass. (Desai, et. al., 2007). This type of instability can lead to progressive rock failure which can destabilize the crown pillar. Therefore, the local stability of the rock mass between the rock support necessary for the global stability is checked using UNWEDGE. Based on rock mass discontinuity data (dip and dip direction) and excavation direction, the maximum geometrically formable blocks between the rock bolts are calculated. The rock wedges are scaled in terms of excavation area, i.e. no larger than the bolt spacing of the surrounding rock support. Using gravity loading and hydrostatic water pressure, the factor of safety ($FS_{min}=1.5$) is then calculated for the local stability of the rock blocks between the rock bolts.

3.7 Selected Method for this Analysis

At the current level of preliminary engineering, which is 10% design, limited boreholes are available. The cavern design process has not commenced. Therefore, this analysis was based on empirical design methods exclusively. As the design progresses, a cavern design methodology will be set up which analyzes each reach of each cavern separately. During final engineering the subsequent methodologies of numerical modeling, closed-form solutions, and rock wedge analysis should be incorporated and compared to verify the developed cavern design.

4 EMPIRICAL CAVERN ROCK LOAD ESTIMATION METHODS

Project-specific geotechnical data may be limited during early stages of design, during which the designer faces the need to select appropriate mining methods that affect the ultimate structural support that will need to be provided for the cavern excavation under long-term conditions as well as the expected mining production rate. Exploration programs must provide an accurate assessment of these conditions. Nevertheless, it is difficult to correlate the results of exploration with the ultimate support requirements. Significant geotechnical conditions are subtle and difficult to interpret from the boring program, especially when the available number of borings are limited.

Rock load is required as an input parameter in the design of subway station cavern linings. (Desai, et. al., 2007). Specifically, rock load summarizes the impacts of the ground conditions, cavern geometry, and represents a convenient measure of ground behavior. Rock load is first estimated by rules of thumb and then verified by empirical, numerical modeling (2D and 3D, Continuum and Discontinuum), stress strain relationships, and rock wedge analysis methods. The magnitude of the rock load carried by the support is

determined by the support stiffness and the rock arching capacity to support the hoop stresses generated by the excavation. In shallow excavations that require stiff support, the rock load has been shown based upon measured cavern load data to range from 13% to 61% of the overburden depth from the cavern crown to the ground surface after a comprehensive rock bolting program. (Cording, et. al., Vol 1., 1983)

Empirical design methods have been developed to assess the stability of underground cavern excavations by the use of statistical analysis of observed behavior of underground structures subjected to rock loads. The engineering rock mass classification systems are the best known empirical approach for estimating geotechnical loads in rock, which will ultimately be expressed as final lining loads. Although numerical techniques have advanced significantly in rock mechanics engineering, empirical design approaches are still preferable for a wide spectrum of design applications.

Empirical rock mass classification systems provide a systematic approach to correlate observed (or predictable) underground geotechnical conditions with observable behaviors on past projects in rock such as arching action, standup time, deformation, support requirements, probability of failure, and of course, rock loads. These classifications are intended to be used as guides, and in conjunction with analytical studies, field observations and measurements, rather than solely as ultimate design solutions. However, the available databases are substantial, and the systems have been developed to a point where their track records are very reliable.

Several rock mass classification methods for estimating rock loads are summarized as follows;

- Equivalent Rock Load, Karl Terzaghi, (1946)
- Deere, et. al., (1970)
- Rose (1982)
- University of Illinois at Urbana-Champaign, Cording, et. al., (1972)
- Geomechanics System, Penn State University, Bieniawski, (1973)
- Rock Tunneling Quality Index, Norwegian Geotechnical Institute, Barton, et. al. (1974)
- Scaled Crown Span, Golder Associates, Carter, (1992).

These are the most relevant rock mass rating and rock load estimating systems for designing mined excavations in rock. A general estimate of the time periods during which these systems have been used is provided in **Table 4-1**.

TABLE 4-1. APPROXIMATE TIME PERIODS FOR ROCK MASS RATING SYSTEMS

Approximate Date Ranges

	Rating System	Approximate Time Period	Use of System
1	Terzaghi	1946 to 1970	Used for estimation of rock loads and design of steel sets.
2	Deere	1970 to 1982	Used for estimation of rock loads for cavern excavations.
3	Rose	1982 to Present	Used for estimation of rock loads for cavern excavations.
4	Cording	1972 to 1983	Used for estimation of rock loads for underground caverns.
5	Bieniawski	1073 to Present	Used for standup time, initial lining, and comparison of geotechnical conditions between tunnel projects.
6	Barton	1974 to Present	Used for initial and final linings, rock loads, and comparison of geotechnical conditions between tunnel projects.
7	Carter	1992 to Present	Used for statistical analysis of crown pillar failures.

4.1 Equivalent Rock Load, Karl Terzaghi, (1946)

The Terzaghi Rock Load method was developed by Karl Terzaghi in 1946, for its use in designing steel sets for rock tunnel supports in widely varying rock conditions. (Proctor and White, 1946). This is a method of predicting the degree to which an arching action is developed over the crown of the tunnel due to prescribed geotechnical conditions. The equivalent rock load is the predicted weight of the broken ground resulting from the excavation of the tunnel. This “was a landmark paper in tunneling literature, and for many years it provided the basis for the rational design of tunnels, particularly those constructed in North America. There are still many valuable lessons to be learned from this work, and it is recommended reading for anyone seriously interested in the practical aspects of tunnel design and construction.” (Hoek, 2000)

One of the important lessons from this method was the correlation of guidelines for estimating the rock load with observable differing geotechnical conditions. More importantly, this method recognized and incorporated a range of ground conditions from essentially stable to immediate collapse. These opposing conditions are represented by a range of equivalent rock loads from 0 in hard intact rock to 250 feet in swelling conditions. Although the steel set method has fallen out of favor by designers due to its high cost, it is still used in North America as a last resort for addressing the toughest ground conditions.

Terzaghi recommended a procedure in 1946 for estimating the rock load that forms on tunnel supports based on varying geotechnical conditions. The term rock load indicates the

height of the mass of rock which tends to drop out of the tunnel crown if no support is provided. (Proctor, et. al., 1946). The conceptual basis of this equivalent rock loading concept is provided in **Figure 4-1** and in **Table 4-2**. The loading concept for crushed rock is shown in **Figure 4-1**. **Table 4-2** provides the guidelines for various rock loads of increasing intensity as experienced by the tunnel lining as the geotechnical conditions become increasingly worse. The rock load is distinguished from the earth pressure as follows;

- Rock load – rock load depends on randomly distributed details such as the spacing and orientation of the rock joints, which may change from point to point.
- Earth pressure – earth pressure depends on the average properties of the material surrounding the tunnel.

This distinction becomes very important in tunneling because the random variations are more difficult to determine by a geotechnical boring program than average properties.

Terzaghi developed models for estimating rock load based on geotechnical conditions as follows;

- Hard intact rock – no rock load develops
- Stratified blocky rock – generally limited to half the cavern diameter, $0.5 B$
- Crushed rock – arching action will limit rock loads to $1.5 (B + H_t)$
- Swelling rock – may be up to 250 feet.

In these equations, B is the cavern width and H_t is the cavern height. Therefore, based on this method, the rock loads for cavern stations assumed 60 feet wide for the D2 project can be expected to range from 0 feet to 30 feet. (0 for Hard Intact Rock to 30 for Hard Stratified Rock, See Table 4-2)

FIGURE 4-1. THE TUNNEL EQUIVALENT ROCK LOAD CONCEPT FOR ARCHING ACTION OF TERZAGHI (PROCTOR, ET. AL., 1946).

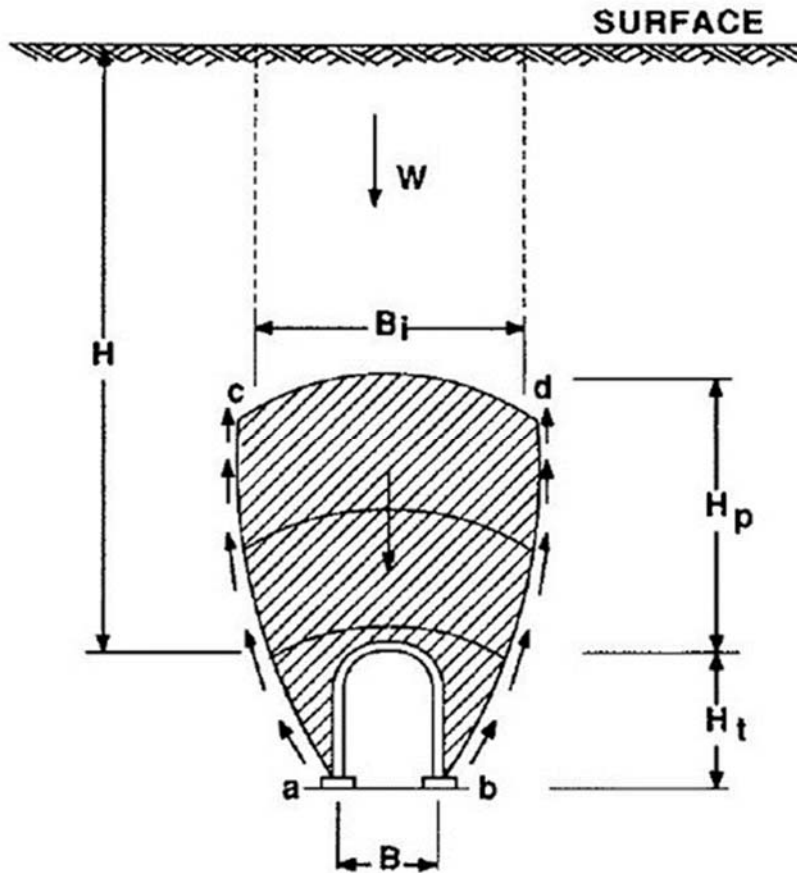


TABLE 4-2. ROCK LOAD H_p IN EQUIVALENT FEET OF ROCK ON ROOF SUPPORTS IN TUNNEL WITH WIDTH B (FEET) AND HEIGHT H_t , (FEET) AT DEPTH OF MORE THAN 1.5 B .

Rock Condition	Rock Load H_t in Feet	Remarks	
1. Hard and Intact	Zero	Light lining, required only if spalling or popping occurs.	Essentially Stable
2. Hard, Stratified, or Schistose	0 to 0.5 B	Light support. Load may change erratically from point to point.	
3. Massive, Moderately Jointed	0 to 0.25 B	Light support. Load may change erratically from point to point.	
4. Moderately Blocky and Seamy	0.25 B to 0.35 ($B + H_t$)	No side pressure.	
5. Very Blocky and Seamy	(0.5 to 1.10) ($B + H_t$)	Little or no side pressure.	↓ Immediate Collapse
6. Completely Crushed but Chemically Intact	1.10 ($B + H_t$)	Considerable side pressure. Softening effect of seepage towards bottom of tunnel requires either continuous support for lower ends of ribs or circular ribs.	
7. Squeezing Rock, Moderate Depth	(1.10 to 2.10) ($B + H_t$)	Heavy side pressure. Invert struts required. Circular ribs recommended.	
8. Squeezing Rock, Great Depth	(2.10 to 4.50) ($B + H_t$)	Heavy side pressure. Invert struts required. Circular ribs recommended.	
9. Swelling Rock	Up to 250 irrespective of ($B + H_t$)	Circular ribs required. In extreme cases use yielding support.	

4.2 Deere, et. al., (1970)

The Terzaghi concept for designing steel sets based on rock loads determined by research and experimentation was used extensively for designing steel sets in the United States. This concept continues to be used today for designing steel sets for heavy ground conditions. This system has experienced two major modifications after 1946. The first update was published by Don Deere, Ralph Peck, Harvey Parker, of University of Illinois, with J. Monsees

and B. Schmidt from the construction industry in 1970. This study was a comprehensive evaluation of the design procedures for tunnel lining systems. A result of this evaluation led to the provision of new guidelines for the selection of support systems for 20-ft to 40-ft diameter tunnels in rock, which are provided in a revised table as shown in **Figure 4-2**. The revised table was based on construction experience and the results of field measurements. Several changes were incorporated in the updated table as follows;

- Recommendations are keyed to rock conditions that are described and quantified by a weighted or modified core recovery, Rock Quality Designation, (RQD)
- Terzaghi rock classifications were correlated with expected RQD ranges
- Rock loads were expressed in terms of excavation width B only rather than $(B + H_t)$
- Steel set spacing recommendations were provided
- Specific guidelines for selection of support systems including rock bolts and shotcrete for 20-ft to 40-ft diameter tunnels in rock are provided in tabular form
- Separate rock loads for mechanical and drill and blast excavation were provided
- Recommended rock load coefficients were reduced for several categories because the rock is not disturbed by blasting.

It should be noted that irrespective of the additions and refinements to the rock load estimation methodology, the range of rock loads remained 0 to 250 feet. However, with the Deere method, the estimation method became tied to numerical values of RQD rather than the qualitative rock descriptions employed by Terzaghi. Based on this system, as expressed in the table below, with the rock loads tied to RQD and mechanical excavation assumed rather than drill and blast, the estimated rock loads for similar DART D2 caverns 60 feet wide would be as follows;

- | | | |
|---------------------------|----------|------|
| • RQD = 90 to 100 percent | 0 to 12 | feet |
| • RQD = 75 to 90 percent | 0 to 24 | feet |
| • RQD = 50 to 75 percent | 24 to 60 | feet |

This equation should be used with caution because the data set is expressly used for caverns up to only 40 feet width.

4.3 Rose (1982)

The Terzaghi rock load method as modified by Deere, was further modified by Rose in 1982. The resulting system is provided in **Table 4-3**. The paper published by Rose was primarily concerned with reducing the cost of tunnel supports in tunneling practice in the United States, which continued to use steel sets after European practice had shifted toward shotcrete linings. Rose cited several references concluding that the rock loads for three classes could be lowered to eliminate some of the conservatism that was originally



acknowledged by Terzaghi. As shown in the table, the rock load method Rose published provided a 50% reduction in the estimated rock load for: Moderately Blocky and Seamy, Very Blocky and Seamy, and Completely Crushed but Chemically Intact rock. Rose's recommendation is to use the lower rock loads for rock conditions that are not affected by groundwater.

This method continues to be employed in tunneling projects for tunnel design. A case in point is the design of the final lining for the PR-53 highway tunnels from Maunabo to Yabucoa, Puerto Rico during 2005. The exploration was carried out by horizontal core logging along the tunnel alignment. Rock loads based on the method by Rose were used as input for STADD modeling of the cast-in-place concrete final lining.

Based on the modifications proposed by Rose, the rock load for similar caverns in the DART D2 project would range from 0 to $0.2 (B + H_t)$, which would be 0 to 20 feet for an RQD range from 75 to 100 percent. (0 for Hard Intact Rock to $0.2 * (60 + 38) = 20$ feet for Moderately Blocky and Seamy, See Table 4-3)

FIGURE 4-2. UPDATES TO TERZAGHI ROCK LOAD GUIDELINES, SPECIFICALLY FOR 20 TO 40 FOOT DIAMETER TUNNELS, PUBLISHED BY DEERE, PECK, PARKER, MONSEES, AND SCHMIDT IN 1970. (DEERE, ET. AL., 1970)

Rock Quality	Construction Method	Steel Sets		Rock Bolts ^a (Conditional use in poor and very poor rock)		Shotcrete ^b (Conditional use in poor and very poor rock)	
		Rock Load (B = Tunnel Width)	Weight of Sets	Spacing ^c	Spacing of Pattern Bolts	Additional Requirements and Anchorage Limitations ^a	Total Thickness
							Crown Sides
Excellent ^d RQD > 90	Boring machine	(0.0 to 0.2)/B	Light	None to occasional	None to occasional	Rare	None to occasional local application
	Drilling and blasting	(0.0 to 0.3)/B	Light	None to occasional	None to occasional	Rare	None to occasional local application 2 to 3 in.
Good ^d RQD = 75 to 90	Boring machine	(0.0 to 0.4)/B	Light	Occasional	Occasional	Occasional mesh and straps	Local application
	Drilling and blasting	(0.3 to 0.6)/B	Light	5 to 6 ft	5 to 6 ft	Occasional mesh or straps	Local application 2 to 3 in.
Fair RQD = 50 to 75	Boring machine	(0.4 to 1.0)/B	Light to medium	5 to 6 ft	4 to 6 ft	Mesh and straps as required	2 to 4 in.
	Drilling and blasting	(0.6 to 1.3)/B	Light to medium	4 to 5 ft	3 to 5 ft	Mesh and straps as required	4 in. or more
Poor RQD = 25 to 50	Boring machine	(1.0 to 1.6)/B	Medium circular	3 to 4 ft	3 to 5 ft	Anchorage may be hard to obtain. Considerable mesh and straps required.	4 to 6 in.
	Drilling and blasting	(1.3 to 2.0)/B	Medium to heavy circular	2 to 4 ft	2 to 4 ft	Anchorage may be hard to obtain. Considerable mesh and straps required.	6 in. or more
Very poor RQD < 25 (Excluding squeezing and swelling ground)	Boring machine	(1.6 to 2.2)/B	Medium to heavy circular	2 ft	2 to 4 ft	Anchorage may be impossible. 100 percent mesh and straps required.	6 in. or more on whole section as required
	Drilling and blasting	(2.0 to 2.8)/B	Heavy circular	2 ft	3 ft	Anchorage may be impossible. 100 percent mesh and straps required.	6 in. or more on whole section as required
Very poor, squeezing or swelling ground	Both methods	up to 250 ft	Very heavy circular	2 ft	2 to 3 ft	Anchorage may be impossible. 100 percent mesh and straps required.	6 in. or more on whole section as required



TABLE 4-3. TERZAGHI'S ROCK LOAD CLASSIFICATION CURRENTLY IN USE AS MODIFIED BY DEERE ET. AL. (1970) AND ROSE (1982) SOURCE: (BIENIAWSKI, PAGE 35, 1989)

Rock Condition	RQD	Rock Load Ht in Feet	Remarks
1. Hard and Intact	95-100	Zero	Light lining, required only if spalling or popping occurs.
2. Hard, Stratified, or Schistose	90-99	0 to 0.5 B	Light support. Load may change erratically from point to point.
3. Massive, Moderately Jointed	85-95	0 to 0.25 B	Light support. Load may change erratically from point to point.
4. Moderately Blocky and Seamy	75-85	0.25 B to 0.20 (B + Ht)	No side pressures. Reduced by 50% from Terzaghi values because water table has little effect on rock load.
5. Very Blocky and Seamy	30-75	(0.20 to 0.60) (B + Ht)	Little or no side pressure. Reduced by 50% from Terzaghi values because water table has little effect on rock load.
6. Completely Crushed but Chemically Intact	3-30	(0.6 to 1.10) (B + Ht)	Considerable side pressure. Softening effect of seepage towards bottom of tunnel requires either continuous support for lower ends of ribs or circular ribs. Reduced by 50% from Terzaghi values because water table has little effect on rock load.
6a. Sand and Gravel	0-3	(1.10 to 1.40) (B + Ht)	
7. Squeezing Rock, Moderate Depth	N/A	(1.10 to 2.10) (B + Ht)	Heavy side pressure. Invert struts required. Circular ribs recommended.
8. Squeezing Rock, Great Depth	N/A	(2.10 to 4.50) (B + Ht)	Heavy side pressure. Invert struts required. Circular ribs recommended.
9. Swelling Rock	N/A	Up to 250	Circular ribs required. In extreme cases use yielding support.

Essentially Stable



Immediate Collapse

4.4 University of Illinois at Urbana-Champaign, Cording, et. al., (1972)

During the construction of the Subway Station Caverns for the Washington, D. C. metro system in the 1970's, Ed Cording and others at the University of Illinois carried out an extensive geotechnical study which included observations and measurements of geotechnical data as related to geologic conditions at the sites of nine 59 to 80 foot wide station caverns. (Cording and Mahar, 1974) (Mahar, et.al. 1972) (Cording, et. al., Volume 1, 1983) (Cording, et. al., Volume 3, 1983) The results of these cavern studies indicated that foliation shears and shear zones were the geological features causing the most significant geotechnical rock loads in the underground construction projects. They caused the most serious tunneling difficulties, overbreak, loosening and progressive loosening of rock blocks, and fallouts leading to heavy rock loads, when;

- their strike was within 25° of the tunnel axis
- they are highly continuous
- they were intersected by another joint, shear, or shear zone
- they are bounded by wet slickensided surfaces
- horizontal stresses are low due to proximity to the surface
- continuous, smooth joint planes form large blocks of rock.

Cording, et. al. developed a procedure for estimating rock loads imposed on the structural support by considering the critical wedges surrounding the opening that ultimately require support. These studies produced a method of estimating rock loads based on foliation dip angle which was described by Cording, et. al. (Cording, et. al., Volume 3, Page 11, 1983). The basic concept for calculating rock loads with this method is summarized in **Table 4-4**. As the table indicates, rock blocks are formed by the dip angle of the foliation, bedding planes, or high angle joint planes in the rock mass. Rock loads are directly related to the formation of rock blocks. This system can be used whether the primary geotechnical feature is foliation or bedding planes with high angle joint planes or faults. It is important to note that the range of rock loads conveyed by this method ranges from zero to several 10s of feet and are consistent with measured rock loads in both Washington, D. C. and in New York City. As with the other rock load estimation methodologies being discussed herein, this system produces a wide range of rock loads from zero in the best conditions to over 60 feet for the worst conditions in a 60 foot cavern.

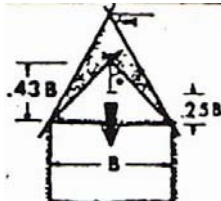
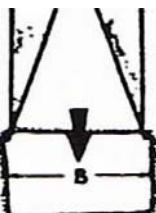
This method, including the formulation in **Table 4-4**, is included herein for two reasons. First, it allows for a more complete description of the history of the available methods for rock load estimation for station caverns. Second, it re-emphasizes that the basic nature of



rock loads is that they can range from zero to many tens of feet depending strictly on the geotechnical conditions of the rock above the cavern.

Assuming that high angle joint planes will be in the range of 55 to 65 degrees, averaging 60 degrees, the use of this method for estimating rock loads on DART D2 caverns would result in a rock load of .43 B, which will be 26 feet for a 60-foot cavern. A complete wedge analysis will be performed during final engineering based upon geotechnical data from a completed project specific boring program.

TABLE 4-4. WEDGE FORMATION CONDITIONS AND ROCK LOADS IN NEW YORK CITY AND WASHINGTON, D. C. SUBWAY STATION CAVERNS (CORDING, ET. AL., VOLUME 3, PAGE 11, 1983)

Dip Angle (Degrees)	Half Angle (Degrees)	Height of Equivalent Rock Load in Feet (m x B)	Minimum Condition for Failure	Remarks	
0 – 30	90 – 60	(0 to 0.15) B	Both planes wavy, offset		Essentially Stable ----- Immediate Collapse
30 - 45	60 – 45	(0.15 to 0.25) B	One plane wavy or offset, One plane smooth to slightly wavy		
45 - 60	45 – 30	(0.25 to 0.43) B	One plane sheared, continuous and planar, One plane slightly wavy		
60 - 75	30 – 15	(0.43 to 1.0) B	Both planes sheared, continuous and planar		
75 - 90	15 – 0	(>1) B	Low lateral stresses in arch, surfaces planar, smooth, possibly open, or progressive failure aided by separation along low angle joints.		

4.5 Geomechanics System, Penn State University, Bieniawski, (1973)

The Geomechanics classification system, or Rock Mass Rating (RMR) was developed by Bieniawski in 1972 and was updated in 1976 to clarify the significance of some of the input parameters. The system is based on 351 case histories in various applications in hard rock mining. The RMR classification is an empirical method of rating relative rock mass qualities for mining and construction activities. Initially, it was intended to represent a structural region of a discontinuous rock mass by providing a single index value. Based on this value, uniform appropriate limitations on the excavation sequence may be correlated with other mining operations. It has been widely used and modified in over 1000 mining and tunneling case histories worldwide.

The RMR value can be used to develop an estimate of the degree of ground support required, expected rock failure modes, estimates of prudent ground stand-up times, and appropriate limitations on excavation sequences.

4.6 Rock Tunneling Quality Index, Norwegian Geotechnical Institute, Barton, et. al. (1974)

The Norwegian Geotechnical Institute's Rock Tunneling Quality index, (Q) system, is a rock mass classification system used worldwide for the design of rock support for civil and mining construction projects. It was first used in hydropower projects in Norway and in a water transfer project in Peru in 1974. (Barton and Grimstad, 2014). The system was developed based on application in Norwegian road tunnels during which hundreds of case studies were examined. This system provides a simple means of communication for geologists, rock engineers, mining engineers and lawyers. The Q system is used, often in combination with the geomechanics system, in thousands of tunneling projects around the world and in all principal mining countries.

The Q-system developed by Nick Barton is an empirical method of predicting probable ground behavior considering discontinuous geotechnical and stress-strength relationships. (Desai, et. al., 2007) As required for the design of support systems, the Q-system has been developed with a view to determining the mechanism and mode of failure in the rock mass based roughly on the block size, inter-block shear strength, and the active stress regime, with the aim of evaluating stability as one of the first steps in designing an underground excavation.

The rock tunneling quality index, Q value is calculated by the following equation:

Rock Tunneling Quality Index:
$$Q = RQD/J_n \times J_r/J_a \times J_w/SRF$$

Where:

RQD = rock quality designation

Jn = joint set number

Jr = joint roughness number

Ja = joint alteration number

Jw = joint water reduction factor

SRF = stress reduction factor.

The Q system provides an evaluation in terms of both rock quality and cavern width. The Q index has been employed to provide a first indication of initial ground support, as well as final support for tunnels and caverns to put these designs on the same page as other tunneling projects worldwide.

The original chart for the NGI – Q method of classifying a rock mass based on its rock quality, Q, is provided in **Figure 4-3**. (Barton, et. al., p. 212, 1974). This chart was developed based on 191 case records for tunnels with rock support installed according to the requirements of the ground. Since the development of this system in 1974, the number of tunneling case studies is now in the thousands.

The 1974 paper on the Q system provided a rock load equation for tunnel ground support as follows; (Barton, et. al., p. 209, 1974)

Roof Pressure: $P_{\text{roof}} = (2 / Jr) * Q^{-1/3}$

Where:

P_{roof} = roof pressure [kg/cm²]

Q = rock tunneling quality index

Jr = joint roughness number

This equation is expressed graphically in **Figure 4-4**.

The Q system was updated in 1993, 2003, and 2007. The 2007 version, published by Grimstad is provided in **Figure 4-5**. Originally developed in 1974, the Q system was updated in 1993 to include steel fiber reinforced shotcrete. This update included an additional sample of 1050 sections of highway tunnels and hydropower tunnels where support was selected by experienced engineers. This 1993 revision of the Q system focused on updated (increased) stress reduction ratios for high stresses in rock. A subsequent update in 2003 was primarily concerned with identifying the requirements for reinforced ribs of shotcrete where the toughness and energy absorption of sprayed shotcrete has been taken into consideration in bad ground conditions where deformation may be expected. These changes were based on hundreds of new tunnel case histories and focused on replacing cast concrete linings with reinforced ribs of shotcrete in the poorest rock quality classes. In 2007, Grimstad published a subsequent revision in which ground support classes 3 and 4 were combined into a single ground support class.



Using the roof pressure equation with an assumed expected joint roughness number of 1.5 would result in the following range of equivalent rock loads for DART D2 caverns, based on Q values as follows;

- Best Case $Q=8.9, J_r=1.5$ $P_{\text{roof}} = 8 \text{ feet}$
- Expected Case $Q=2.4, J_r=1.5$ $P_{\text{roof}} = 12 \text{ feet}$
- Worst Case $Q=0.2, J_r=1.5$ $P_{\text{roof}} = 27 \text{ feet}$

FIGURE 4-3. Q - CHART PUBLISHED IN 1974 (BARTON, ET. AL., 1974)

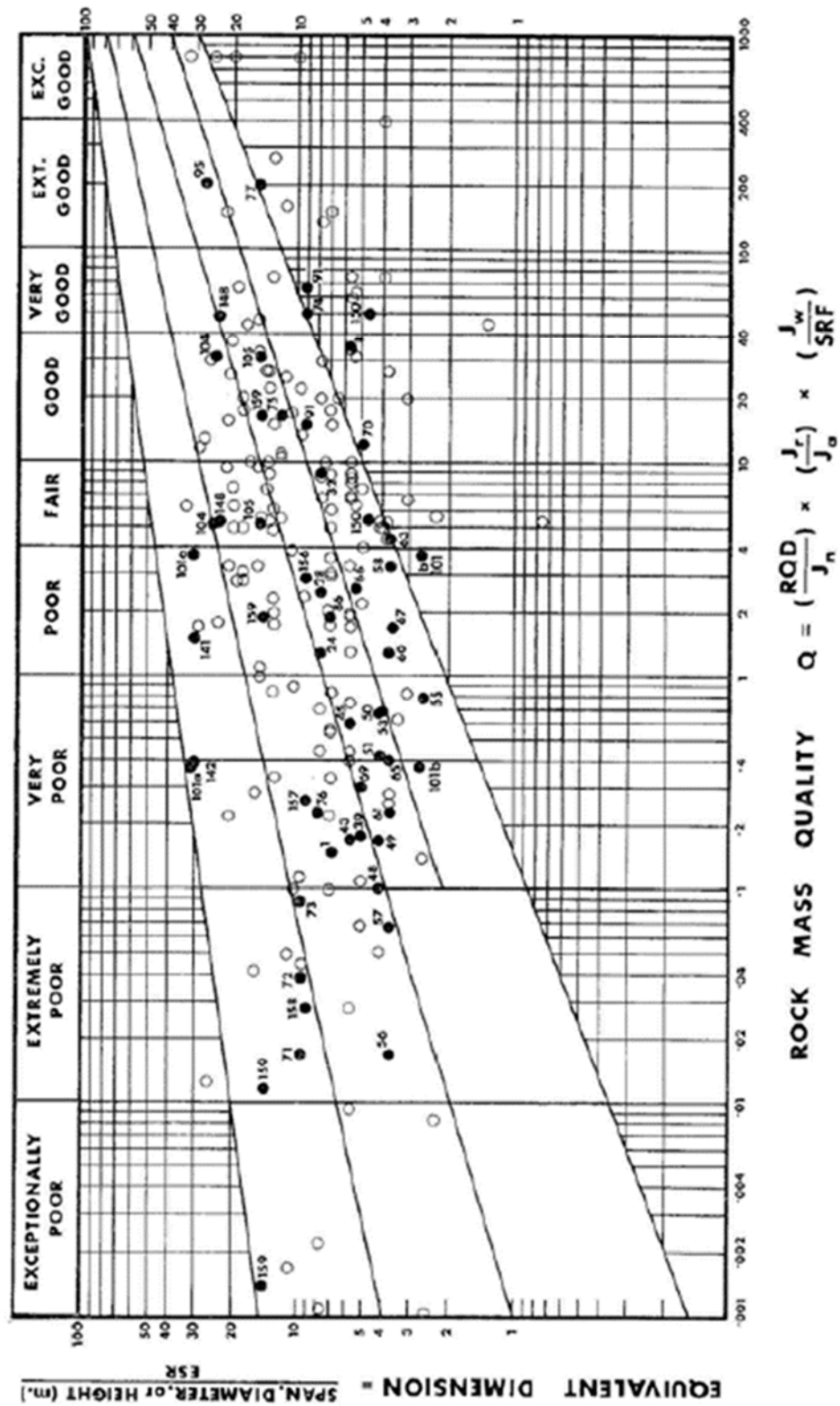




FIGURE 4-4. TUNNEL ROOF PRESSURES AS A FUNCTION OF NGI Q VALUES (BARTON, ET. AL., 1974)

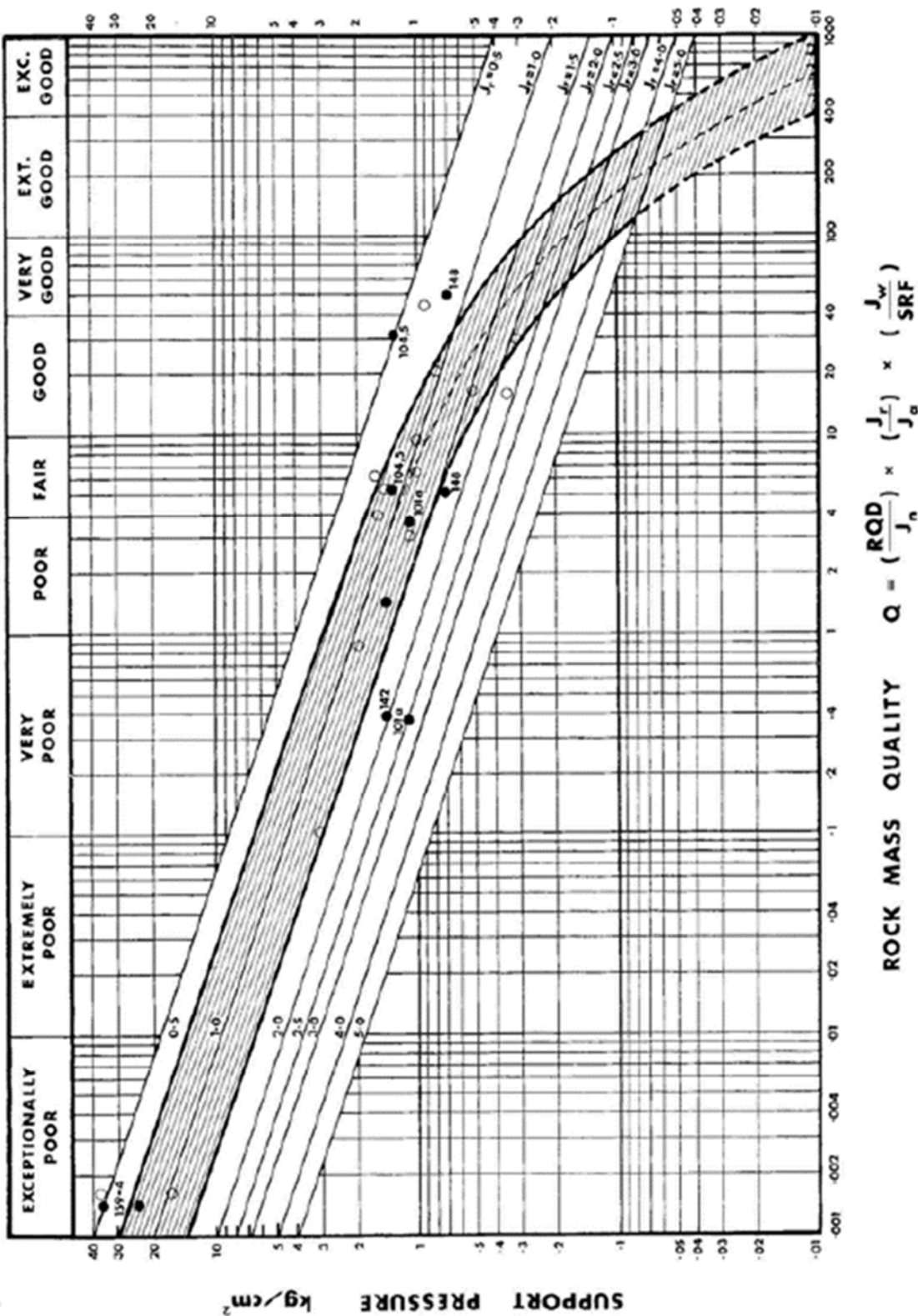
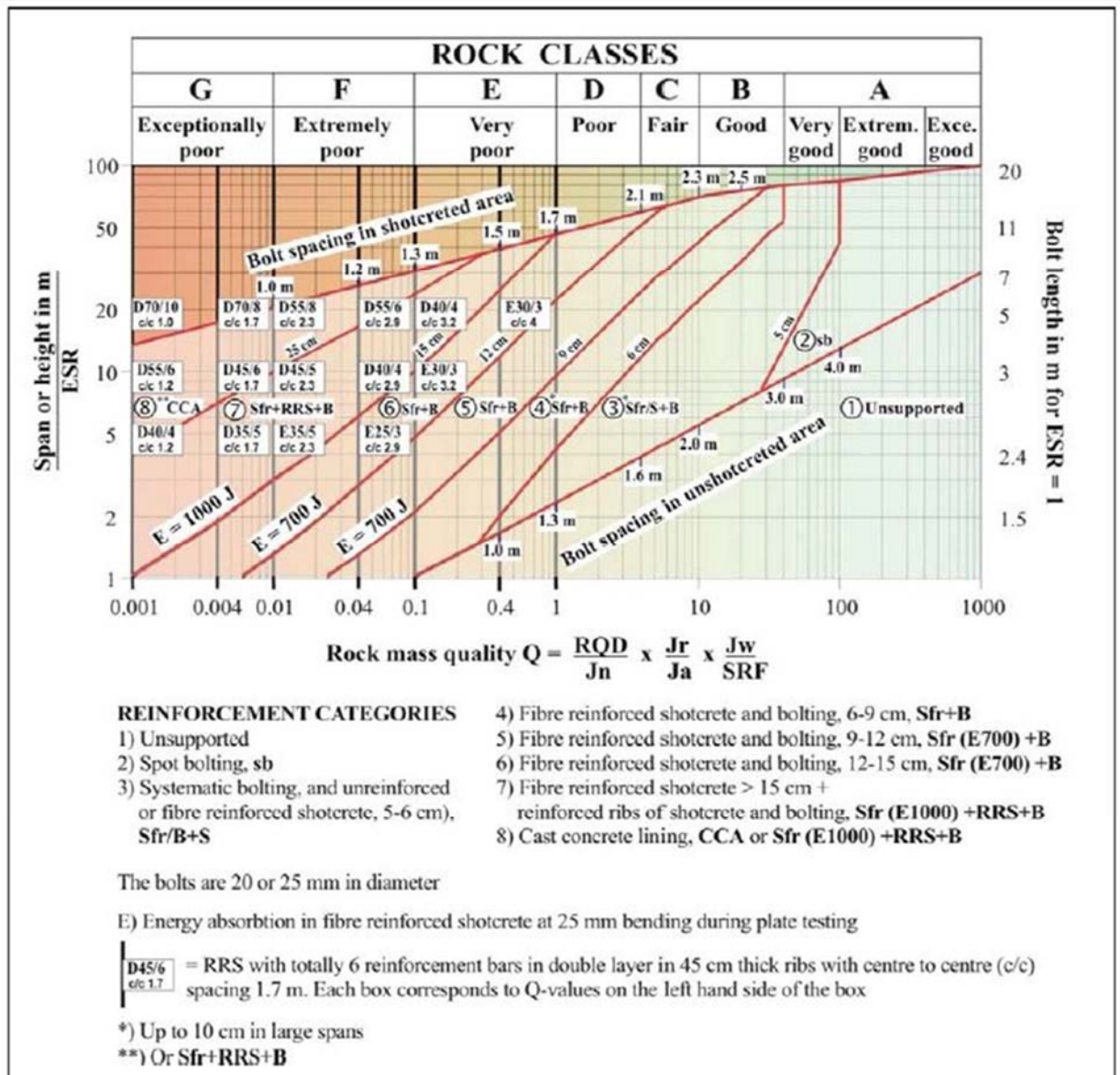


FIGURE 4-5. Q – CHART PUBLISHED IN 2007 (BARTON AND GRIMSTAD, 2014)



4.7 Scaled Crown Span, Golder Associates, Carter, (1992)

The Scaled Crown Span empirical method has been developed by T. G. Carter over a period of two decades. Relevant publications on this method are provided in (Carter, 1992; Carter and Miller, 1995; Carter, 2000, Carter, 2014). This method uses case studies of both stable and unstable crown pillars for various known crown pillar conditions and normalizes these cases with respect to the scaled span of the crown pillars. This method, although general in nature, provides a realistic assessment of the stability of a crown pillar. The scaled crown span method, equations, and utilization in crown pillar assessment is described in detail in Technical Memorandum #08 – Assessment of Minimum Rock Cover Over Station Crowns.

5 HISTORICAL DATA FOR WASHINGTON D.C. SUBWAY STATION CAVERNS

During the construction of the Subway Station Caverns for the Washington, D. C. metro system in the 1970's, Ed Cording and others at the University of Illinois carried out an extensive geotechnical study which included observations and measurements of geotechnical data as related to geologic conditions at the sites of nine 59 to 80 foot wide station caverns. (Cording, et. al., Vol 1, 1983) This extensive study included field observations, displacement measurements, and cavern lining measurements which were correlated with extensive site geological data. As anticipated by all of the empirical rock classification systems, cavern displacements and rock loads on cavern linings tend to increase over a wide range with respect to varying geotechnical conditions.

These caverns were driven with rock reinforcement and steel set-shotcrete structural linings as initial support and experienced an equivalent rock load on the structural lining of 2 to 60 feet after a heavy rock bolting program. Since the initial support served as the final structural lining in this case, these caverns are very well suited to provide an indication of cavern final lining loads in general. Rock loads and rock displacements were highest where the geotechnical conditions included shear zones. Rock loads were lowest, and correlated with elastic theory, where there was an absence of shear zones.

With these caverns serving as a basis for providing actual measured rock loadings on cavern final linings, and the use of the Q and scaled crown span classification systems for correlating the geotechnical conditions with the lining loads, an excellent rock load prediction methodology can be developed. The development of this methodology is the subject of this technical memorandum.

5.1 Station Cavern Data Base

The descriptions of geotechnical conditions for Washington D. C. subway station caverns provided in **Attachment 1** have been condensed from the information provided in the report by Cording, et. al. (Cording, et. al., Vol 1, 1983) A summary table for measured rock loading data from this report is provided as **Figure 5-1**. (Cording, et. al., Vol 1, p. 235, 1983). The cavern construction sequences for the associated rock caverns are included as **Figure 5-2**. (Cording, et. al., Vol 1, p. 27, 1983)

Equivalent rock loadings on the station caverns linings back calculated from strain gage measurements have been estimated by Cording et. al. to be as follows; (Cording, et. al., Vol 3, p. 126, 1983) (Cording, et. al., Vol 1, p. 235, 1983)

- | | | |
|------------------------------|--------------|---------|
| • Medical Center Station | Cavern | 12 feet |
| • Rosslyn Station | Cavern | 16 feet |
| • Bethesda Station | Cavern | 20 feet |
| • Cleveland Park Station | Cavern | 23 feet |
| • Van Ness Station | Cavern | 23 feet |
| • Zoological Park Station | Cavern | 30 feet |
| • Dupont Circle Station | Cavern | 30 feet |
| • Tenley Circle Station | Cavern | 28 feet |
| • Tenley Circle Station | Intersection | 41 feet |
| • Friendship Heights Station | Cavern | 42 feet |
| • Friendship Heights Station | Intersection | 49 feet |

FIGURE 5-1. SUMMARY TABLE OF EQUIVALENT ROCK LOADS ON THE WASHINGTON D. C. STATION CAVERN LININGS AT THE END OF THE CONSTRUCTION PERIOD. (CORDING, ET. AL., VOL 1, p. 235, 1983)

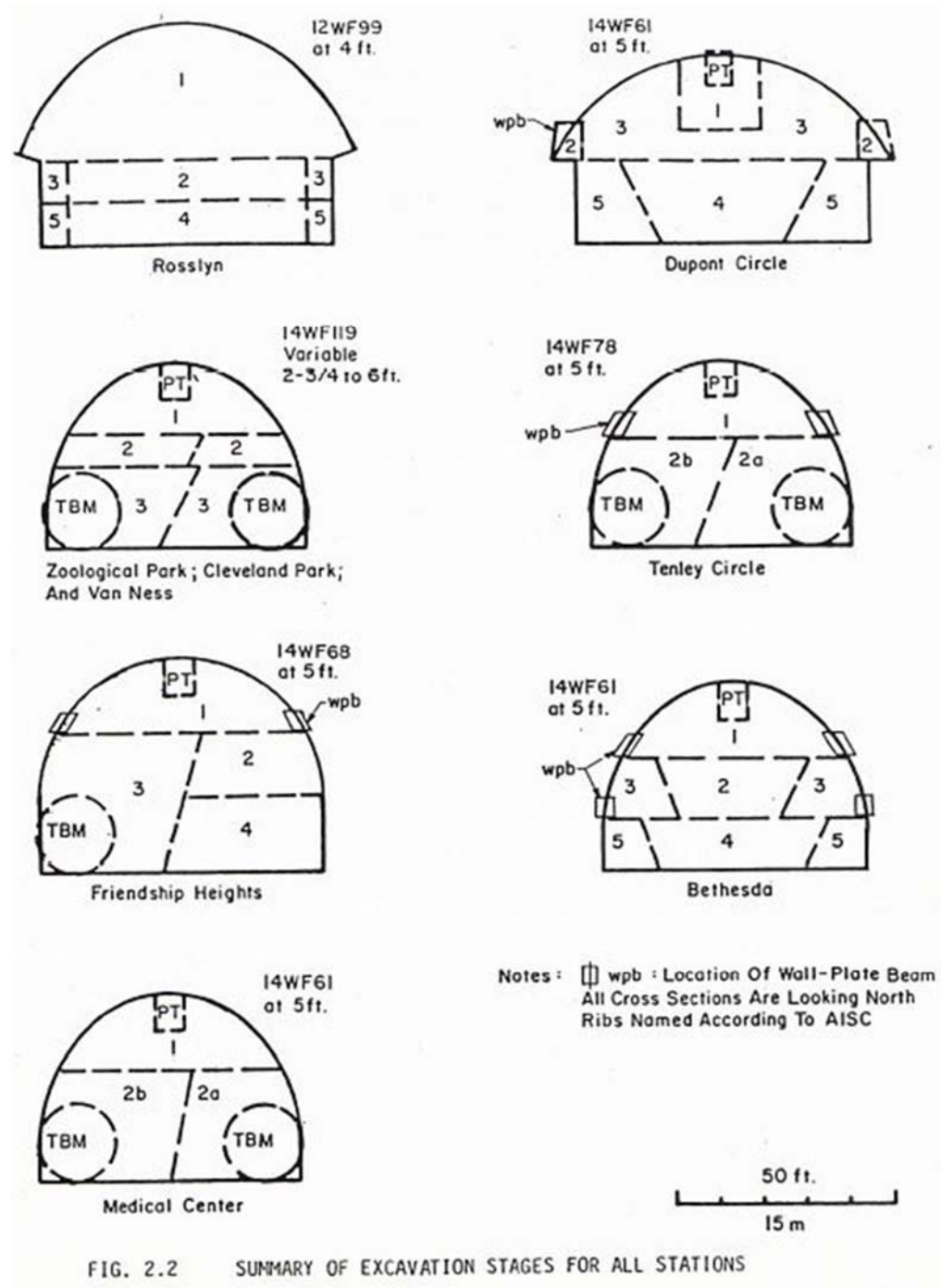
EQUIVALENT ROCK LOADS ON THE LINING AT THE END OF CONSTRUCTION

STATION	H (ft)	t (in.)	A _l (in. ² /ft)	R (ft)	B (ft)	H _t (ft)	H _p (ft)	$\frac{H_p}{B}$	$\frac{H_p}{H}$
ROSSLYN	70	30	43	42	80	55	16	.20	.23
DUPONG	60	30	40	40	76	44	30	.38	.50
ZOO PARK	115	31	50	29	58	44	29	.50	.25 (E)
		31	47	29	58	44	31	.52	.27
CLEVELAND	60	31	48	29	58	44	23	.40	.39
VAN NESS	65	31	45	29	58	44	23	.40	.35
TENLEY	85	35	50	30	60	46	41	.68	.48 (E)
		35	47	30	60	46	28	.47	.33
FRIENDSHIP	80	26	36	33.5	67	48	49	.73	.61 (E)
		26	35	33.5	67	48	42	.63	.53
BETHESDA	100	31	44	31	62	47	20	.32	.20 (E)
		31	44	31	62	47	18	.29	.18
M. CENTER	90	31	44	31	62	47	12	.19	.13 (C)
		31	41	31	62	47	12	.19	.13

Notes: H : total overburden above the crown (ft)
t : assumed thickness of finished lining (in.)
A_l : equivalent area of lining, in.² per linear foot, for the transformed cross section
R : average radius of curvature of the lining (ft)
B : width of the chamber (ft)
H_t : height of the chamber (ft)
H_p : height of equivalent uniform rock load on the liner (ft)
E : Entranceway

1 in. = 2.54 cm
1 ft = 30.48 cm

FIGURE 5-2. EXCAVATION SEQUENCES FOR ROCK CAVERNS ON THE WASHINGTON D. C. STATION CAVERNS. (CORDING, ET. AL., VOL 1, P. 27, 1983)



5.2 Contrast of Geotechnical Conditions Between Washington Caverns and DART D2 Caverns

It is important to note the differences in the geotechnical conditions between the Washington D. C. subway caverns and the DART D2 subway caverns. Such differences apply to the geology, intact rock, and rock mass properties. The contrasts in these two widely differing rock types is provided in **Table 5-1**.

TABLE 5-1. CONTRAST OF GEOTECHNICAL CONDITIONS

Characteristic	Washington Caverns	DART D2 Caverns
Rock Type	Metamorphic	Sedimentary
Specific Gravity	Dense	Less Dense
Compressive Strength	High Strength	Low to Moderate Strength
Horizontal Stress	Compressive	Extensional
Stratification	Foliation	Massive Bedded
Discontinuities	Numerous pronounced brittle and ductile fault zones, along and across foliation.	Recent normal faults oblique to bedding and joint sets.
Dip Angle	Foliation dips >45 degrees	Horizontal bedding dips <5 degrees

Bieniawski has stated that considering the main design approaches to the design of underground structures, rock mass classifications form an integral part of the most predominant design approach, the empirical design methods. (Bieniawski, 1989) One of the objectives of rock mass classification is to relate the experience of rock conditions at one site to the conditions and experience encountered at others. It has furthermore been shown in the last two decades that both the RMR system and the Q system have been used on thousands of rock mass classification rated tunnel intervals to describe the rock conditions in terms of a single index value, based on the prevailing geotechnical conditions, and relate those conditions to mining performance and rock behavior. The very high number of rock mass classification rated tunnel intervals in all types of rock conditions that have been reported in numerous geotechnical reports allows the RMR and Q systems to be used interchangeably on various projects of extremely varying rock characteristics.

5.3 Estimated Q and Cs Parameters for this Analysis Based on Geotechnical Data

The estimated Q parameters assumed for the Dallas DART D2 Subway Station Caverns are included in **Table A-1**, which has been included as **Attachment 2**. The rationale for these assumptions is explained in a subsequent section.

Based on the geotechnical conditions for the subway station caverns in Washington, D. C., as discussed above, and included in **Attachment 1**, the estimated Q and Cs parameters for the subway station caverns with measured rock loadings are calculated and summarized in **Table A-2**, which is included as **Attachment 3**. The data from WMATA cavern excavations has been used to correlate geotechnical conditions with measured rock loads. The geotechnical and cavern geometry data is reduced in the analysis spreadsheet of **Table A-2**, which is subsequently used to estimate rock loads for DART D2 caverns. The data tabulated in **Table A-2** for the Washington Caverns is used to calculate rock loads for various geotechnical conditions, which is subsequently used to determine estimated rock loads for the DART D2 caverns. This is an empirical method for assessing crown pillar stability conditions which is made possible by the portability of the rock mass classification systems, both Q and RMR, which were developed to compare widely varying underground conditions in thousands of tunnel intervals.

This data is plotted graphically in **Figure A-1**, which is included as **Attachment 4**. The resulting chart of rock loads has the rock tunneling quality index, Q, plotted on the x-axis and the estimated scaled crown span plotted on the y-axis. The locations for the various station caverns are plotted with visual symbols of decreasing intensity as shown on the figure. What is important to note is that the caverns with the largest measured rock loads are concentrated at the lower end of the quality scale. Likewise, the caverns with the lower measured rock loads are plotted at the higher end of the rock quality scale.

5.4 Rock Load Estimation Method

The measured data from the Washington Caverns plots approximately with the rock quality index as explained above. The trend of data is for low rock loads at the higher quality end of the scale to progress to heavy rock loads at the lower end of the quality scale. Note that the station with the highest rock quality, Q=3, for Medical Center Station has the lowest measured rock load at 12 feet. On the other end, the station with the lowest rock quality, Q=.03, for Friendship Heights station has the highest measured rock loading at 49 feet. The progression of measured data supports an increasing trend of rock loads with decreasing rock quality. Also note that the dispersion of rock data is very wide. The trend in the data is not smooth due to the widely varying nature of geotechnical conditions, and the extreme difficulty of getting smooth measured data with the excavation sequences as variable as those shown in **Figure 5-2** above.

As a predictive method, therefore, this method is not adequate to predict the actual values of measured rock loads. However, this method can be used to provide a good estimate of design loads for future caverns if the estimated loads are set at the high end of the variance in data. To do this, the rock load for the 100% probability of failure can be set at 40 equivalent feet of rock, which corresponds with the data for Friendship Heights and Tenley Circle Station at 41 to 42 equivalent feet. The line corresponding to a 0.5% probability of failure is then set at 2 feet corresponding to periphery control rock loads.

This allows an equation of the following form to be established.

$$\text{Estimated Rock Load (Feet):} \quad P(Cs, Q) = 3.297758 \times Cs^{0.969} \times Q^{-0.3926}$$

Where:

$P(Cs, Q)$ = Estimated Rock Load (Feet)

Cs = scaled crown span

Q = rock tunneling quality index

This equation was established by assuming an equation of the form $P = A \cdot Cs^B \cdot Q^C$.

Solution of the following three equations;

$$P(.8, .001) = 40 \text{ feet}$$

$$P(100, 150) = 40 \text{ feet}$$

$$P(9.8, 1000) = 2 \text{ feet}$$

leads to the establishment of the constants as follows;

$$A = 3.297758099$$

$$B = 0.969083869$$

$$C = -0.39259$$

6 ESTIMATED Q AND Cs DATA FOR DALLAS DART D2 SUBWAY STATION CAVERNS

6.1 Assumptions for DART D2 Q Values

As previously indicated, the assumed values for the Q parameters for the best, expected, and worst cases for the three dart subway station caverns have been tabulated in **Table A-1**, which is included as **Attachment 2**. It should be noted that these values are assumed early

during preliminary engineering. Resulting Q values and cavern rock loads are therefore, preliminary, and subject to change during final engineering.

In addition to the 55 degree and 65-degree joint sets mapped in the Dallas area, the near-horizontal joint set developed along bedding should also be considered. Therefore, consider a joint set number, J_n , ranging from 9 for three joint sets, to 15 for four joint sets. The joint roughness number, J_r , ranges from 2 for smooth and undulating joints to 3 for irregular and undulating. The joint alteration number, J_a , ranges from 1 for unaltered joint walls to 6 for strongly over-consolidated, non-softening clay mineral fillings. During concept engineering the worst-case values for J_a are assumed to be 6 for clay, 3 for calcite, and 6 for clay, based on borehole B-1, TS-202, and TS-111 respectively. These values must be updated on completion of further geotechnical borings during final engineering.

RQD values are taken from boreholes B1, B2, and TS-202. The ground water parameter, J_w , for the NGI-Q calculations is taken as 1 for minor inflow. The stress reduction factor, SRF, will typically range from 2.5 for low stress, near surface, open joints to 5 for Single weakness zones containing clay or chemically disintegrated rock less than 164 feet depth.

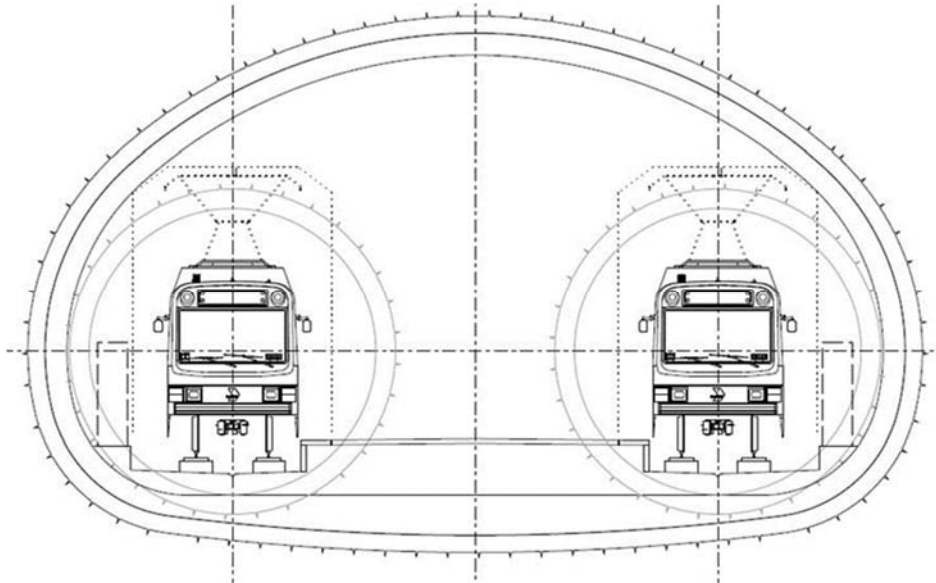
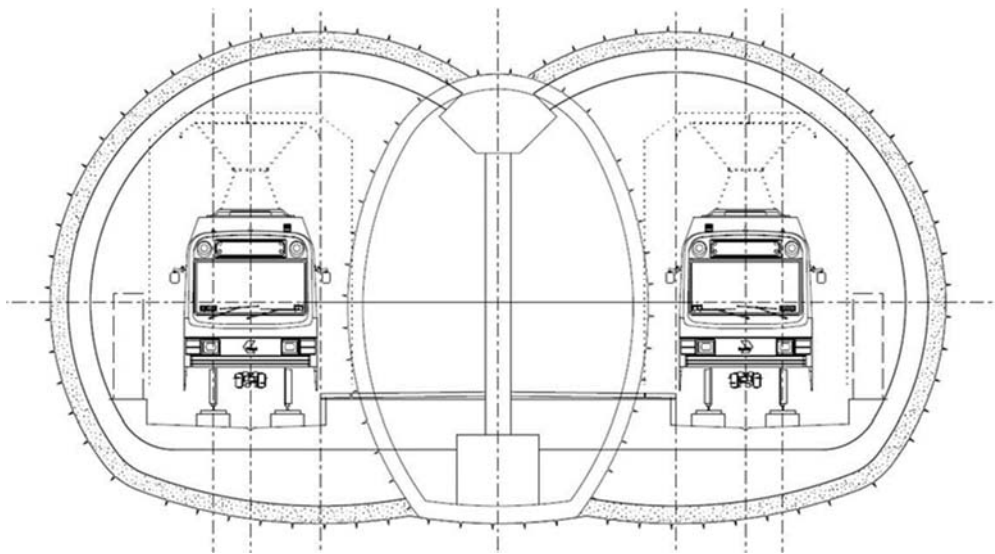
Values for Q parameters will be required to be updated upon completion of further boreholes during final engineering.

6.2 Assumptions for DART D2 Scaled Crown Span Values

The parameters for calculating the scaled crown span and the estimated rock loads are summarized in **Table A-2**, which has been included as **Attachment 3**. The rock load estimates are indicated graphically in **Figure A-1**, which is included as **Attachment 4**.

The best, expected, and worst-case conditions are plotted for several mined station cavern configurations as follows; the Commerce Station high arch, low profile arch, and binocular configurations, the double crossover cavern (deleted in the 10% South of Swiss Alignment of March 8, 2019), and the Metro Center high arch and binocular configurations (defined as cut and cover stations in the 10% South of Swiss Alignment of March 8 2019). For reference, arched and binocular cavern configurations are provided in **Figure 6-1**. As shown in this figure, the data for the DART D2 caverns plots as follows.

In general, binocular caverns tend to have lower rock loads than arched type caverns. This is due to the consideration that high arched caverns in proximity to the top of rock surface carry greater construction risks. The high rock loads associated with the worst cases represent the high rock loads that would be required to support sections of any caverns where fault zones or multiple shear zones cross the cavern geometry. However, for the two cases shown in the graph, as for the other cavern configurations analyzed, the actual depth of soil and rock above the crown is exceptionally small and will be used as the recommended rock load.

FIGURE 6-1. ARCHED AND BINOCULAR CAVERN CONFIGURATIONS*(a) Arched Mined Station Cavern Configuration**(b) Binocular Mined Station Cavern Configuration*

6.3 Implications for DART D2 Cavern Designs

The graph shown in **Figure A-1** has several implications for the DART D2 cavern designs. The end result of these implications is stated in the rock cavern final lining loads presented in **Table 7-1**. The first implication is that there is no indication in the measured cavern rock loadings of existing subway stations or in the estimated values for DART D2 subway caverns that the rock loading for DART D2 caverns would be less than 12 equivalent feet of rock (note Medical Center Station). DART D2 NGI Q values of less than 10 for all caverns result in calculated values of estimated rock loading greater than 14 feet. Much higher values of rock quality than are currently anticipated would lower the estimated rock load into the range of 2 to 7 equivalent feet for Commerce Station.

Another implication of this graph is that for the group of caverns considered, and for identical rock qualities, the increasingly less favorable crown conditions with respect to the crown arch thickness result in higher estimated rock loadings. This is the general case and remains the reason why it is important to perform a crown pillar stability analysis for a planned rock cavern in proximity to the ground surface. Near the surface, the range of rock loadings for caverns with the highest Q values ranges from 14 feet to 40 feet. This large spread in identical rock quality is due to the lack of sufficiency of the thickness of the crown arch.

A final implication concerns the region in which caving conditions would be expected without immediate rock support. The low Q values for the worst case are due to anticipated faults and joints which are expected to be present in some areas of the station caverns. Note that measured values for Friendship Heights and Tenley Circle Stations were in the range of 41 to 49 equivalent feet of rock, indicating that very high values of rock loading should be expected if the actual rock quality and scaled crown span values are in this range. Within this range the calculated estimated rock loading value may turn out to be higher than the actual combined soil and rock zone over the crown. In such cases the full overburden load is used rather than the calculated load. This is the case for Commerce Station, as indicated in **Table 7-1**, 31 to 39 equivalent feet of rock.

7 CONCLUSION

The methods for estimating rock loadings on subway station caverns has been developed over a period of eight decades, with many different approaches to the problem. Geotechnical conditions remain hidden underground and difficult to determine with sparse geotechnical borings. Nevertheless, principles can be developed to provide estimates of rock loading conditions on subway station caverns.

The minimum and maximum recommended estimated rock loads for the DART D2 subway station caverns are listed in **Table 7-1**. These values are based on geotechnical information available during the 10% design level. Currently, there is an insufficient number of boreholes relative to each underground station cavern to provide final design estimates of rock

loading. Therefore, these recommended values must be re-evaluated during final design based on additional geotechnical information.

The lack of rock cover over two of the station caverns on the original LPA alignment was significant and had a profound impact on the results of this analysis. This impact included the following;

- Cavern lining loads would approach the full loads from ground surface to the station crown.
- The Metro Center and CBD East station caverns would project above the top of rock elevations and therefore have no rock cover.
- The Metro Center and CBD East Station Caverns have been incorporated into the 10% South of Swiss Alignment, March 8, 2019 as cut and cover stations.

The original LPA alignment was therefore lowered to the level shown in the 10% South of Swiss, March 8, 2019 alignment, and the Metro Center and CBD East Stations were configured as cut and cover station. These changes mitigated these impacts resulting from the original LPA alignment.

TABLE 7-1. RECOMMENDED ROCK LOADS - 20% DESIGN

Equivalent Feet of Rock

HISTORICAL ROCK LOAD ESTIMATION METHODS FOR REFERENCE				
Method	Year	Low Rock Load (Feet)	High Rock Load (Feet)	Comment
Terzaghi Rock Load	1946	0	30	Based on rock descriptions
Deere et. al.	1970	0	60	Based on RQD, 40 ft cavern
Rose	1982	0	20	Less conservative loads based on RQD values
Cording, et. al.	1972	26	26	Based on high angle fractures such as shears and joint planes
Rock Tunneling Quality Index, Q	1974	8	27	Based on Q index value
Washington D.C. Measured Loads	1983	12	49	Actual range of values
RECOMMENDED ROCK LOADS FOR DART D2 CAVERN LININGS (10% LEVEL OF DESIGN)				
Cavern Configuration		Low Rock Load (Feet)	High Rock Loads (Feet)	Recommended DART D2 Cavern Final Lining Rock Loads
Metro Center Station – Arched		N/A	N/A	Non-practical: Do not use arched station.
Metro Center Station – Binocular		N/A	N/A	Non-practical: Crown pillar is unstable. Use full vertical load from ground surface to top of station crown.
Commerce Station – High Arch		20.3	31	10% South of Swiss Alignment, March 8, 2019
Commerce Station – Low Profile Arch		18	35	10% South of Swiss Alignment, March 8, 2019
Commerce Station – Binocular		16.4	39	10% South of Swiss Alignment, March 8, 2019
Crossover Cavern – Low Arch		33	33	Crown pillar is unstable. Use full vertical load from ground surface to top of station crown.
CBD East Station – Binocular		N/A	N/A	Non-practical: Crown pillar is unstable. Use full vertical load from ground surface to top of station crown.



8 RECOMMENDATIONS FOR PE 20% DESIGN

8.1 Design Recommendation #1

8.1.1 DESIGN RECOMMENDATION

Design rock loads for 20% design are provided in Table 7-1.

8.1.2 BASIS OF RECOMMENDATION

10% South of Swiss Alignment, March 8, 2019.

Historical empirical design methodologies as discussed above.

GDR Dallas CBD LRA DART D2, 10% Submittal, February 28, 2019.

8.1.3 SOURCES OF UNCERTAINTY

Additional boring data could indicate less or more severe geotechnical conditions than those presently available. Changes in geotechnical conditions would alter the recommended rock loads.

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10 ATTACHMENT 1: DATA FROM WASHINGTON D.C. CAVERNS (as an illustration - site specific data pending)

10.1 Medical Center Station

The predominant rock type is a blocky diorite gneiss. This rock had RQD values of 70% to 100%. The foliation is not prominent at Medical Center Station. In many sections of the station, continuous shears were absent.

Rock joints are very planar and continuous with offsets of less than .25 inches. Typical joint openings are .1 inches and many joints have slickensides and gouge coated surfaces. Face stability of the top heading was controlled by the steeply dipping joint set 4, which strikes nearly perpendicular to the station axis. The stability of the bench excavation was controlled by joint set 2, which were continuous across the excavation. The over-break in the sides of the top heading and bench excavation were controlled by joint set 3 (concurrent with foliation set 1). Most shear zones strike at high angles to the station axis and are not continuous.

Thickness of rock cover above the crown is about 40 feet, with 40 to 50 feet of decomposed rock and soil deposits above the rock.

The water table was measured about 60 to 70 feet above the station crown prior to excavation.

The strain gage data from Medical Center Station at the end of the construction period were equivalent to the radial pressure caused by the weight of 12 feet of rock across the width of the station cavern.

Based on this description, we estimate the following Q and scaled crown span parameters;

● RQD = rock quality designation	75	
● Jn = joint set number	9	
● Jr = joint roughness number	.5	
● Ja = joint alteration number	3	
● Jw = joint water reduction factor	0.5	
● SRF = stress reduction factor.	2.5	
● S = actual crown pillar span (m)	62 ft	18.9 m
● L = actual crown pillar strike length (m)	900 ft	274 m
● T = actual crown pillar thickness (m)	40 ft	12 m
● S.G.= rock specific gravity	2.6	
● θ = foliation dip (degrees)	70	
● Ht = actual cavern excavation height (m)	44 ft	13.3 m (not an input)
● D = soil depth above crown rock (m)	45 ft	13.7 m (not an input)

10.2 Rosslyn Station

Rosslyn Station is a large, near-surface metro station cavern in Washington, D. C. This cavern was excavated in hard rock by conventional drill-and-shoot methods and supported with steel ribs and shotcrete.

The predominant rock type is quartz hornblend gneiss. This rock had RQD values of 70% to 100%. (Bock, 1974) The rock is faintly foliated, blocky and jointed to occasionally seamy. Foliation is almost vertical (75 to 82 degrees) and strikes parallel to the north south station cavern axis. Rock joints are extremely smooth, continuous for many tens of feet, and coated by thin clay seams. Shears are thin, but clay filled. Most shear zones are thin, only a few are larger than 0.5 feet thick, and several of these shear zones are not continuous across the excavation. Several shears and shear zones have orientation different from the foliation or any other joint set. Thickness of rock cover above the crown varies from 40 to 60 feet and is overlain by 20 to 30 feet of weathered rock and surficial deposits.

Design phase considerations for the 82'x56'x722' station excavation ranged from support of full overburden load to some portion of the overburden load. The adopted design was W12X99 ribs at 4 foot centers which was considered adequate to support 2.8 ksf or 16 feet of rock. (Bock, 1874) The load related strains in the steel set-shotcrete arch at the end of the construction period were roughly equivalent to the radial pressures caused by the weight of 16 feet of rock. (Cording, et. al., Vol 1, 1983)

Based on this description, we estimate the following Q and scaled crown span parameters;

• RQD = rock quality designation	85		
• Jn = joint set number	12		
• Jr = joint roughness number	1		
• Ja = joint alteration number	4		
• Jw = joint water reduction factor	0.66		
• SRF = stress reduction factor.	5		
• S = actual crown pillar span (m)	80 ft	24.4 m	
• L = actual crown pillar strike length (m)	722 ft	220 m	
• T = actual crown pillar thickness (m)	50 ft	15 m	
• S.G.= rock specific gravity	3.2		
• θ = foliation dip (degrees)	78		
• Ht = actual cavern excavation height (m)	28 ft	8.5 m	(not an input)
• D = soil depth above crown rock (m)	25 ft	7.6 m	(not an input)
• UCS = peak compressive strength (psi)	10,500 – 14,500 psi		(not an input)

10.3 Bethesda Station

The predominant rock type is a blocky quartz diorite gneiss with an area of hornblend schist and chlorite bearing gneiss in the northwest of the station. This rock had RQD values of 70% to 100%. The foliation is well developed striking approximately north south (0 to 15 degrees to the right of the station axis) and dipping 50 to 70 degrees to the west. Major continuous foliation shear zones cross the station, varying in width from 3 inches to 1 foot, with one 4 feet thick on the north end of the station, and spacing from 50 to 100 feet. Joints are smooth, planar, and often coated with chlorite, with continuous joints spaced 3 to 9 feet. Foliation joints were spaced 14 inches in the vicinity of foliation shear zones. Clay filled joints were particularly prevalent in the north end of the station.

Thickness of rock cover above the crown is 10 to 30 feet, with 65 to 85 feet of weathered rock and soil deposits.

The water table was measured about 75 feet above the station crown prior to excavation. No major water flows were tapped during the excavation of the cavern.

The strain gage data from Bethesda Station at the end of the construction period were roughly equivalent to the radial pressure caused by the weight of 42 feet of rock across the width of the station cavern along most of the station cavern and 50 feet at the intersection of the lateral entranceway.

Based on this description, we estimate the following Q and scaled crown span parameters;

● RQD = rock quality designation	75	
● Jn = joint set number	12	
● Jr = joint roughness number	1	
● Ja = joint alteration number	6	
● Jw = joint water reduction factor	0.5	
● SRF = stress reduction factor.	5	
● S = actual crown pillar span (m)	62 ft	18.9 m
● L = actual crown pillar strike length (m)	800 ft	243 m
● T = actual crown pillar thickness (m)	20 ft	6 m
● S.G.= rock specific gravity	2.6	
● θ = foliation dip (degrees)	60	
● Ht = actual cavern excavation height (m)	48 ft	14.6 m (not an input)
● D = soil depth above crown rock (m)	75 ft	22.8 m (not an input)

10.4 Cleveland Park Station

The predominant rock type is a blocky granite gneiss and quartz diorite gneiss. This rock had RQD values of 70% to 100%. Rock is moderately to highly jointed, with prominent joints tight, but with occasional thin clay seams and slickensides. Joint spacing ranges up to three feet. Major shear zones were observed at the north end of the station oriented parallel to foliation, striking 0 to 7 degrees east, dipping 50 to 70 degrees. Thickness of rock cover above the crown is 12 to 50 feet, with 12 to 50 feet of decomposed rock and soil deposits. The location of the instrumented steel set is at the north end of the station, in the location where the crown pillar is 12 feet thick.

The static water table was 50 feet above the crown of the tunnel. A negligible flow of water was observed during the excavation of the pilot tunnel. There is no reference to water seepage during the construction of the top heading on the daily construction reports.

The steel ribs at the Cleveland Park Station were designed to provide the permanent support of the station without the contribution of the shotcrete lining. Because the over-break was large, the average thickness of the shotcrete was also large, probably 2 feet. The thickness of the shotcrete arch was such that the steel rib contributed only a minor portion of the lining stiffness.

The strain gage data from Cleveland Park Station at the end of the construction period were roughly equivalent to the radial pressure caused by the weight of 23 feet of rock across the width of the station cavern.

Based on this description, we estimate the following Q and scaled crown span parameters;

● RQD = rock quality designation	75		
● Jn = joint set number	12		
● Jr = joint roughness number	3		
● Ja = joint alteration number	2		
● Jw = joint water reduction factor	1		
● SRF = stress reduction factor.	7.5		
● S = actual crown pillar span (m)	58 ft	17.8 m	
● L = actual crown pillar strike length (m)	900 ft	274 m	
● T = actual crown pillar thickness (m)	12 ft	3.6 m	
● S.G.= rock specific gravity	2.8		
● θ = foliation dip (degrees)	60		
● Ht = actual cavern excavation height (m)	44 ft	13 m	(not an input)
● D = soil depth above crown rock (m)	50 ft	15 m	(not an input)

10.5 Van Ness Station

The predominant rock type is Kensington granite gneiss. This rock had RQD values of 70% to 100%. The rock is jointed and blocky, and locally blocky and seamy. Rock is jointed, with prominent joints tight, but with occasional thin clay seams and slickensides. Joint spacing ranges up to three feet. Foliation is weakly to moderately developed, with foliation joints smooth, continuous, and spaced up to 6 feet apart. Clay gouge is common in low angle and foliation joints. There are several major shears, greater than 1 foot thick along the cavern axis. Some contained zones of crushed rock with clay gouge. Many minor shears and soft weathered zones are parallel to joint sets at orientations different from foliation.

Several sets of steeply dipping shears (50 to 60 degrees) were encountered in the south end of the station, the southwest shaft, and the lateral pilot tunnel.

Thickness of rock cover above the crown is 20 to 65 feet, with 65 to 70 feet of overburden above the crown of the cavern. The location of the instrumented steel set is at the north end of the station, in the location where the crown pillar scales approximately 38 feet thick.

The static water table was measured 45 feet above the crown of the tunnel prior to construction. A slight water inflow was observed during excavation of the pilot tunnel. However, strong seepage was found at the north end of the pilot tunnel construction. There is no reference to water seepage during the construction of the top heading on the daily construction reports.

The steel ribs at the Van Ness Station were designed to provide the permanent support of the station without the contribution of the shotcrete lining.

The strain gage data from Van Ness Station at the end of the construction period were roughly equivalent to the radial pressure caused by the weight of 23 feet of rock across the width of the station cavern.

Based on this description, we estimate the following Q and scaled crown span parameters;

● RQD = rock quality designation	75	
● Jn = joint set number	15	
● Jr = joint roughness number	2	
● Ja = joint alteration number	3	
● Jw = joint water reduction factor	0.66	
● SRF = stress reduction factor.	5	
● S = actual crown pillar span (m)	58 ft	17.8 m
● L = actual crown pillar strike length (m)	900 ft	274 m
● T = actual crown pillar thickness (m)	38 ft	11.6 m
● S.G.= rock specific gravity	2.8	



- θ = foliation dip (degrees) 55
- H_t = actual cavern excavation height (m) 44 ft 13 m (not an input)
- D = soil depth above crown rock (m) 32 ft 10 m (not an input)

10.6 Zoological Park Station

The predominant rock type is a blocky schistose gneiss, becoming more schistose toward the north end of the station. This rock had RQD values of 70% to 100%. Foliation is moderately to well developed, with some joints formed along the foliation trends. The rock is blocky and also seamy in shear zones. Foliation dip averages 60 degrees. Rock joints are planar, with a small degree of roughness, continuous, the majority of which contain thin coatings of quartz, calcite, and clay. Joint spacing ranges up to several feet. Two major shear zones were encountered with thicknesses ranging from 2 to 30 feet. These shear zones dipped 50 to 65 degrees from horizontal and ran at a varying angle to the cavern axis. Thickness of rock cover above the crown varies from 50 to 70 feet and is overlain by 30 to 50 feet of decomposed rock and soil deposits.

The steel ribs at the Zoological Park Station were designed to provide the permanent support of the station without the contribution of the shotcrete lining. The strain gage data from Zoological Park Station could not be correlated in a meaningful way with the construction history but load related strains in the steel set-shotcrete arch at the end of the construction period were consistent with the radial pressure caused by the weight of 30 feet of rock across the width of the station cavern.

Based on this description, we estimate the following Q and scaled crown span parameters;

● RQD = rock quality designation	75		
● Jn = joint set number	12		
● Jr = joint roughness number	3		
● Ja = joint alteration number	3		
● Jw = joint water reduction factor	0.66		
● SRF = stress reduction factor.	5		
● S = actual crown pillar span (m)	58 ft	17.7 m	
● L = actual crown pillar strike length (m)	800 ft	244 m	
● T = actual crown pillar thickness (m)	60 ft	18 m	
● S.G.= rock specific gravity	2.7		
● θ = foliation dip (degrees)	60		
● Ht = actual cavern excavation height (m)	44 ft	13 m	(not an input)
● D = soil depth above crown rock (m)	40 ft	12 m	(not an input)

10.7 DuPont Circle Station

DuPont Circle Station was among the first two stations constructed in rock in the Washington D. C. metro system. This station was heavily instrumented so that information could be provided for design verification of the station cavern as well as provide relevant displacement and rock load data for the construction of subsequent caverns.

The predominant rock type is mica-quartz schist. This rock had RQD values of 50% to 100%. (Cording, et. al., Vol 3, p. 2, 1983) The rock is very blocky and seamy. Foliation dips 50 to 60 degrees west and strikes sub-parallel to the station cavern axis. Rock joints are sub-parallel to the foliation, smooth, slickensided, or slightly wavy, continuous, and may be gouge filled. Gouge filling in three of the four joint sets was most common in the vicinity of foliation shear zones. Eight major shear zones, gouge filled, were mapped during construction. Shear zones are 1 to 5 feet thick, continuous across the excavation, and spaced 5 to 15 feet apart. Shear zones are typically oriented along the foliation. Thickness of rock cover above the crown varies from 33 feet and is overlain by 37 feet of weathered rock and surficial deposits.

Load related strains in the steel set-shotcrete arch at the end of the construction period were roughly equivalent to the radial pressures caused by the weight of 30 feet of rock.

Based on this description, we estimate the following Q and scaled crown span parameters;

● RQD = rock quality designation	75	
● Jn = joint set number	15	
● Jr = joint roughness number	1.5	
● Ja = joint alteration number	5	
● Jw = joint water reduction factor	0.66	
● SRF = stress reduction factor.	7.5	
● S = actual crown pillar span (m)	76 ft	23 m
● L = actual crown pillar strike length (m)	724 ft	221 m
● T = actual crown pillar thickness (m)	30 ft	9 m
● S.G.= rock specific gravity	2.7	
● θ = foliation dip (degrees)	55	
● Ht = actual cavern excavation height (m)	44 ft	13 m (not an input)
● D = soil depth above crown rock (m)	35 ft	10.6 m (not an input)

10.8 Tenley Circle Station

The predominant rock type is a blocky and seamy dark gray schistose gneiss with prominent foliation. This rock had RQD values of 70% to 100%. The numerous major continuous foliation shear zones cross the station at a strike of N5 deg E (25 degrees from the station axis) and dip 70 to 80 degrees to the west, varying in width from 6 inches to 6 feet, and their spacing from 30 to 50 feet. Conjugate shears strike N 40 deg E (60 degrees to the right of the station cavern axis) and dip 35 to 45 degrees to the southeast and are spaced 20 to 60 degrees apart. Rock outside the shear zones is sound, but occasionally weathered along joint planes. Foliation joints are iron stained and may be filled with weathered gouge.

Thickness of rock cover above the crown is 60 to 70 feet, with 25 to 35 feet of weathered rock and soil deposits. The location of the instrumented steel sets is at the south end of the station.

A flow of water was observed during the excavation of the pilot tunnel at the north end of the station. Water flow from the crown of the station was observed at the north end of the station during the top heading excavation.

The strain gage data from Tenley Circle Station at the end of the construction period were roughly equivalent to the radial pressure caused by the weight of 28 feet of rock across the width of the station cavern along most of the station cavern and 41 feet at the intersection of the lateral entranceway.

Based on this description, we estimate the following Q and scaled crown span parameters;

● RQD = rock quality designation	75		
● Jn = joint set number	15 (Main Station)	45 (Intersection)	
● Jr = joint roughness number	2.5		
● Ja = joint alteration number	2		
● Jw = joint water reduction factor	0.66		
● SRF = stress reduction factor.	7.5		
● S = actual crown pillar span (m)	60 ft	18.3 m	
● L = actual crown pillar strike length (m)	800 ft	244 m	
● T = actual crown pillar thickness (m)	65 ft	19.8 m	
● S.G.= rock specific gravity	2.6		
● θ = foliation dip (degrees)	75		
● Ht = actual cavern excavation height (m)	44 ft	13 m	(not an input)
● D = soil depth above crown rock (m)	30 ft	9.1 m	(not an input)

10.9 Friendship Heights Station

The predominant rock type is a blocky, seamy, and often slabby, diorite gneiss and schistose gneiss with weathering along foliation surfaces. This rock had RQD values of 70% to 100%. The numerous major continuous foliation shear zones cross the station at a strike of N 5 - 10 deg E (20 degrees to the right of the station axis) and dip 70 degrees to the west, varying in width from 2 inches to 7 feet, and their spacing from 30 to 50 feet. Some of these shear zones have 4 inches of clay gouge. Two other shears strike N 82 deg E (65 degrees to the left of the station cavern axis) and dip 70 degrees to the north. Foliation rock joints are smooth and wavy with an amplitude of 4 inches over a wavelength of 3 to 10 feet. These joints are very pronounced, often sheared, slickensided, with red iron oxide staining.

Thickness of rock cover above the crown is 40 to 60 feet, with 18 to 26 feet of weathered rock and soil deposits.

The water table was measured 53 to 65 feet above the station crown prior to excavation. A flow of water was observed during the excavation of the pilot tunnel and during the construction of the top heading.

The strain gage data from Friendship Heights Station at the end of the construction period were roughly equivalent to the radial pressure caused by the weight of 42 feet of rock across the width of the station cavern along most of the station cavern and 50 feet at the intersection of the lateral entranceway.

Based on this description, we estimate the following Q and scaled crown span parameters;

● RQD = rock quality designation	75		
● Jn = joint set number	15 (Main Station)	45 (Intersection)	
● Jr = joint roughness number	1.5		
● Ja = joint alteration number	6		
● Jw = joint water reduction factor	0.66		
● SRF = stress reduction factor.	10		
● S = actual crown pillar span (m)	67 ft	20.4 m	
● L = actual crown pillar strike length (m)	950 ft	289 m	
● T = actual crown pillar thickness (m)	50 ft	15.2 m	
● S.G.= rock specific gravity	2.6		
● θ = foliation dip (degrees)	70		
● Ht = actual cavern excavation height (m)	50 ft	15 m	(not an input)
● D = soil depth above crown rock (m)	22 ft	6.7 m	(not an input)



11 ATTACHMENT 2: TABLE A-1: ASSUMED NGI – Q PARAMETERS FOR DART D2 CAVERN STATIONS

TABLE A-1: ASSUMED NGI – Q PARAMETERS FOR DART D2 CAVERN STATIONS

		Assumed NGI – Q Parameters for DART D2 Station Caverns		
		Worst Case	Expected Case	Best Case
Metro Center Station				
(See Borehole B-1)	RQD	73	90	100
	Jn	15	12	9
	Jr	1	1.5	2
	Ja	6	2	1
	Jw	1	1	1
	SRF	5	2.5	2.5
	Q	0.162	2.25	8.889
Commerce Station				
(See Borehole TS-202)	RQD	93	97	100
	Jn	15	12	9
	Jr	0.5	1.5	2
	Ja	3	2	1
	Jw	1	1	1
	SRF	5	2.5	2.5
	Q	0.207	2.425	8.889
CBD East Station				
(See Borehole B-2)	RQD	74	97	100
	Jn	15	12	9
	Jr	1.5	1.5	2
	Ja	6	2	1
	Jw	1	1	1
	SRF	5	2.5	2.5
	Q	0.247	2.425	8.889



12 ATTACHMENT 3: TABLE A-2: CROWN PILLAR STABILITY ANALYSIS

DART D2 - Subway Station Cavern Crown Pillar Analysis (Compared to WMATA Cavern Data)															
Station Cavern Name	Station Cavern Configuration	Borehole Number	Ground Elevation	Top of Rock Elevation	Track TOR Elevation	Cavern Height above TOR	Cavern Width	Cavern Length	Crown Lining Thickness	Rock Type	Rock Quality Expectation	Rock Specific Gravity	Bedding or Foliation Dip Angle	Rock Quality Designation	Joint Set Number
							S	L							
		(#)	(Feet)	(Feet)	(Feet)	(Feet)	Note 1	(Feet)	(Feet)	(Feet)			(Degrees)	RQD	Jn
Metro Center Station (DART D2)															
Metro Center Station	Binocular	B-1	428.37	403.37	367	25.75		400	2	Austin Chalk	Best	2.14	5	100	9
Metro Center Station	Binocular	B-1	428.37	403.37	367	25.75		400	2	Austin Chalk	Expected	2.14	5	90	12
Metro Center 367.1 to 360.1	Binocular	B-1	428.37	403.37	367	25.75		400	2	Austin Chalk	Worst	2.14	5	73	15
Metro Center Station	Binocular	TS-104	427.71	402.71	367	25.75		400	2	Austin Chalk	Best	2.14	5	100	9
Metro Center Station	Binocular	TS-104	427.71	402.71	367	25.75		400	2	Austin Chalk	Expected	2.14	5	95	12
Metro Center Station	Binocular	TS-104	427.71	402.71	367	25.75		400	2	Austin Chalk	Worst	2.14	5	90	15
Metro Center Station	Low Profile Arch	B-1	428.37	403.37	367	29.75		400	2	Austin Chalk	Best	2.14	5	100	9
Metro Center Station	Low Profile Arch	B-1	428.37	403.37	367	29.75		400	2	Austin Chalk	Expected	2.14	5	90	12
Metro Center Station	Low Profile Arch	B-1	428.37	403.37	367	29.75		400	2	Austin Chalk	Worst	2.14	5	73	15
Commerce Street Station (DART D2)															
Commerce Street Station	Binocular	TS-202	427	411	360	25.75		650	2	Austin Chalk	Best	2.14	5	100	9
Commerce Street Station	Binocular	TS-202	427	411	360	25.75		650	2	Austin Chalk	Expected	2.14	5	97	12
Commerce Street Station	Binocular	TS-202	427	411	360	25.75		650	2	Austin Chalk	Worst	2.14	5	93	15
Commerce Street Station	Low Profile Arch	TS-202	427	411	360	29.75		650	2	Austin Chalk	Best	2.14	5	100	9
Commerce Street Station	Low Profile Arch	TS-202	427	411	360	29.75		650	2	Austin Chalk	Expected	2.14	5	97	12
Commerce Street Station	Low Profile Arch	TS-202	427	411	360	29.75		650	2	Austin Chalk	Worst	2.14	5	93	15
Commerce Street Station	High Arch	TS-202	427	411	360	33.92		650	2	Austin Chalk	Best	2.14	5	100	9
Commerce Street Station	High Arch	TS-202	427	411	360	33.92		650	2	Austin Chalk	Expected	2.14	5	97	12
Commerce Street Station	High Arch	TS-202	427	411	360	33.92		650	2	Austin Chalk	Worst	2.14	5	93	15
Crossover Cavern (Optional East of Commerce Station) (DART D2)															
Crossover Cavern	Low Arch	TS-202	440	411	378	27		900	2	Austin Chalk	Best	2.14	5	100	9
Crossover Cavern	Low Arch	TS-202	440	411	378	27		900	2	Austin Chalk	Expected	2.14	5	97	12
Crossover Cavern	Low Arch	TS-202	440	411	378	27		900	2	Austin Chalk	Worst	2.14	5	93	15
CBD East Station (DART D2)															
CBD East Station	Binocular	TS-209	465	427	428	25.75		400	2	Austin Chalk	Best	2.14	5	100	9
CBD East Station	Binocular	TS-209	465	427	428	25.75		400	2	Austin Chalk	Expected	2.14	5	99	12
CBD East Station	Binocular	TS-209	465	427	428	25.75		400	2	Austin Chalk	Worst	2.14	5	98	15
CBD East Station	Low Profile Arch	TS-209	465	427	428	29.75		400	2	Austin Chalk	Best	2.14	5	100	9
CBD East Station	Low Profile Arch	TS-209	465	427	428	29.75		400	2	Austin Chalk	Expected	2.14	5	99	12
CBD East Station	Low Profile Arch	TS-209	465	427	428	29.75		400	2	Austin Chalk	Worst	2.14	5	98	15

Station Cavern Configuration	Joint Rough- ness Number	Joint Alter- ation Number	Joint Water Factor	Stress Reduction Factor	Rock Tunneling Quality Index	Station Crown Elevation	Actual Depth of Overhead Soil and Rock	Actual Crown Pillar Thickness	Scaled Crown Span	Scaled Crown Span	Recommend- ed Minimum Crown Pillar Thickness	Recommend- ed Minimum Crown Pillar Thickness	Critical Span	Critical Span	Crown Pillar Probability of Failure	Measured Cavern Rock Loads	Estimated Cavern Rock Loads	Estimated Cavern Rock Loads
	Jr	Ja	Jw	SRF	Q		H	T	Cs	Cs	Tmin	Tmin	Sc	Sc	Pf		A= 0.1 cs exp= 1.63 q exp= -0.79	3.297758099 0.969083869 -0.39259
									Note 2 (1992)	Note 2 (1992)	Note 3 (1992)	Note 3 (1992)	Note 4 (2008)	Note 4 (2008)	Note 5 (2014)	Note 6 (1983)	Pcs,q Note 7 (2007)	Pcs,q Note 8 (2018)
						(Feet)	(Feet)	(Feet)	(Meters)	(Feet)	(Meters)	(Feet)	(Meters)	(Feet)	(%)	(Feet)	(Feet)	(Feet)
Metro Center Station																		
Binocular	2	1	1	2.5	8.89	395	33	8.37	20.5	67	3.4	12	9.4	31	99.9	-----	2.4	26.1
Binocular	1.5	2	1	2.5	2.25	395	33	8.37	20.5	67	4.4	15	5.1	17	100	-----	7.2	33.0
Binocular	1	6	1	5	0.16	395	33	8.37	20.5	67	7.2	24	1.6	6	100	-----	33.0	33.0
Binocular	2	1	1	2.5	8.89	395	33	7.71	21.4	70	3.4	12	9.4	31	100	-----	2.6	27.2
Binocular	1.5	2	1	2.5	2.38	395	33	7.71	21.4	70	4.3	14	5.2	18	100	-----	7.4	33.0
Binocular	1	6	1	5	0.2	395	33	7.71	21.4	70	6.9	23	1.8	6	100	-----	33.0	33.0
Low Profile Arch	2	1	1	2.5	8.89	399	29	4.37	28.4	93	3.4	12	9.4	31	100	-----	4.2	29.0
Low Profile Arch	1.5	2	1	2.5	2.25	399	29	4.37	28.4	93	4.4	15	5.1	17	100	-----	12.3	29.0
Low Profile Arch	1	6	1	5	0.16	399	29	4.37	28.4	93	7.2	24	1.6	6	100	-----	29.0	29.0
Commerce Station																		
Binocular	2	1	1	2.5	8.89	388	39	23	12.7	42	3.4	12	9.4	31	89.7	-----	1.1	16.4
Binocular	1.5	2	1	2.5	2.43	388	39	23	12.7	42	4.3	14	5.3	18	100	-----	3.1	27.3
Binocular	0.5	3	1	5	0.21	388	39	23	12.7	42	6.9	23	1.8	6	100	-----	21.6	39.0
Low Profile Arch	2	1	1	2.5	8.89	392	35	19	14	46	3.4	12	9.4	31	95.3	-----	1.3	18.0
Low Profile Arch	1.5	2	1	2.5	2.43	392	35	19	14	46	4.3	14	5.3	18	100	-----	3.7	30.0
Low Profile Arch	0.5	3	1	5	0.21	392	35	19	14	46	6.9	23	1.8	6	100	-----	25.3	35.0
High Arch	2	1	1	2.5	8.89	396	31	15	15.8	52	3.4	12	9.4	31	98.5	-----	1.6	20.3
High Arch	1.5	2	1	2.5	2.43	396	31	15	15.8	52	4.3	14	5.3	18	100	-----	4.5	31.0
High Arch	0.5	3	1	5	0.21	396	31	15	15.8	52	6.9	23	1.8	6	100	-----	30.9	31.0
Crossover Cavern (Commerce Street)																		
Low Arch	2	1	1	2.5	8.89	407	33	4	30.7	101	3.4	12	9.4	31	100	-----	4.7	33.0
Low Arch	1.5	2	1	2.5	2.43	407	33	4	30.7	101	4.3	14	5.3	18	100	-----	13.2	33.0
Low Arch	0.5	3	1	5	0.21	407	33	4	30.7	101	6.9	23	1.8	6	100	-----	33.0	33.0
CBD East Station																		
Binocular	2	1	1	2.5	8.89	456	9	-29	NA	NA	3.4	12	9.4	31	NA	-----	NA	NA
Binocular	1.5	2	1	2.5	2.48	456	9	-29	NA	NA	4.3	14	5.3	18	NA	-----	NA	NA
Binocular	1.5	6	1	5	0.33	456	9	-29	NA	NA	6.3	21	2.2	8	NA	-----	NA	NA
Low Profile Arch	2	1	1	2.5	8.89	460	5	-33	NA	NA	3.4	12	9.4	31	NA	-----	NA	NA
Low Profile Arch	1.5	2	1	2.5	2.48	460	5	-33	NA	NA	4.3	14	5.3	18	NA	-----	NA	NA
Low Profile Arch	1.5	6	1	5	0.33	460	5	-33	NA	NA	6.3	21	2.2	8	NA	-----	NA	NA

DART D2 - Subway Station Cavern Crown Pillar Analysis (Compared to WMATA Cavern Data)															
Station Cavern Name	Station Cavern Configuration	Borehole Number	Ground Elevation	Top of Rock Elevation	Track TOR Elevation	Cavern Height above TOR	Cavern Width	Cavern Length	Crown Lining Thickness	Rock Type	Rock Quality Expectation	Rock Specific Gravity	Bedding or Foliation Dip Angle	Rock Quality Designation	Joint Set Number
							S	L				S.G.	θ	RQD	Jn
		(#)	(Feet)	(Feet)	(Feet)	(Feet)	(Feet)	(Feet)	(Feet)				(Degrees)		
Washington D. C. Caverns (WMATA)															
Medical Center Station	Arched						62	900		Gneiss		2.6	70	75	9
Rosslyn Station	Arched						80	722		Gneiss		3.2	78	85	12
Bethesda Station	Arched						62	800		Gneiss		2.6	60	75	12
Cleveland Park Station	Arched						58	900		Gneiss		2.8	60	75	12
Van Ness Station	Arched						58	900		Gneiss		2.8	55	75	15
Zoological Park Station	Arched						58	800		Gneiss		2.7	60	75	12
Dupont Circle Station	Arched						76	724		Schist		2.7	55	75	15
Tenley Circle Station	Arched						60	800		Gneiss		2.6	75	75	15
Tenley Circle Station	Intersection						60	800		Gneiss		2.6	75	75	45
Friendship Heights Station	Arched						67	950		Gneiss		2.6	70	75	15
Friendship Heights Station	Intersection						67	950		Gneiss		2.6	70	75	45
Notes															
Note 1 - Equations are taken from references cited.															
Note 2 - Scaled Crown Span (1992) is given by $Cs = S \times [S.G. / (T (1+S/L) (1-0.4 \cos \theta))] 0.5$.															
Note 3 - Minimum Crown Pillar Thickness (1992) is given by $Tmin = 5.11 \times Q^{-0.19} \times [\sinh 0.0016 (Q)]$.															
Note 4a - Critical Span (2008) is given by $Sc = 3.58 Q^{.44}$.															
Note 4b - This critical span is the span at which 50% of crown pillars are expected to fail if unsupported during excavation.															
Note 5 - The crown pillar probability of failure is given by $Pf = 100/[1+440 \times \exp(-1.7 \times Cs/Q 0.44)]$.															
Note 6 - Source of measured rock load data: (Cording, et. al, Vol. 1, Oct. 1983).															
Note 7 - The original equation (2007) was in the form $P(Cs,Q) = 0.1 Cs^{1.6} \times Q^{-0.79}$.															
Note 8 - The current equation (2018) is in the form $P(Cs,Q) = 3.29776 Cs^{0.96908} \times Q^{-0.39259}$.															
Note 9 - If the calculated rock load exceeds the overburden depth, the overburden depth is used.															
Note 10 - Metro Center Station and CBD East Station have been raised at 10% to allow cut and cover station excavation.															
Note 11 - Metro Center and CBD East data is included to indicate required depth of lowering to allow rock cavern excavation.															

Station Cavern Configuration	Joint Roughness Number	Joint Alteration Number	Joint Water Factor	Stress Reduction Factor	Rock Tunneling Quality Index	Station Crown Elevation	Actual Depth of Overhead Soil and Rock	Actual Crown Pillar Thickness	Scaled Crown Span	Scaled Crown Span	Recommended Minimum Crown Pillar Thickness	Recommended Minimum Crown Pillar Thickness	Critical Span	Critical Span	Crown Pillar Probability of Failure	Measured Cavern Rock Loads	Estimated Cavern Rock Loads	Estimated Cavern Rock Loads
	Jr	Ja	Jw	SRF	Q		H	T	Cs	Cs	Tmin	Tmin	Sc	Sc	Pf		A= 0.1 cs exp= 1.63 q exp= -0.79	3.297758099
									Note 2 (1992)	Note 2 (1992)	Note 3 (1992)	Note 3 (1992)	Note 4 (2008)	Note 4 (2008)	Note 5 (2014)	Note 6 (1983)	Pcs,q Note 7 (2007)	Pcs,q Note 8 (2018)
						(Feet)	(Feet)	(Feet)	(Meters)	(Feet)	(Meters)	(Feet)	(Meters)	(Feet)	(%)	(Feet)	(Feet)	(Feet)
Washington D.C. Caverns																		
Arched	5	3	0.5	2.5	2.78		80	40	9.1	30	4.2	14	5.6	19	97.8	12	1.6	18.8
Arched	1	4	0.66	5	0.23		65	50	11.1	36	6.7	22	1.9	7	100	16	16.1	60.5
Arched	1	6	0.5	5	0.1		90	20	13.3	44	7.9	26	1.3	5	100	20	41.9	90.0
Arched	3	2	1	7.5	1.25		60	12	16.8	55	4.9	16	3.9	13	100	23	8.3	46.5
Arched	2	3	0.66	5	0.44		65	38	9.6	31	6	20	2.5	9	100	23	7.6	40.7
Arched	3	3	0.66	5	0.83		100	60	7.3	24	5.3	18	3.3	11	99.9	30	3.0	24.4
Arched	1.5	5	0.66	7.5	0.13		60	30	13.6	45	7.5	25	1.5	5	100	30	35.3	60.0
Arched	2.5	2	0.66	7.5	0.55		85	65	6.7	22	5.7	19	2.8	10	100	28	3.6	26.3
Intersection	2.5	2	0.66	7.5	0.18		85	65	6.7	22	7.1	24	1.7	6	100	41	8.6	40.8
Arched	1.5	6	0.66	10	0.08		70	50	8.8	29	8.2	27	1.2	4	100	42	25.5	70.0
Intersection	1.5	6	0.66	10	0.03		70	50	8.8	29	9.9	33	0.8	3	100	49	55.3	70.0



13 ATTACHMENT 4: FIGURE A-1: ROCK LOADS FOR DART D2 STATION CAVERNS

